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30 April 2024

Town Planning Group
1 Coronet Peak Station Road
PO Box 2559
Queenstown

Our reference: 503073

Attention: Brett Giddens and Sam Kealey

Dear Brett and Sam

Infrastructure Support Letter for Silver Creek Fast-Track Submission

1. Introduction

We write to provide you with a supporting letter for the Silver Creek Development to form part of your Fast-Track submission to the Ministry for the Environment. This letter specifically comments on the servicing of the development with respect to:

- Stormwater;
- Wastewater;
- Potable water;
- Electricity supply; and
- Telecommunication supply.

1.1. The Current Development

The Silver Creek development is a residential subdivision located on Queenstown Hill between Frankton and the CBD, some 80m to 200m above and overlooking Lake Wakatipu. Refer to Appendix A for the Master Plan for the development.

We note a Resource Consent (RM 210908) has already been issued by the Queenstown Lakes District Council (QLDC) for the development, which illustrates the sites suitability for residential development. The project has now moved into a detailed design phase where Engineering Approval design packages and further Resource Consents are being prepared and are due to be submitted to QLDC imminently for their review and acceptance. Following approval by QLDC, physical construction works will commence.

1.2. Eliot Sinclair's Role

Eliot Sinclair have multiple roles in the development, comprising:

- Project managers for the entire development;
- Design civil engineers; and
- Surveyors.

Eliot Sinclair has over 90 years' experience and success in managing, designing, and delivering various subdivision developments across the South Island, and we are proud to be involved the in the Silver Creek development where it will yield a significant number of residential units in a region that has a need for new housing.

2. Development Servicing and Feasibility to Support a Fast Track Application

2.1. Proposed Master Plan

The Master Plan in Appendix A summarises the site and illustrates intended staging and proposed densities which can range between 580 to ~1000 units. There are a mix of housing topologies from standalone houses and 1-2 bedroom apartments, through to workers accommodation villages. The large variation in yield is mainly due to the workers accommodation which can be increased or decreased based on market demand, and can be adjusted by altering the height and density of the buildings.

2.2. Previous Reporting and Applicability

The following sections summarize how the attached Master Plan is to be serviced based on the information submitted as part of the subdivision consent process (RM 210908) and Eliot Sinclair's knowledge obtained as part of the detailed design process for the development.

2.2.1. Stormwater

As part of the subdivision consenting process (RM 210908), AWA previously reported on the sites feasibility to treat and discharge stormwater from the development in accordance with best practice and QLDC's Engineering Code of Practice. We have attached AWA's reporting as **Appendix B**.

Essentially, AWA indicated that any stormwater is to be conveyed through conventional stormwater networks (i.e. pipes, open swales etc.) to appropriately sized stormwater basins predominantly located within the existing Silver Creek waterway. From the basins, the stormwater is to be discharged into the Silver Creek waterway at controlled rates similar to pre-development flows to ensure there are no detrimental downstream effects. Ultimately, the treated stormwater discharges to Lake Wakatipu.

We believe the stormwater management strategy proposed by AWA is feasible subject to further investigation and detailed design.

2.2.2. Wastewater

Hydraulic Analysis Ltd (HAL) were engaged by the developers to report on wastewater solutions for the development as part of the subdivision consent process (RM 210908). We attach their reporting to this letter as **Appendix C**.

HAL have indicated in their reports that the existing Frankton Tack gravity sewer in which the proposed Silver Creek development ultimately discharges into has capacity constraints. We are aware QLDC have near term intentions (circa next 2 years) to install a secondary rising main (proposed Frankton Track Rising main) along Frankton Road / SH6a to reduce the load on the existing Frankton Tack gravity sewer and increase its capacity. HAL has recommended QLDC to investigate options to reduce peak flows and mitigate the risk of overflows from the existing Frankton Track gravity sewer as part of the design of the proposed Frankton Track Rising Main to ensure spare capacity is available in the existing Frankton Track gravity sewer for future development. HAL have also indicated in their report that there'll be sufficient capacity in the Council's wastewater system for the full development once the downstream constraints are resolved (proposed Frankton Tracking Rising Main is constructed and their pump stations are operating simultaneously), as envisaged in QLDC Wastewater Master Plan (2020).

QLDC has indicated that a short-term solution (until such time the Frankton Track rising main has been installed) is to allow up to 150 new lots from the Silver Creek development to discharge wastewater to their existing network via an attenuation tank and a restricted flow outlet. Beyond the 150 lots, Silver Creek in conjunction with QLDC will need to investigate options to reduce peak flows so that discharge of wastewater into the existing Frankton Track gravity sewer is possible should there be a delay of the installation of the proposed Frankton Track Rising Main. We envisage any interim wastewater attenuation system(s) can be decommissioned once QLDC is satisfied that their wastewater system upgrade is complete and can accommodate the unattenuated flows from the development.

We believe that the Silver Creek Development can be serviced with wastewater in the long-term provided that QLDC undertake the recommendations by HAL to investigate and optimise their wastewater network as part of the design of the proposed Frankton Track Rising Main. We believe the interim wastewater attenuation system proposed by HAL to provide a short-term solution to service up to 150 lots is feasible subject to further investigation and detailed design. In the event that the QLDC wastewater network upgrade is not complete by the time that more than 150 lots are required to be developed, we believe there are interim options for the development to discharge wastewater into the existing network which will need to be further investigated.

2.2.3. Potable Water

Watershed Engineering Ltd (WSE) were engaged by the developers as part of the subdivision consent process (RM 210908) to complete potable water modelling for the Silver Creek development. The modelling was required to illustrate how any existing QLDC water infrastructure could service the development, and what new infrastructure (if any) would be required to meet the expected demands of the 580 potential lots. WSE reports are attached to this letter as **Appendix D**.

WSE have indicated in their report that Silver Creek Development can be serviced with potable water by limiting the flow of water from the existing Middleton Road watermain to 150 lots and by changing the valve arrangement on the existing Frankton Road watermain to service the full Silver Creek Development (beyond 150 lots) under a low flow demand scheme (250 L/s/day). As indicated in the WSE report, new reservoir(s) and pump station(s) are required to supply adequate pressures to Silver Creek Development. It should be noted that the change of valves to enable the Silver Creek Development has already been undertaken and detailed design of the water supply scheme is underway.

We believe the water supply scheme proposed by WSE is feasible subject to further investigation and detailed design.

2.2.4. Power Supply

As part of the subdivision consent process, PowerNet on behalf of Lakeland Network confirmed the existing network has adequate capacity to service the proposed Silver Creek development.

We attach their confirmation letter as Appendix E.

2.2.5. Telecommunications Supply

The developers confirmed with Chorus as part of the subdivision consent process that development could be serviced with fibre, and detailed design of the development is well underway.

2.3. Eliot Sinclair's Current Detailed Design

Eliot Sinclair is currently undertaking detailed design of the development with the intent to submit Engineering Approval packages to QLDC imminently. We can confirm that the recommendations of the various consultants mentioned above in 2.2.1; 2.2.2; and 0 is generally achievable and the development is serviceable subject to some minor adjustments and/or efficiencies. Our detailed design is also subject to the approval of QLDC as part of the Engineering Approval process.

Eliot Sinclair is also liaising with power, streetlight, and telecommunications (fibre) suppliers to commence with their respective detail designs to provide adequate servicing to and within the development. These suppliers have not expressed any concerns or identified any constraints.

3. Conclusion/Summary

Eliot Sinclair is excited to be part of this development offering at least 580 units (and possibly up to ~1000 units) to the Queenstown region. We and the various suppliers are well underway with detailed design, specifically the stormwater; wastewater; potable water; power; and telecommunications (fibre), and we have not encountered any significant constraints that would inhibit the development (as shown on the Master Plan) from proceeding. Additionally, Eliot Sinclair intend to submit the first Engineering Approval package to QLDC in the immediate future.

If any of the above requires clarification, or if you have further questions, please do not hesitate to get in touch.

Yours sincerely

Jenny Bull
3 Waters Engineer
BE(Hons) Civil CMEngNZ CPEng

Ryan Mulligan
Surveyor | Director
BSurv MS+SNZ LCS

Appendix A. Scheme Plan

SITE LANDSCAPE ARCHITECTS ^

SILVER CREEK

MASTERPLAN SHEET 1

CONCEPT_ 1:3,000 @ A3

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Appendix B. AWA's Stormwater Report

Hydrology and Hydraulic Assessment

Date 22/09/2022

Excellence Heart Innovation

HYDROLOGY AND HYDRAULIC ASSESSMENT

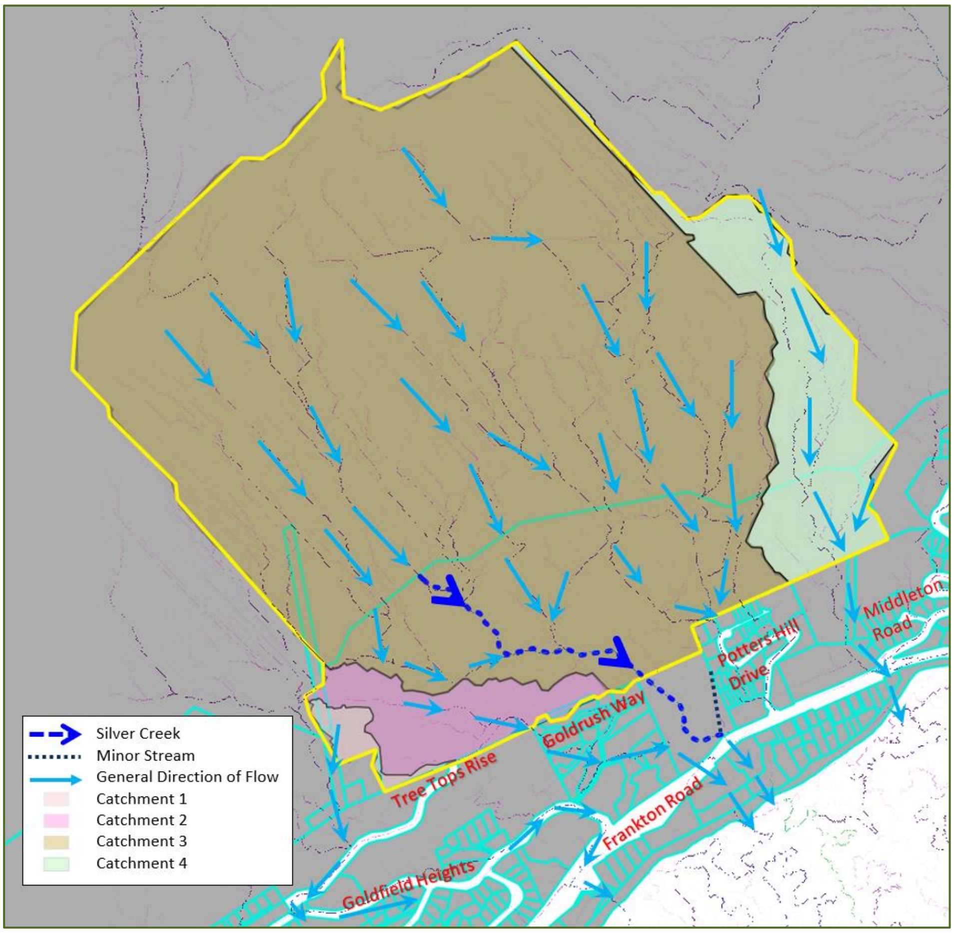
Project number	J000582
Version number	First Issue
Date	22/09/2022
Project Manager	Sejal Sangwai
Author	Sejal Sangwai
Additional technical contributors	Chenonn Chin, James Taylor, Navin Weeraratne

VERSION	DATE	DESCRIPTION	AUTHOR	REVIEWED
1	22/09/2022	First Issue	SRS	JT, NW

EXECUTIVE SUMMARY

Awa Environmental Limited (Awa) have been engaged by Mooreliving Limited to undertake a hydrology and hydraulic assessment for the proposed Silver Creek Residential Development. The purpose of this work is to assess the impacts of the proposed development on the local drainage network and overland flow path regime; and provide recommendations to mitigate any adverse impacts identified. The Silver Creek development proposes up to 585 new residential dwellings to be delivered in two stages; with Stage 1 delivering 150 units, followed by Stage 2 that will deliver an additional 435 Units.

A sub catchment delineation of the existing site of development identified four separate sub-catchments; with the most significant sub catchment area (approximately 85% of the total relevant stormwater sub catchment area) draining into Silver Creek. The other three sub catchments drain into Goldfield Heights, Goldrush Way and a natural watercourse located through 678 Frankton Road. A sub catchment delineation plan is provided below. It is noted that this delineation has been largely completed using existing LiDAR data with supplemented topographic survey data. The impacts of minor features such as roadside table drains, existing culverts and other surface features have therefore not been fully considered given the focus being on the 1% AEP design event. To this extent, the delineation provides a conservative result that may be optimised during detail design when more site-specific survey data will be available.



Sub catchment Delineation of Proposed Site of Development (including upstream contributing areas)

A hydrological model based on the Soil Conservation Service (SCS) Curve Number (CN) Loss Model was developed using HEC-HMS. Design rainfall depths were obtained from NIWA's HIRDS V4 RCP8.5 Scenario for the period 2081-2100. In order to meet the general requirements of the Queenstown Lakes District Council's (QLDC's) Proposed District Plan (PDP) and Otago Regional Council's Regional Plan (Water for Otago); existing and proposed development scenarios were assessed for the 1% AEP (+RCP8.5) scenario with the aim of mitigating any risks of flooding downstream.

A hydraulic assessment of flows entering the existing 1,050mm culvert at 634 Frankton Road (and draining Silver Creek) was modelled using HY-8 Culvert Analysis Software. The results of this were applied as a regulation curve into a MIKE 1D network model to assess the performance and associated effects within the downstream network. This model has been used to estimate attenuation requirements within the Silver Creek sub catchment (i.e., sub-catchment 3). Stormwater management and attenuation requirements within sub catchments 1, 2 and 4 have been developed to provide hydraulic neutrality noting the existing developments and drainage networks located immediately downstream.

Based on the above analysis, approximately 4,450m^3 of attenuation storage has been estimated as required to mitigate the effects of the proposed development on downstream receiving environments and networks (refer Plan provided in Appendix A). This is broken down into the relevant sub catchment areas as follows:

- Sub-catchments 1 and 2 (draining into Goldfield Heights and Goldrush Way) to require approximately 1,420m^3 of onsite attenuation by way of a proposed attenuation pond or wetland.
- Sub-catchment 3 (draining into Silver Creek) to require approximately 1,700 m^3 of additional storage (on top of the existing attenuation of 1,150 m^3 estimated to be available upstream of the stormwater culvert at 634 Frankton Road; assuming a 200mm freeboard from existing road levels). It is assumed that this will be provided via a series of attenuation ponds or wetlands, with existing sediment retention ponds on site reused as permanent devices wherever practically possible.
- Sub-catchment 4 (draining into the watercourse at 678 Frankton Road) to require approximately 1,320m^3 of onsite attenuation. Further investigation of site terrain will be required to confirm if attenuation via ponds or wetlands will be feasible in this area; alternatively, storage tanks within individual lots may be required (and has been estimated as approximately 3.3 m^3 per 100m^2 of developed land).

In delivering the stormwater design for the proposed development, overland flow paths (OLFPs) shall also be designed to be directed and managed within proposed roading corridors and existing water courses without imposing any risks of flooding to residential property during storm events of up to a 1% AEP (RCP 8.5). It is also proposed that all road runoff as a minimum be treated via biofiltration devices or wetlands to minimise the discharge of sediment and associated pollutants to the downstream receiving environment (i.e., Lake Wakatipu).

Provided the above stormwater management strategy is implemented through the detail design process; the proposed development will not generate any additional stormwater related impacts or risks to existing downstream property or receiving environments.

TABLE OF CONTENTS

1. INTRODUCTION

1.1. BACKGROUND

Awa Environmental Limited (Awa) have been engaged by Moore Living Limited to undertake a hydrology and hydraulic assessment for the proposed Silver Creek Residential Development.

1.2. PURPOSE & SCOPE

The purpose of this work is to assess the impacts of the proposed development on the local drainage network and overland flow path regime; and provide recommendations to mitigate any negative impacts identified.

A hydrological study will be undertaken to develop design storm hydrographs to assess the hydraulic capacity of existing pipe networks and overland flow paths including Silver Creek. Based on the findings of this analysis, recommendations will be provided on how any adverse effects from the development maybe mitigated to comply with the Queenstown Lakes District Council's (QLDC's) Proposed District Plan (PDP) and Otago Regional Council's Regional Plan (Water for Otago).

1.3. EXISTING SITE DESCRIPTION

The 33.71ha site does not presently have a street address but is legally identified as Lot 2 DP 409336. The site is located along the northern fringes of the Goldfields Heights and Potters Hill Drive subdivisions in Queenstown. It can be accessed via a number of roads adjoining the southern boundary including Goldfield Heights, which can be accessed via Frankton Road. The site is zoned Low Density Suburban Residential under the QLDC's PDP.

The site has recently been cleared of vegetation with former forestry tracks and roads reinstated to enable access across the site. A Geotechnical Feasibility Assessment completed by GCL in January 2021 notes the Geological Map of New Zealand (Sheet 18, Wakatipu, 1:250,000) to show the site to be underlain by Caples Terrane Schist, with several topographical features including a Central Lobe, Central Gully (i.e., Silver Creek), steep slopes to the east and moderate slopes to the west and south (refer Figure 2). The report also identified three development zones in terms of geotechnical feasibility:

- Zone A considered to comprise of generally stable conditions for residential development;
- Zone B requiring some further investigations; and
- Zone C requiring significant further geotechnical investigations.

A previous Stormwater Assessment completed by Aurum Survey Consultants Limited noted the site topography to form four sub-sub catchments within the proposed development, with Silver Creek draining a significant portion of the site, with a tributary of the Silver Creek also located to the east of the site and connecting into Silver Creek just above Frankton Road. A separate minor sub-catchment to the southwest of the development site has also been identified to drain into Top Lane and Goldfield Heights; with the most easterly sub-sub catchment draining into a natural watercourse located through 678 Frankton Road. Refer Figure 3 for the sub catchment delineation provided by Aurum Survey Consultants Limited. Refer Figure 4 for contour data for the site and associated upstream sub-catchment.

Figure 1. Site Location Plan of Proposed Silver Creek Development (Source: QLDC Geomaps)

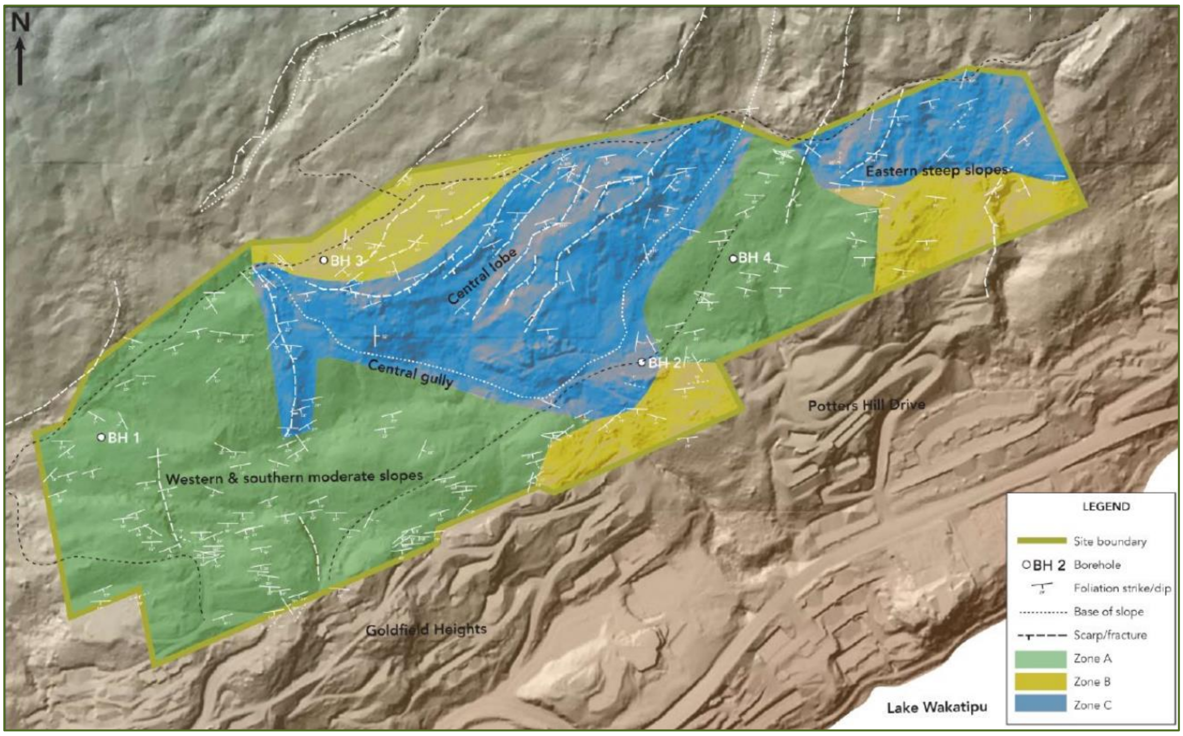


Figure 2. Topographical Features & Geotechnical Feasibility (Source: GCL Ltd)

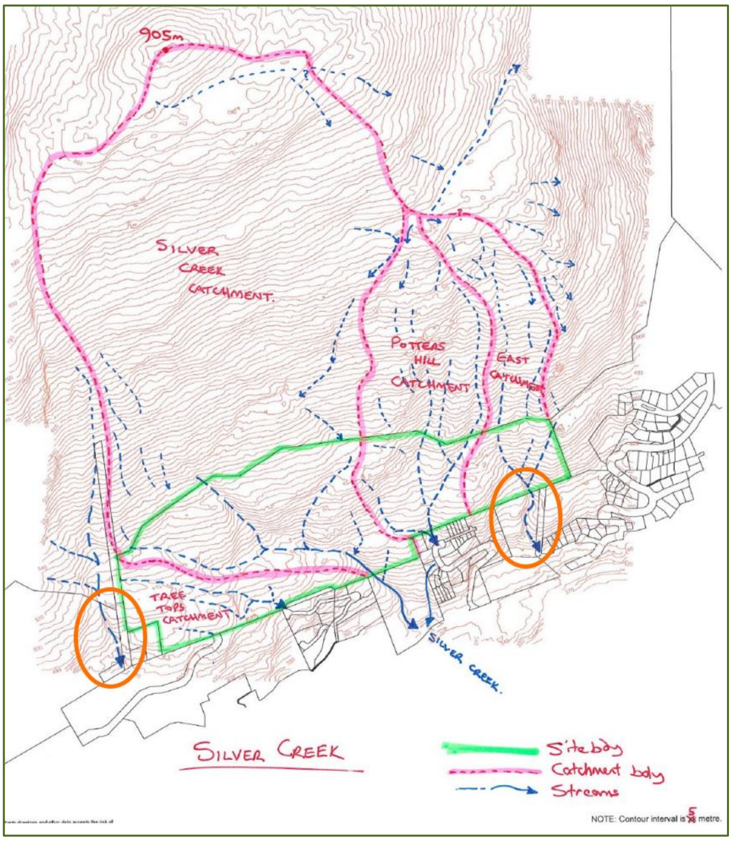


Figure 3. Previous preliminary Sub catchment Delineation (Source: Aurum Survey Consultants Limited)

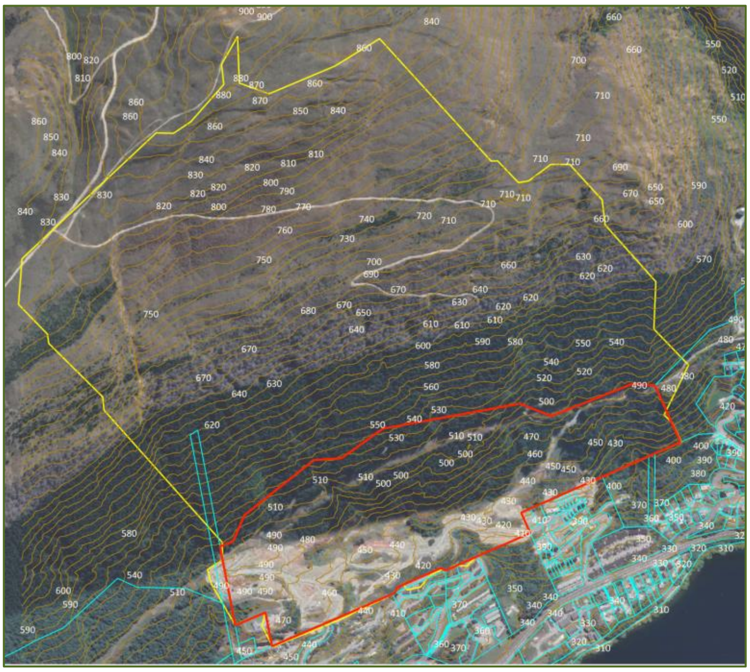


Figure 4. Contour Plan and property parcels

1.4. PROPOSED DEVELOPMENT

The Silver Creek development proposes up to 585 new residential dwellings to be delivered in two stages; with Stage 1 delivering 150 units, followed by Stage 2 that will deliver an additional 435 Units.

2. HYDROLOGICAL ASSESSMENT

This section of the report describes the hydrological assessment undertaken for the site by Awa.

2.1. DESIGN CRITERIA

This stormwater assessment is aimed at quantifying the impacts of the proposed development on downstream property and receiving environments. Based on the findings of this assessment, recommendations will be provided on how any adverse effects from the development maybe mitigated to comply with the Otago Regional Council's Regional Plan (Water for Otago) and QLDC's PDP. Within this context, key requirements that underpin the methodology developed for this assessment have consideredSection 4.3.5 of the QLDC Land Development Code of Practice.

2.1.1. HYDROLOGICAL ANALYSIS

The QLDC Land Development Code of Practice defines 3 primary objectives for stormwater quantity management. These are:

I. Preventing onsite flooding and frequent overland flows discharging from sites across adjacent properties;
II. Preventing the surcharge of downstream primary drainage network and flooding of downstream properties; and
III. Preventing downstream flooding and downstream overland flow path and receiving environment erosion.

In terms of hydrological analysis, the Code of Practice states that for larger sub catchments (i.e., larger than 50 ha) or where significant storage elements (such as ponds) are incorporated, surface water run-off should be determined using an appropriate hydrological or hydraulic model. Within this context, Awa has developed a hydrological model using a design rainfall profile, and calculation of runoff based on the Soil Conservation Service (SCS) Curve Number (CN) Loss Model to enable the assessment of sub catchment runoff to deliver on the objectives of the QLDC Land Development Code of Practice and Otago Regional Council's Regional Plan (Water for Otago).

2.2. SUB CATCHMENT DELINEATION

Digital Elevation Models (DEM) of the pre- and post-development terrain surfaces have been built using data obtained from the sources listed in Table 1.

2.2.1. PRE-DEVELOPMENT

The pre-development terrain surface (shown in *Figure 5*) has been created and used as part of the pre-development sub catchment analysis using LINZ's Otago-Queenstown raw LiDAR data 1m x 1m. The total sub catchment area upstream and including the site have been delineated in QGIS and can be split into four sub catchment areas. The land below the site is largely developed residential land with drainage consisting of a combination of stormwater networks and open streams. These systems cross Frankton Road, downstream residential property further to this, crossing the Frankton Track and ultimately draining in Lake Wakatipu. A sub catchment plan demonstrating the results of the analysis indicating flow paths and sub catchment boundaries is shown in *Figure 6*.

Table 1 – Data Sources

DATA	SOURCE	DESCRIPTION
1m LiDAR DEM	LINZ Data Service	Otago-Queenstown 2021 LiDAR data from LINZ Data Service
Silver Creek Development 3D model	Moore living ltd	A design surface model of the development site including new roads and relevant earthworks. Also contains topographical survey data of the existing site.

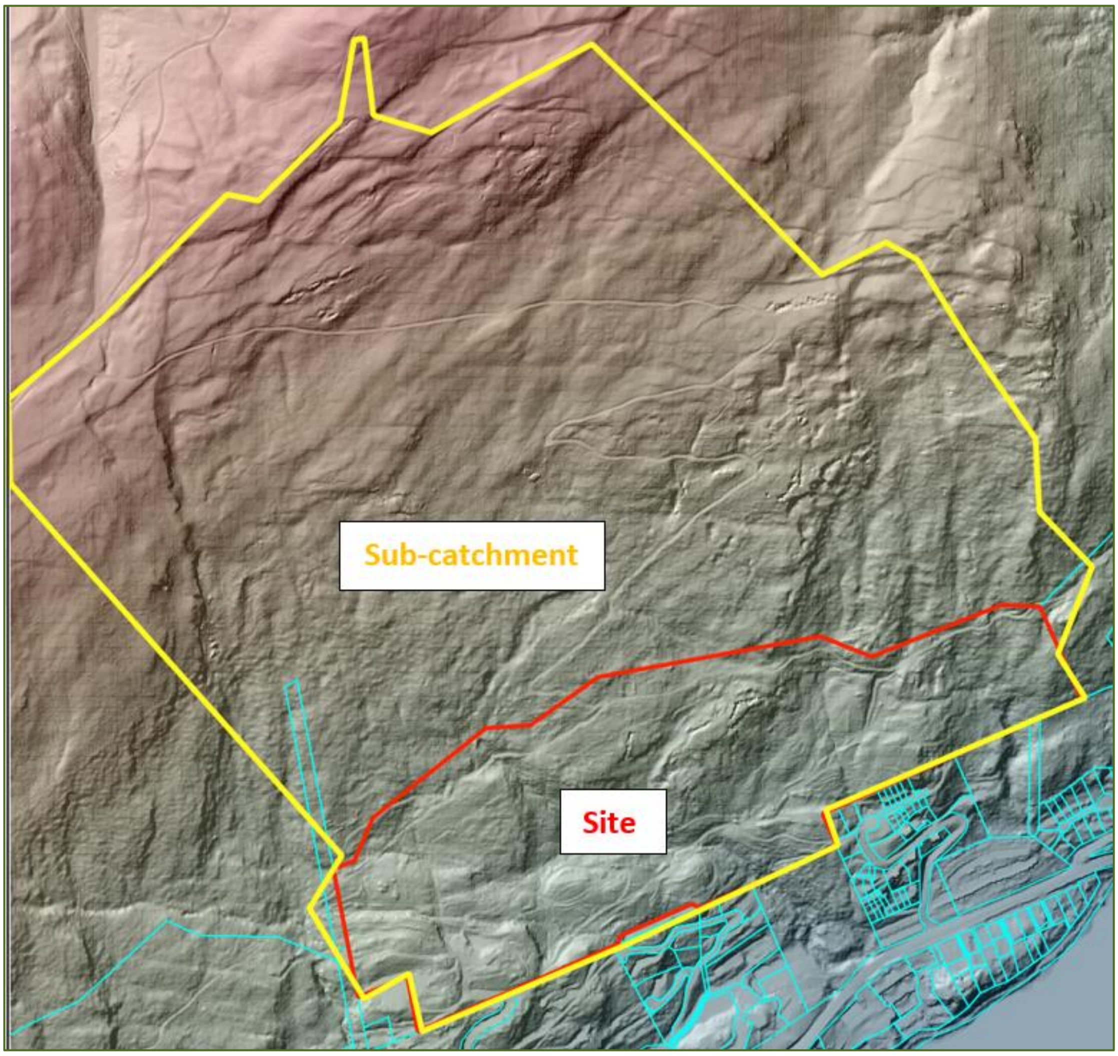


Figure 5. Pre-development terrain surface

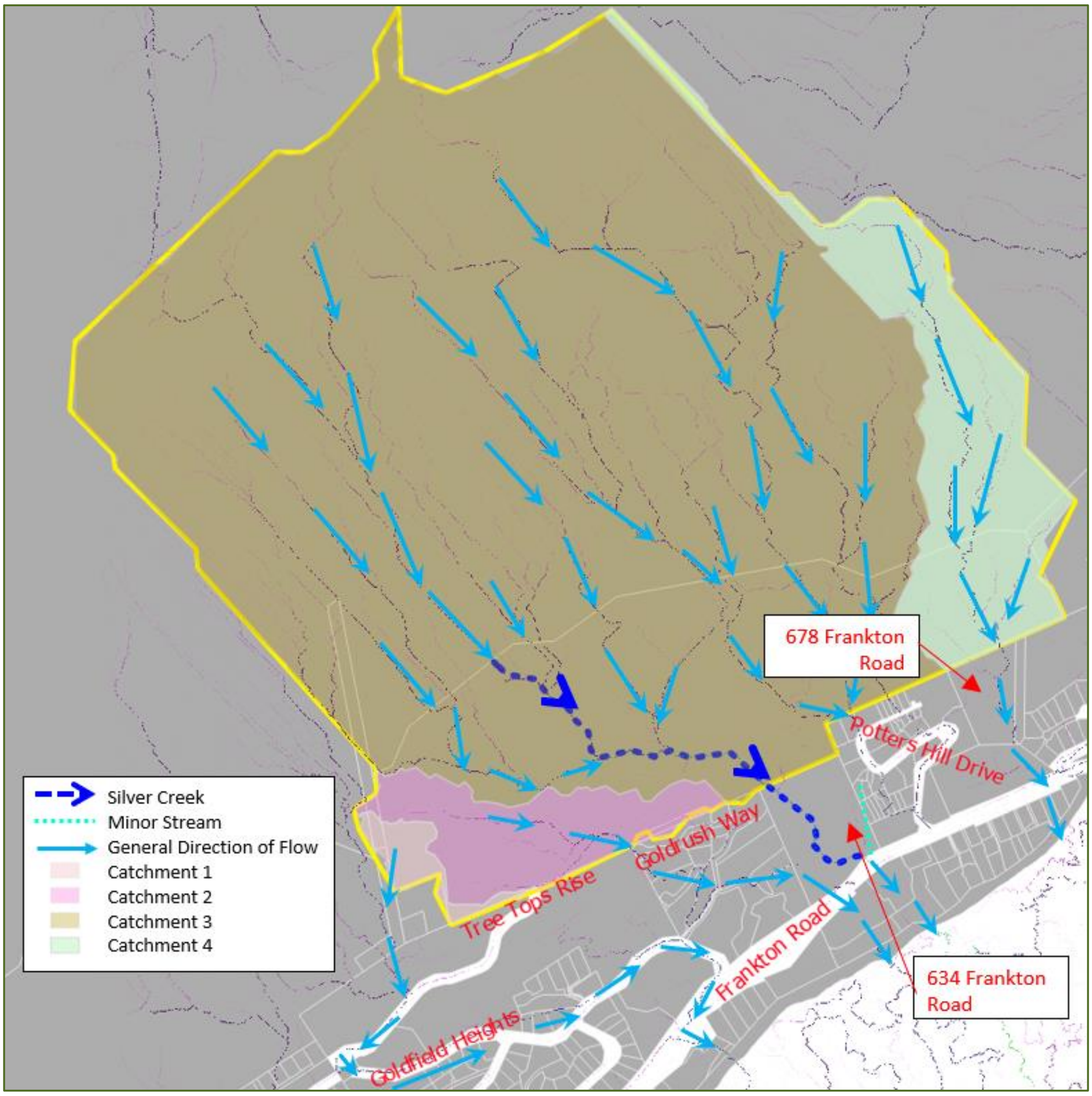


Figure 6. Pre-development Sub catchment Plan and Overland Flows

SUB CATCHMENT OVERVIEW

Sub catchment 1: This small portion of approximately 1.27 Ha drains towards the west to the Goldfield Heights development.

Sub catchment 2: This portion of the sub catchment is approximately 6.4Ha and drains towards Goldrush Way.

Sub catchment 3: This is the largest sub catchment area covering approximately 124.7 Ha. This includes most of the upper mountain slopes, extending to the high point of Queenstown Hill. As noted by Aurum Survey Consultants Limited, *the upper portion of the sub catchment is open grass & tussock land, with lower slopes being a mixture of conifer and other weeds. Much of the sub catchment is also part of an ancient landslide with a variable surface topography, including clefts and depressions. The sub catchment exit point is Silver Creek, a natural and well incised watercourse that passes through 634 Frankton Road.*

A smaller eastern portion within the Sub catchment 3 drains into developed land at the top of Potters Hill Drive before discharging into a minor stream on 634 Frankton Road which then meets Silver Creek above the Frankton Road culvert.

It should be noted that a gravel track running parallel to the northernmost upstream boundary of the site, is present on the site which is understood to have been formed during construction of the overhead power lines bisecting the site (refer the black line in Figure 7).

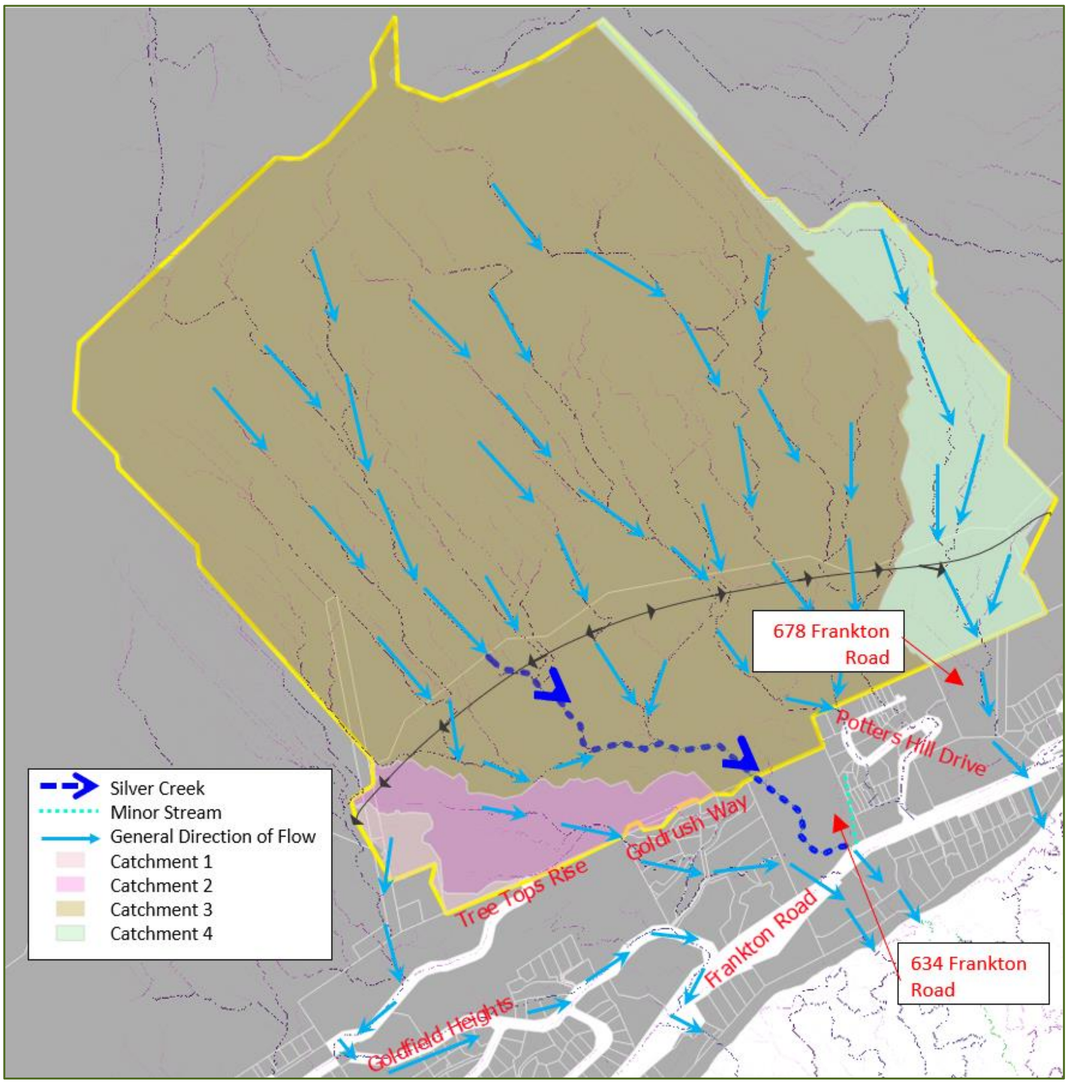


Figure 7 - Catchment plan showing potential road table drain in black

This track is evident in aerial photos, site visit walkovers, and in some earlier LiDAR data. It is understood that a roadside table drain runs along this road, which may redirect some of the overland flow draining from the upper portion of sub-catchment 3 towards both sub-catchment 2 and 4. The size and capacity of this table drain is unable to verified at this stage due to the course nature of existing LiDAR and topographical survey data. Minor culverts are also understood to direct some of the flow across the road into Silver Creek. Further detailed investigations will be carried out at detailed design stage to assess the impact of this road and table drain on the upstream catchment. Due to the focus being on the 1% AEP design event, it is considered appropriate to ignore the impact of fine surface features of this road and table drain for the purposes of this assessment, which leads to a more conservative assessment being carried out.

Sub catchment 4: This part of the development covers approximately 14.5 hectares and contains steep slopes and a series of gullies and bluffs forming irregular topography, extending down to the southern boundary. This sub catchment is understood to drain to a natural watercourse flowing through 678 Frankton Road.

2.2.2. POST-DEVELOPMENT

The terrain surface shown in *Figure* 8 has been used for the post-development sub catchment analysis. It has been developed using LINZ's Otago-Queenstown raw LiDAR data for the upstream and downstream sub catchment, with the design development surface containing proposed roads (refer *Figure 9*) added on. A raster has been created with a 1m x 1m grid size. A sub catchment plan demonstrating the results of the analysis indicating flow paths and sub catchment boundaries is shown in *Figure* 10.

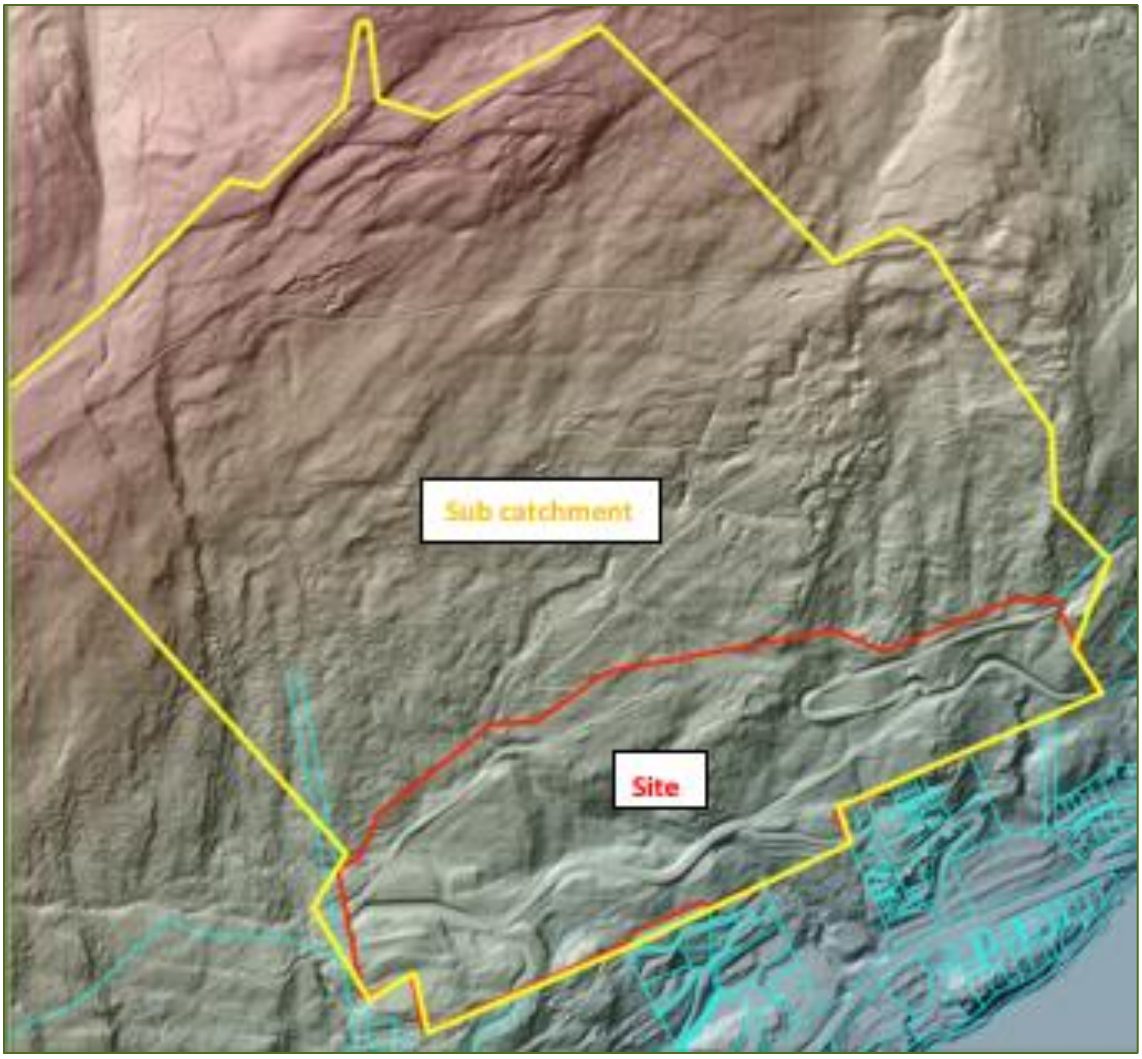


Figure 8. Post-development terrain surface

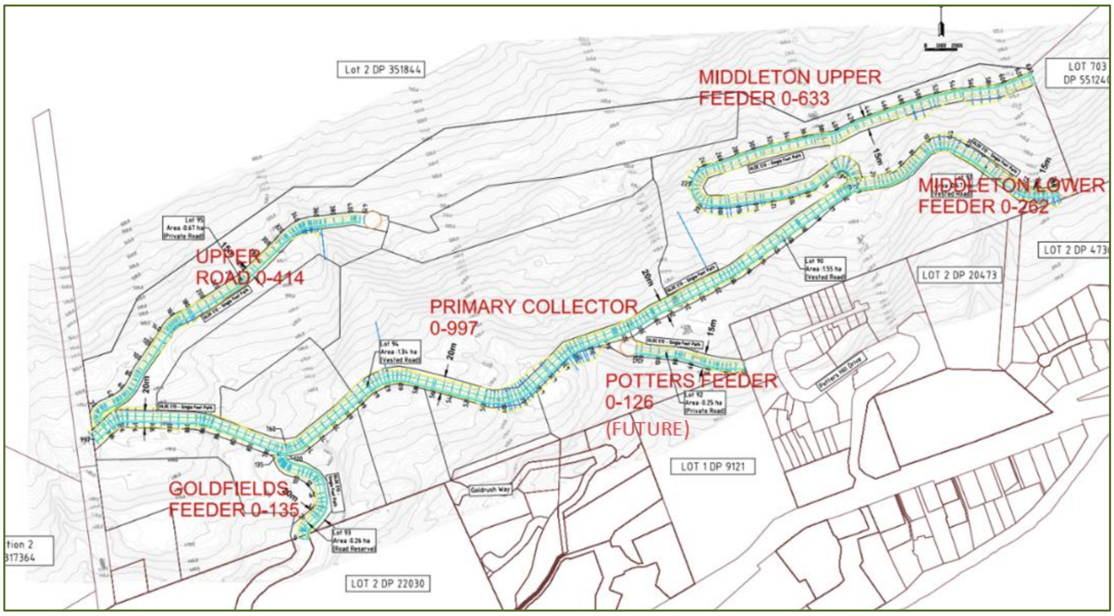


Figure 9. Post-development Roading Plan (Source: Silver Creek Residential Development plans)

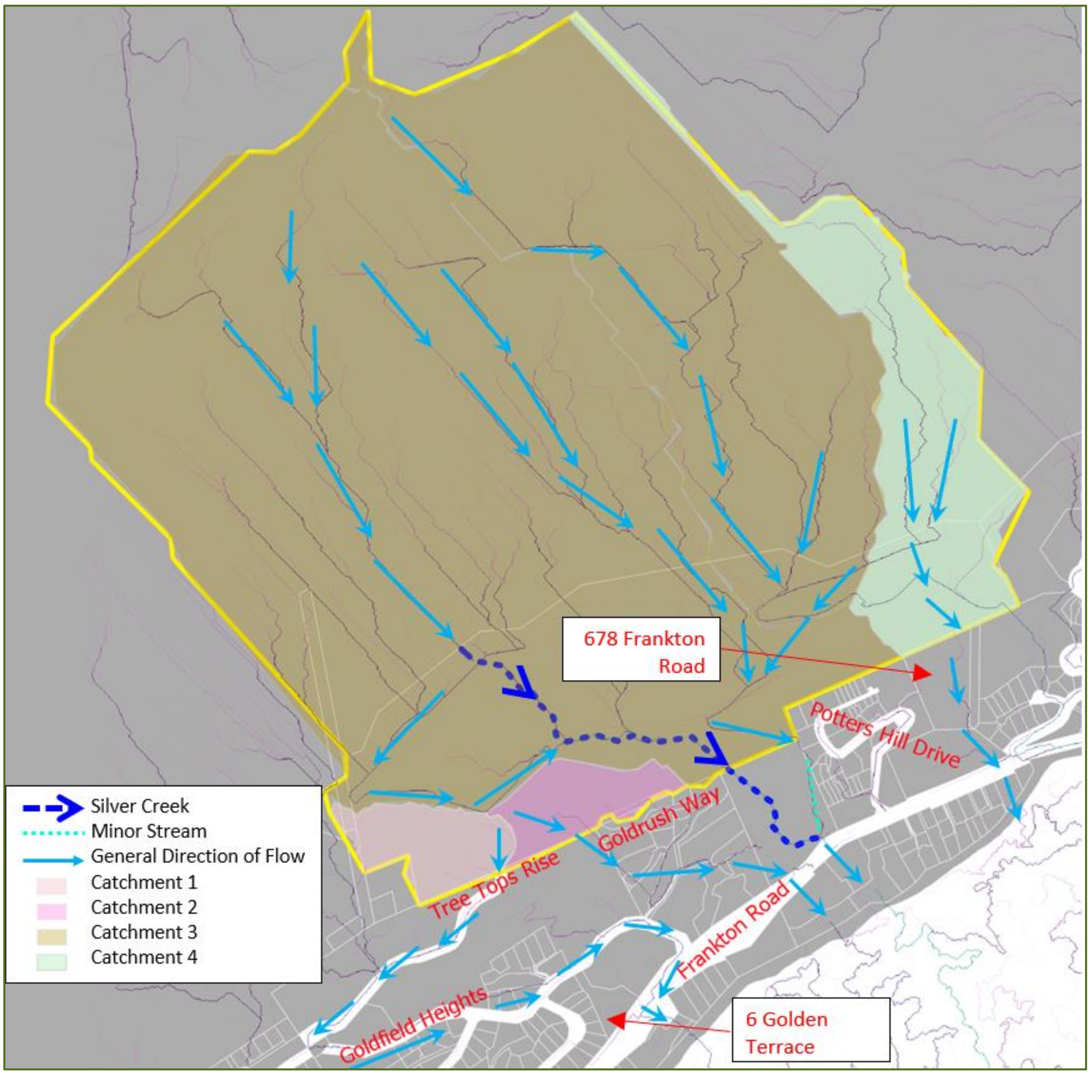


Figure 10. Post-development Sub catchment Plan and Overland Flows

Table 2 – Delineated Sub catchment Areas (Pre and Post-Development)

		SUB CATCHMENT 1	SUB CATCHMENT 2	SUB CATCHMENT 3	SUB CATCHMENT 4
Area (m^2)	Existing	12,759	64,169	1,247,876	145,079
	Proposed	31,809	29,642	1,265,732	145,079

Compared to pre-development, the post development **Sub catchment 1** now drains more than twice the pre-development area, (mainly existing subcatchment 2 area). Roughly 3.2 Ha is collected towards the western side of Goldfield Feeder Road and is expected to drain via Tree Tops Rise before traversing through Goldfield Heights.

Sub catchment 2 is now approximately 3 Ha and will drain towards the southern side of the Primary Collector Road in between the Goldfield Feeder Road and Potters Feeder Road; draining towards Goldrush Way similarly to pre-development.

A very small portion of pre-development Sub catchment 2 (above the Primary Collector Road towards the Western side) is diverted towards Sub catchment 3. The post-development **Sub catchment 3** area is approximately 126.5 Ha. A section of Sub catchment 3 (along the Middleton Upper Feeder) will need to be culverted to achieve this.

This leaves the 14.5 ha **Sub catchment 4** the same as pre-development, primarily draining to a natural watercourse through 678 Frankton Road with some flows exiting the site through the south-eastern corner on Middleton Feeder Road and towards 678 Frankton Road.

2.3. TIME OF CONCENTRATION

Time of concentration (ToC) has been estimated using several methods as shown in the *Tables 3 and 4*. Sub catchment slope was estimated for the longest flow path using the equal-area method for pre- and post-development scenarios.

ToCs calculated using Ramser-Kirpich method and USSCS method were considered to better represent the steep upstream sub catchments, compared to the Branby-Williams and TR20 Lag Methods.

Table 3 – Time of concentration (minutes) using various methods for pre-development

PRE-DEVELOPMENT SUB CATCHMENT	1	2	3	4
Ramser-Kirpich	1.45	5.52	11.29	9.03
Bransby-Williams	4.20	16.25	37.10	35.94
USSCS	1.60	5.45	11.82	9.28
TR20-Lag Method	3.61	15.30	30.46	23.71

Table 4 – Time of concentration (minutes) using various methods for post-development

POST-DEVELOPMENT SUB CATCHMENT	1	2	3	4
Ramser-Kirpich	3.65	2.52	14.63	9.03
Bransby-Williams	9.04	6.51	43.68	35.94
USSCS	3.37	2.45	13.45	9.28
TR20-Lag Method	5.39	3.52	37.73	23.71

Based on the above results, a minimum ToC of 10 minutes was selected for the pre-development scenario; and for Sub catchments 1,2, and 4 in the post-development scenario. A ToC of 15 minutes was selected for Sub catchment 3 under the post-development scenario. The key reason for a higher ToC in Sub catchment 3 under the post-development scenario is an increase in estimated catchment length due to the construction of roads.

2.4. DESIGN RAINFALL DEPTHS

Design rainfall depths for the site have been obtained from HIRDS V4 RCP8.5 for the period 2081-2100 (estimated to represent an approximate 3.7°C increase in temperature for climate change). 100-year ARI storm depths for a range of storm durations obtained from HIRDS is shown in *Table 5*. A critical storm duration of 2 hours was established for the catchments using a series of HEC-HMS simulations based on this data, identifying the storm duration leading to the greatest downstream peak flow.

Table 5 – Rainfall Depth-Duration Data

DURATION (HR)	DURATION DESIGN STORM DEPTH (MM)
1	39.4
2	54.9
6	86
12	109
24	133

2.5. SYNTHETIC HYETOGRPAH

A triangular synthetic hyetograph has been selected as no gauged data is available for the site. This synthetic hyetograph is commonly used throughout New Zealand for ungauged sub catchments. An analysis undertaken by Awa for the nearby gauged Mill Creek Sub catchment provided a similar shaped storm hyetograph, providing confidence in the synthetic triangular hyetograph.

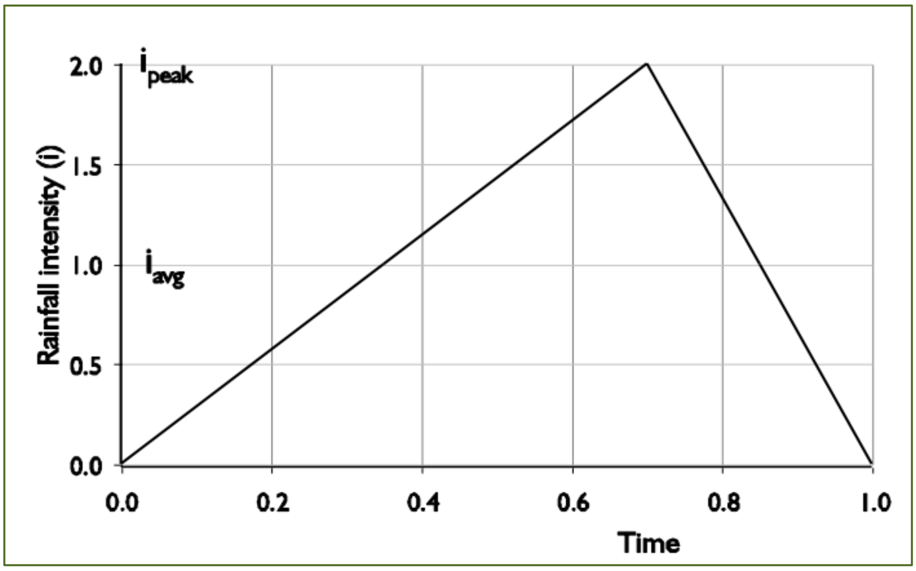


Figure 11. Standard dimensionless hyetograph for rainfall intensity

2.6. CURVE NUMBERS

Curve numbers have been allocated on a sub-catchment basis using Table 2 of NRCS Technical Release 55 (TR-55). The geotechnical report by GCL classifies underlying soil as colluvium and/or glacial with overlying schist bedrock within the site area which has been assumed to be best reflected by Group 'C' soil classification as per TR-55.

For pre-development (existing scenario) both the large upstream and the site sub-catchments have been modelled as deforested area[1]. Deforested areas have been assessed using SCS Curve Number (CN) of 77 for cover type 'Brush', hydrologic condition 'Poor' and soil type 'C'.

For post-development, the sub-catchments have been modelled separately to include the deforested upstream area and proposed pervious and impervious areas within the site. A 30:70 ratio for pervious to impervious area has been assumed, as per QLDC Chapter 7 – Lower Density Suburban Residential Zone of PDP Decisions (version April 2022). Pervious areas have been assessed using a CN of 86 for grass cover less than 50%, and soil type 'C'. Impervious areas have been assessed using a CN of 98.

Table 6 – Curve number based on Land Cover

LAND COVER		CURVE NUMBER
Deforested		77
Developed	Pervious	86
	Impervious	98

Based on the above, a series of HEC-HMS models were developed and simulated to establish peak existing and proposed flow rates, and runoff volumes, as presented in Table 7.

[1] As per QLDC section 92 request, the upstream plantation forest has been represented by a curve number that reflects a deforested condition, leading to higher runoff rates.

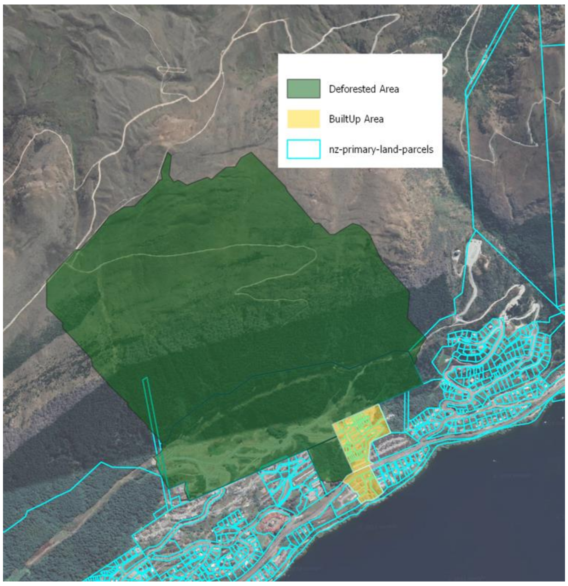


Figure 11.1. Land Cover - Pre-Development

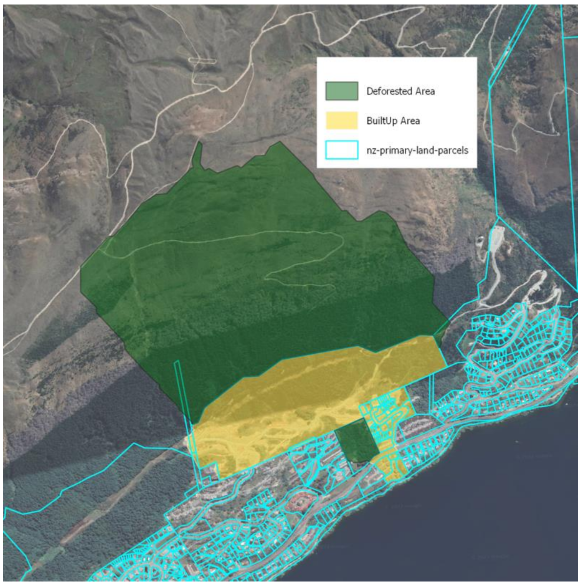


Figure 11.2. Land Cover - Post-Development

Table 7 – Peak runoff flow rates for existing and proposed (if left unmitigated) Sub catchments

		SUB-CATCHMENT 1	SUB-CATCHMENT 2	SUB-CATCHMENT 3	SUB CATCHMENT 4
Peak Flow (m^3/s)	1% AEP Existing	0.07	0.36	7.08	0.82
	1% AEP Proposed	0.40	0.37	8.39	1.10

3. ASSESSMENT OF STORMWATER EFFECTS

A conceptual design of a stormwater solution has been developed to test the impacts of the proposed development and determine mitigation required. The design was aimed at achieving QLDC's primary objectives for stormwater management as listed below:

I. Preventing onsite flooding and frequent overland flows discharging from sites across adjacent properties.
II. Preventing the surcharge of downstream primary drainage network and flooding of downstream properties.
III. Preventing downstream flooding and downstream overland flow path and receiving environment erosion.

Based on the above, the concept design aims to limit peak discharge rates to existing development levels, while also ensuring no increase in flood risk downstream of the site (and in particular, the Alpine Village located at 643 Frankton Road).

3.1. DRAINAGE STRATEGY

This section describes the general strategy for managing stormwater for each sub catchment.

3.1.1. SUB CATCHMENTS 1 & 2

Pre-development flows from Sub-catchment 1 exits the site at the south-western corner and drains though the Goldfield Heights development (Refer *Figure 6).* Post-development if left unmitigated, would flow from Sub-catchment 1 exit to Tree Tops Rise through the Goldfield Heights development (Refer *Figure 10).* Both pre and post development flows eventually drain via a channel drain at 6 Golden Terrace, via a 900mm diameter culvert (Culvert 4, Figure 12) prior to discharging into Lake Wakatipu via 535 Frankton Road.

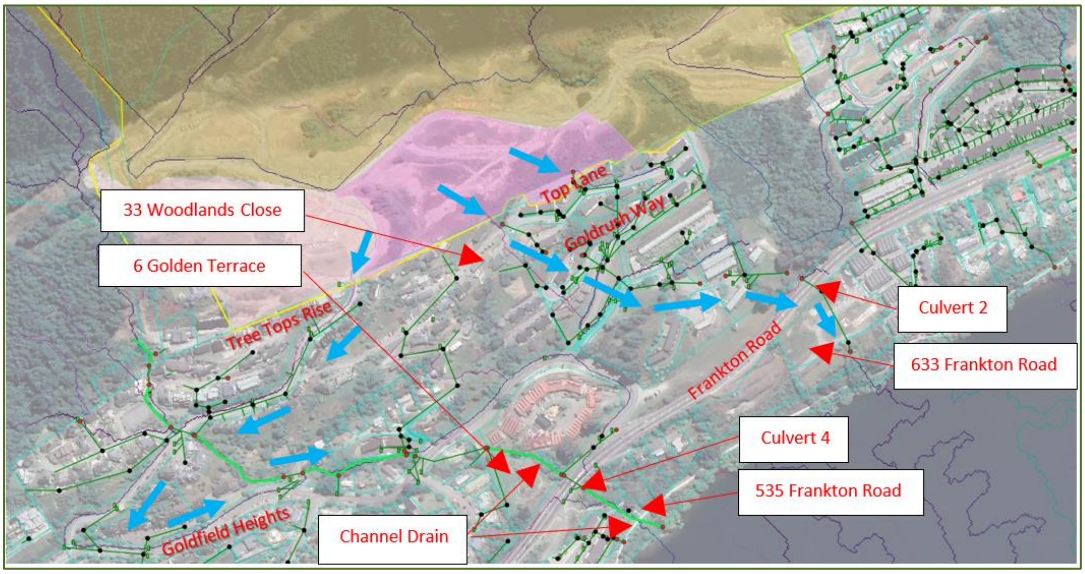


Figure 12. Post Development flows and existing downstream drainage network - Sub catchments 1 and 2 – unmitigated scenario

For both Pre and Post development, Sub catchment 2 drains towards Goldrush Way (through 33 Woodlands Close) and Tops Lane (Refer *Figures 6 and 10*) converging further downstream along Goldrush Way towards 634 Frankton Road. These flows cross Frankton Road via a 600mm dia. culvert (Culvert 2, Figure 12) prior to discharging into Lake Wakatipu via 633 Frankton Road.

RECOMMENDED MITIGATION FOR SUB CATCHMENTS 1 AND 2

It is proposed that the combined discharge from the two sub catchments be attenuated to the Sub catchment 2 pre-development peak flow (i.e., a maximum of 0.36 m^3/s); with Sub-catchment 1 diverted to be discharged along with Sub-catchment 2 via Top Lane *(Refer Appendix A)*. Approximately 1,420m^3 of attenuation storage is assessed as necessary to achieve this (*Refer Table 8*).

A HEC-HMS model has been used to assess the suitability of a conceptual storage device to meet attenuation requirements. A hydraulic outlet control (orifice) has been used to attenuate runoff such that Sub-catchment 2 pre-development peak flow rates are maintained.

Details of these structures will be confirmed at detailed design; however approximate sizing for the 1% AEP event (RCP8.5) has shown the storage device to be able to attenuate runoff from these catchments to 0.32 m^3/s via a 300mm orifice (refer Table 8 and Figure 13).

Table 8 – Mitigated Peak Flows for 100Yr Critical Event

SUB CATCHMENT	DEVELOPMENT SITE PEAK FLOW		OUTLET SIZE	MITIGATED POST DEVELOPMENT
	PRE-DEVELOPMENT	POST-DEVELOPMENT		
1	0.07 m^3/s	0.40 m^3/s		
2	0.36 m^3/s	0.37 m^3/s		
Combined	0.43 m^3/s	0.77 m^3/s	300mm	0.32 m^3/s

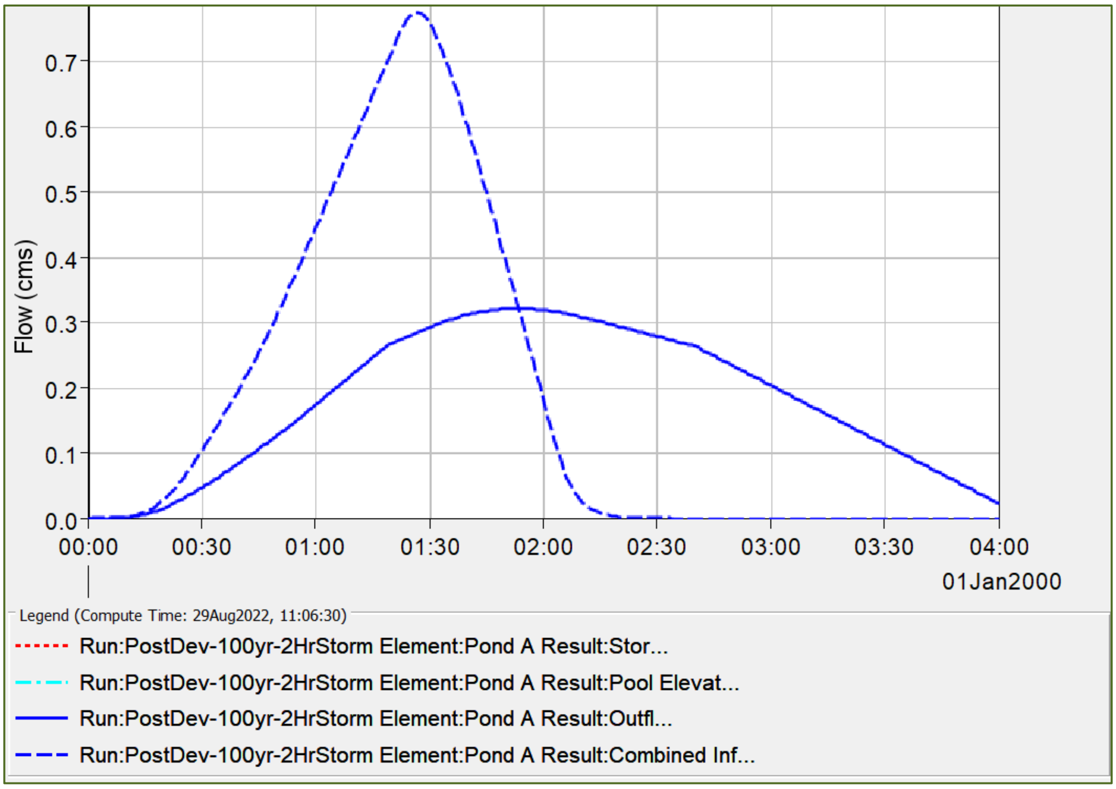


Figure 13. Extract from HEC-HMS model showing attenuation in pond A for 1% AEP (RCP8.5) AEP event

3.1.2. SUB CATCHMENT 3

Flows from Sub-catchment 3 exit the site towards 634 Frankton Road (Refer *Figures 6 and 10)* via Silver Creek, a natural and well incised watercourse that passes through 634 Frankton Road. A smaller eastern portion within the Sub catchment 3 drains into developed land at the top of Potters Hill Drive before discharging into a minor stream on 634 Frankton Road which then meets Silver Creek above Frankton Road prior to discharging via a 1,050mm diameter culvert crossing Frankton Road (Culvert 1, *Figure 14*).

Flows via Culvert 1 discharge to the stormwater network through the downstream Alpine Village development which includes a cascading open channel (forming a waterfall feature), followed by a 650mm diameter culvert below the Frankton Track (Culvert 3, *Figure 15*) discharging into Lake Wakatipu. Figure 15 shows the stormwater network responsible for discharging flows from Sub catchment 3 to Lake Wakatipu based on information gathered from site visits and survey.. Images of the 1,050mm culvert and cascading open channel are also provided overleaf.

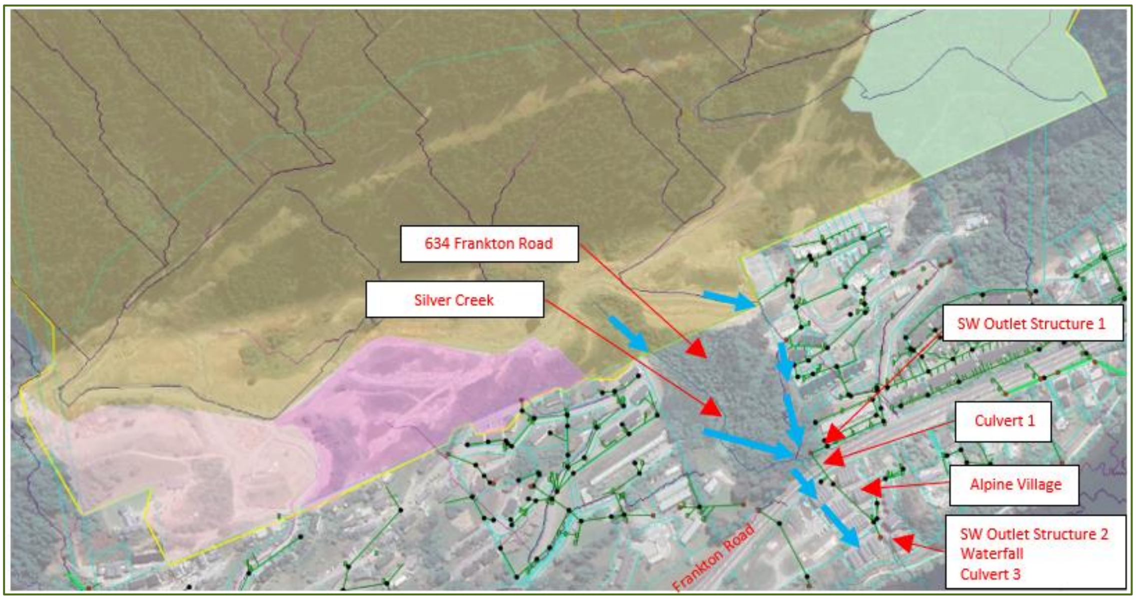


Figure 14. Post Development flows and existing drainage network - Sub catchment 3

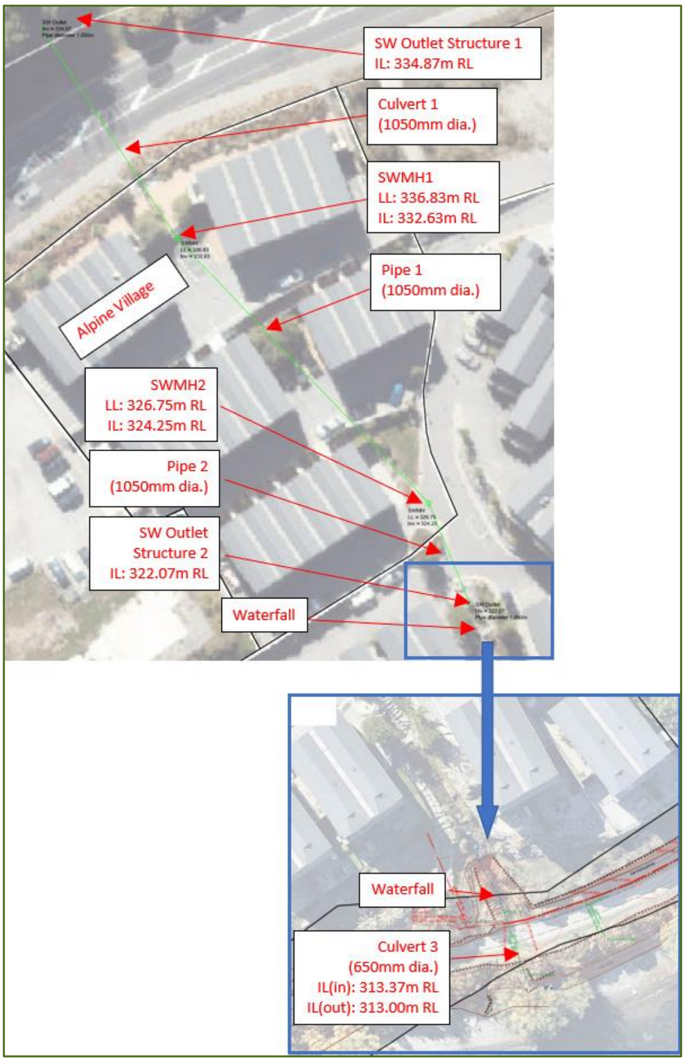


Figure 15. Existing drainage network - Sub catchment 3 (Source: Aurum Survey Consultants Limited)

Image of Culvert 1 Inlet – 634 Frankton Road (Source: Aurum Survey Consultants Limited)

Image of Cascade/Waterfall (Source: Site Visit)

To determine the impact of the proposed development on Sub catchment 3, a hydraulic assessment of flows entering the existing 1,050mm culvert at 634 Frankton Road (and draining Silver Creek) was modelled using HY-8 Culvert Analysis Software. The results of this was applied as a regulation curve into a MIKE 1D network model to assess the performance and associated effects within the downstream network. The manholes were added as nodes with weirs added at each manhole to simulate overflows.

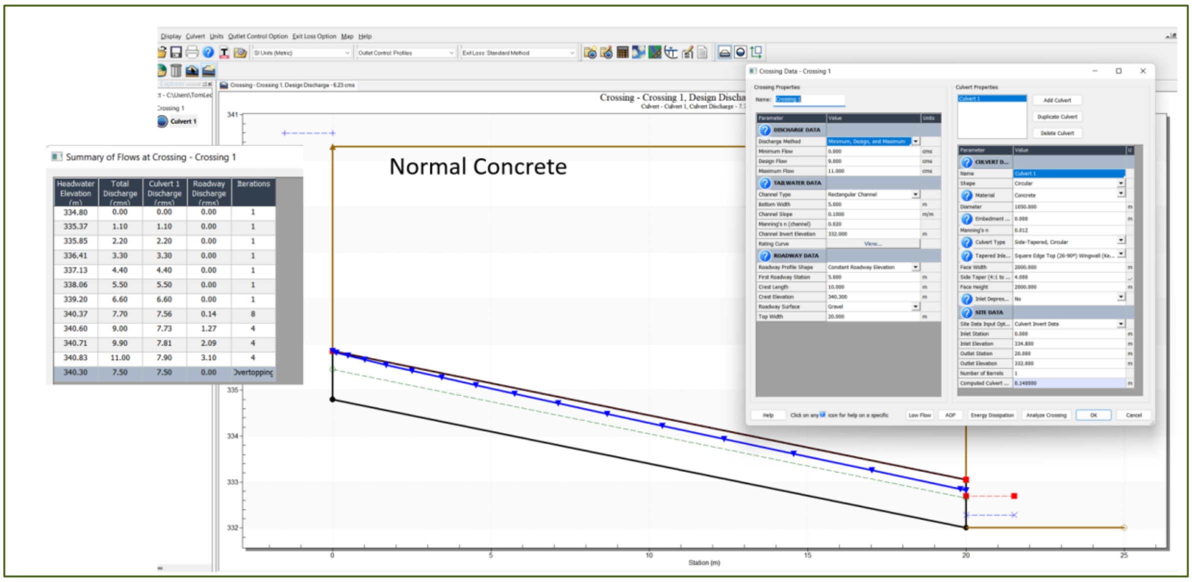


Figure 16. HY-8 Model Outputs for 1050mm Culvert (Culvert 1)

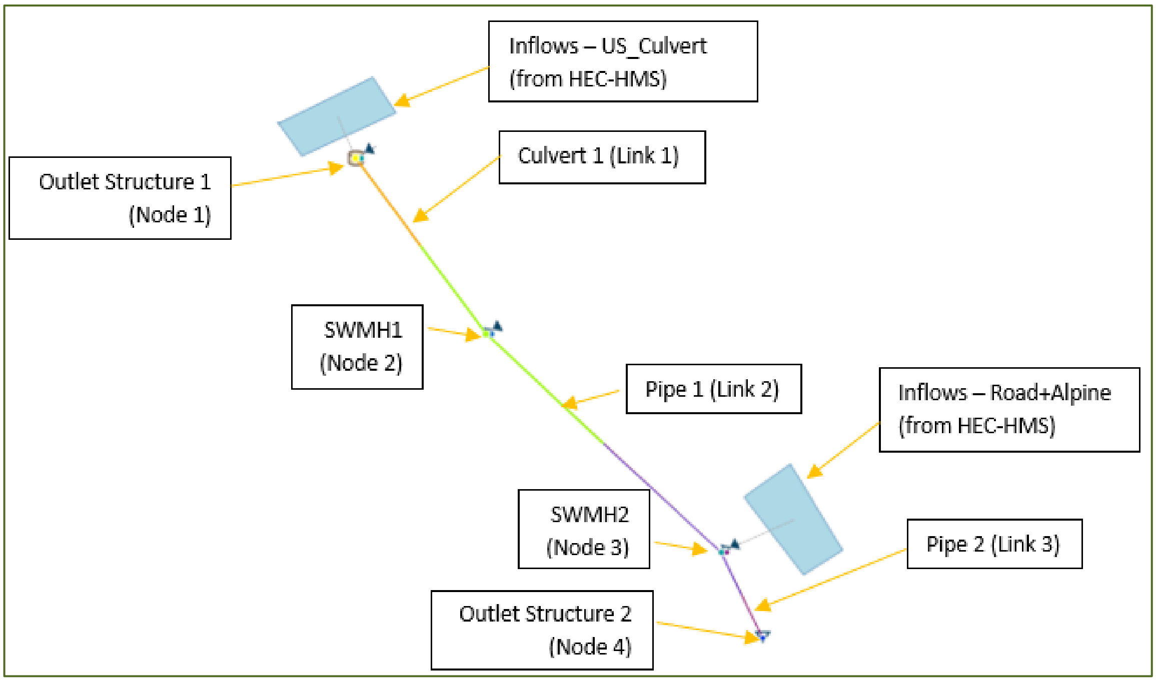


Figure 17. MIKE 1D Model Schematic of Stormwater Network

The network is built based on topographical survey data, with the storage available within the existing topographical depression upstream of Culvert 1 (Refer *Figure 18*) represented by an elevation-area curve developed from the terrain model. Based on the LiDAR contour data the storage depression spill level for design was set at 340.3m RL (allowing a 200mm freeboard from the road spill level).

Figure 18. Existing Storage behind Frankton Road

A normal boundary condition was applied at the outlet as no backwater effects were anticipated given the discharge of the network into the cascading open channel. The runoff hydrograph from the 2-hour duration design storm has been included as the inflow boundary condition.

The pre- and post-development runoff hydrographs for Sub-catchment 3, and other contributing downstream catchments, were loaded into the nodes at the assumed discharge locations (as per *Figure 17*). These inputs are presented in Table 10.

Table 10 – Pre- and Post-Development Peak Inflows modelled as per Figure 17.

CATCHMENT	PEAK DISCHARGE		RUNOFF VOLUME	
	PRE-DEVELOPMENT	POST-DEVELOPMENT	PRE-DEVELOPMENT	POST-DEVELOPMENT
Upstream Culvert	7.70 m^3/s	8.97 m^3/s	19,004 m^3	26,160 m^3
Alpine	7.90 m^3/s	9.15 m^3/s	19,670 m^3	26,820 m^3

MIKE NETWORK MODEL RESULTS

Baseline (pre-development)

Link 1 (*Figure 17*) was identified to be surcharged; however Links 2 and 3 were identified to provide sufficient capacity at their higher gradients. Despite the surcharging along Link 1, no flooding was identified during the pre-development scenario with the peak water level upstream of Frankton Road reaching 339.9m RL (noting a design spill level as described previously of 340.3m RL).

Post-development

Under the developed scenario, the peak water level the peak water level upstream of Frankton Road has been identified to reach 340.73m RL; breaching the design spill level and generating up to 1,700m^3 of overflows across the road. Attenuation of this excess flow upstream of Frankton Road (or within the proposed Silver Creek Development) will therefore be required. Within this context, the existing storage volume available upstream of Culvert 1 (and Frankton Road) has been estimated as 1,150m^3. The total required storage volume to prevent the breaching of Frankton Road has been estimated as 2,850m^3. It is proposed that this additional storage (of up to 1,700m^3) be provided via a series of attenuation devices within the catchment, with existing sediment retention ponds on site reconfigured for permanent use wherever practically possible.

Subject to achieving the above; no additional risk of flooding is anticipated downstream of Frankton Road (i.e., the Alpine Village located at 643 Frankton Road). Results of the MIKE Network model analysis are summarised in *Figures 19 and 20*.

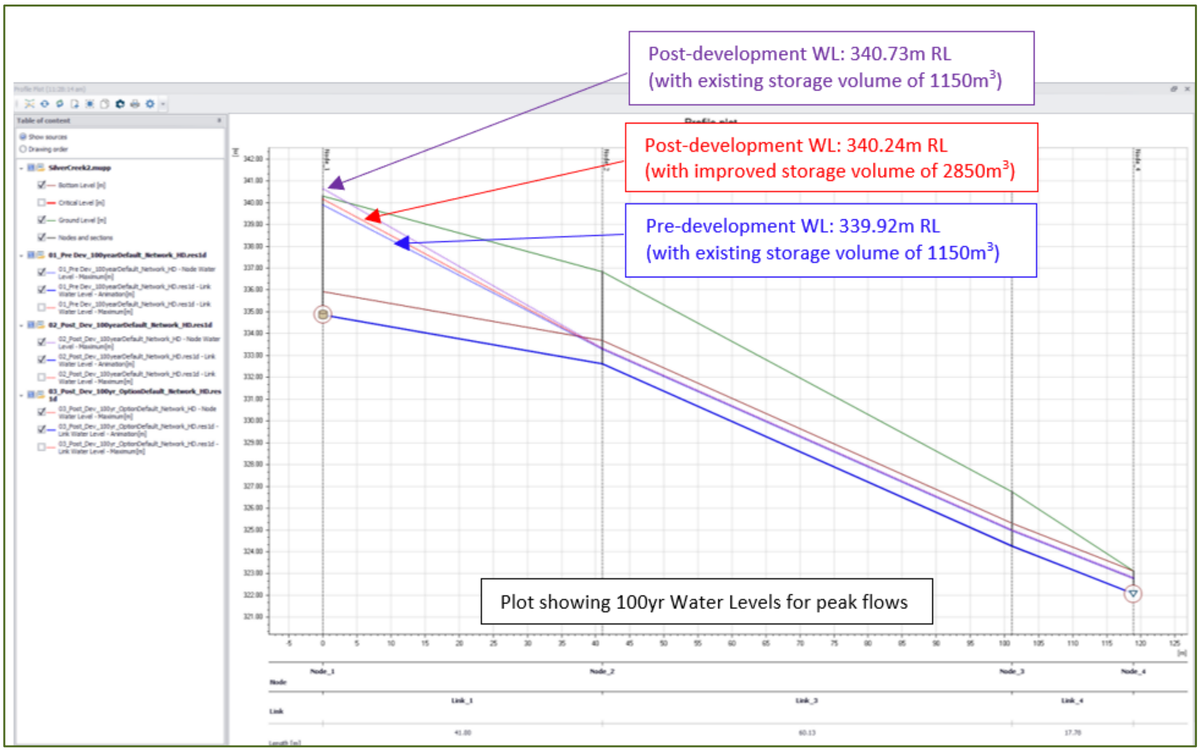


Figure 19. Hydraulic Profile of Stormwater Network Downstream of Culvert 1 (Also refer Appendix B)

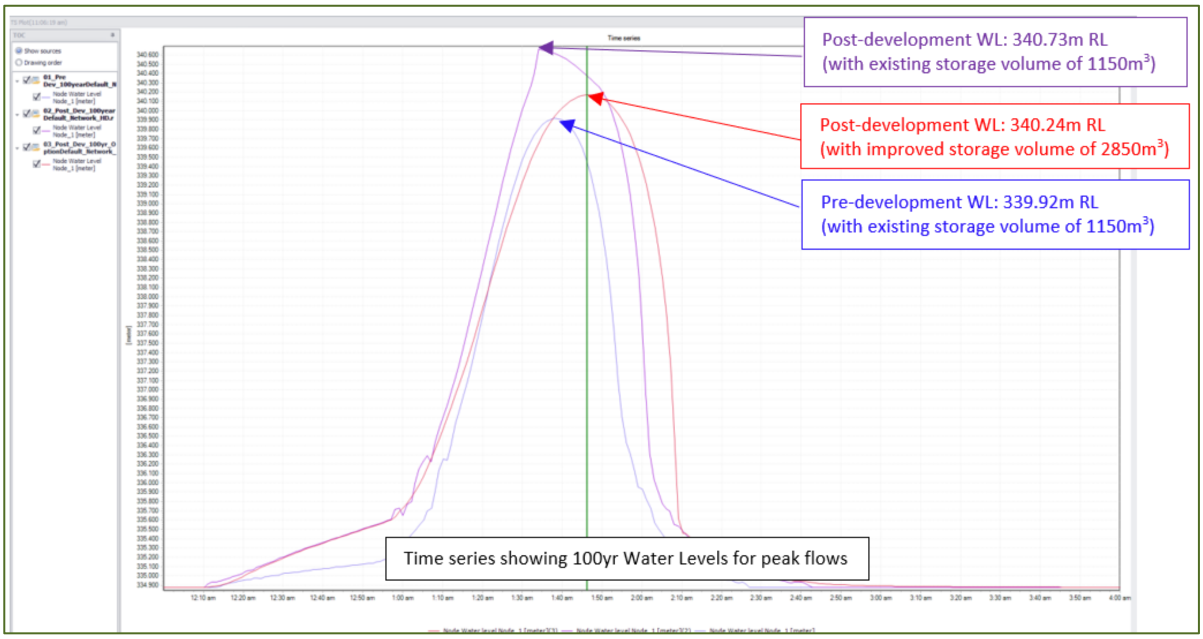


Figure 20. Time Series of Stormwater Network Downstream of Culvert 1 (Also refer Appendix B)

3.1.3. SUB CATCHMENT 4

Pre-development, Sub-catchment 4 primarily drains into a natural watercourse located at 678 Frankton Road, prior to discharging into Lake Wakatipu via a 750mm diameter culvert (Culvert 5, *Figure 21*). It is proposed that this be replicated post-development. In order to ensure discharges into the existing downstream network is not increased post-development, attenuation will be required.

Noting the terrain within this sub catchment area is steep; it may be necessary to provide this attenuation via storage devices. The overall attenuation requirement for this sub catchment has been estimated using a HEC-HMS model as approximately 1,320m^3 (or approximately 3.3 m^3 per 100m^2 of developable land). Results of this HEC-HMS model are provided in Table 11.

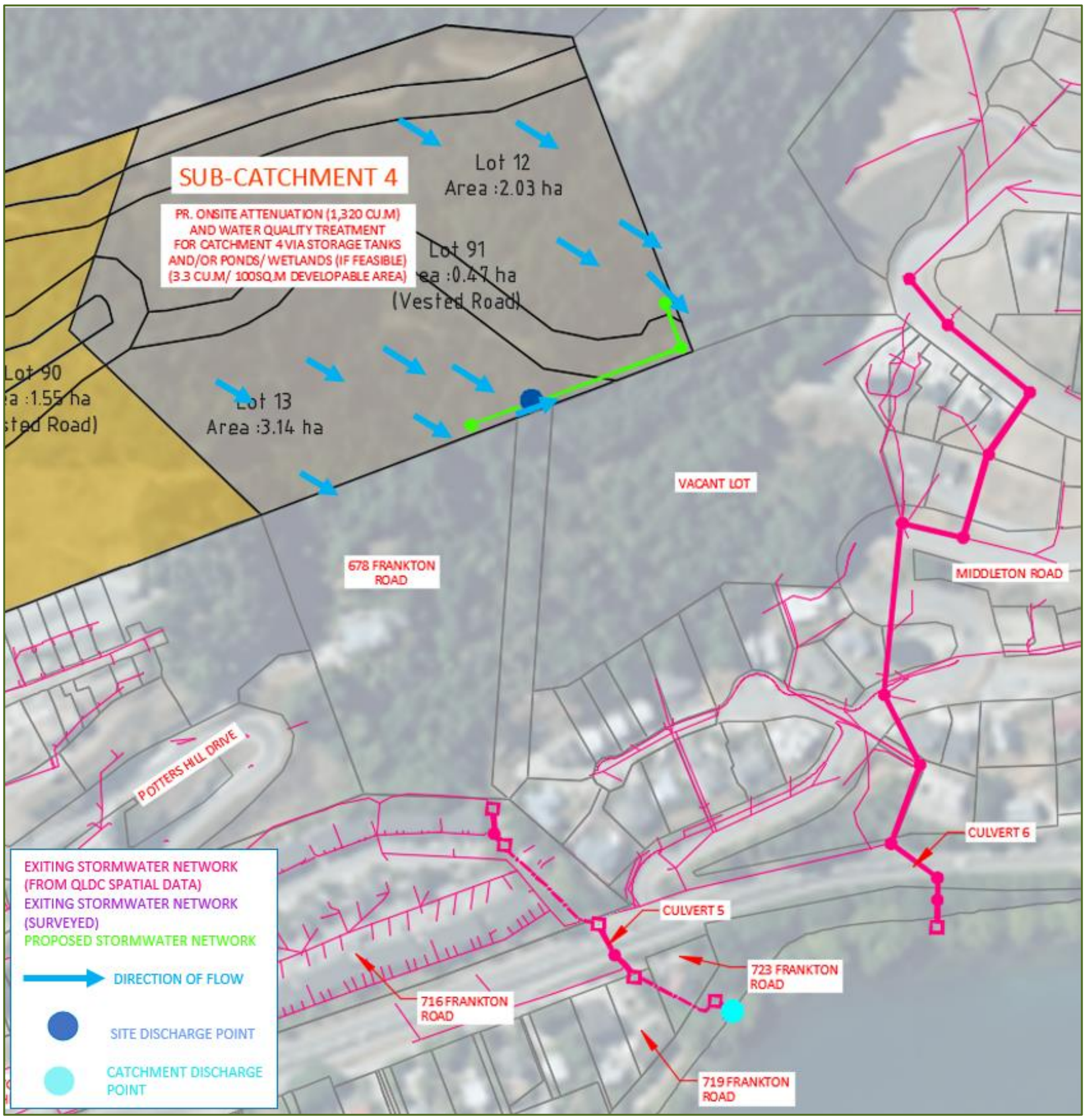


Figure 21. Post Development flows and existing drainage network - Sub catchment 4

Table 11 – Mitigated Peak Flows per 100m^2 for 100Yr Critical Event

SUB CATCHMENT	DEVELOPMENT SITE PEAK FLOW		OUTLET SIZE	MITIGATED POST DEVELOPMENT
	PRE-DEVELOPMENT	POST-DEVELOPMENT		
4	0.54 m^3/s	1.27 m^3/s	15mm	0.54 m^3/s

4. CONCLUSIONS AND RECOMMENDATIONS

4.1. CONCLUSIONS

Based on the hydrological and hydraulic analyses described in this report, approximately 4,450m^3 of attenuation storage has been estimated to be required to mitigate the effects of the proposed development on downstream receiving environments and networks (refer Plan provided in Appendix A). This can be broken down into the relevant sub catchment areas as follows:

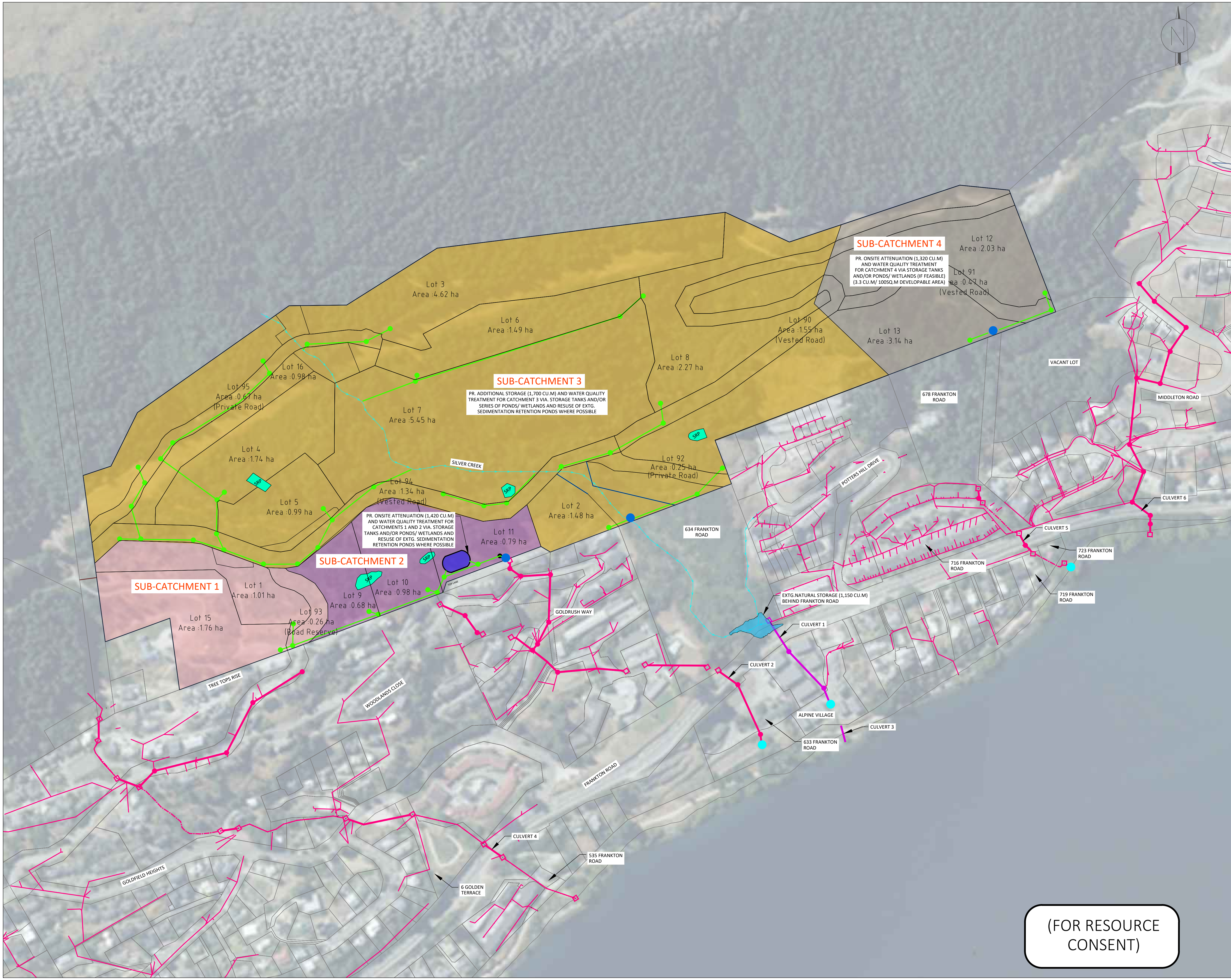
- Sub catchments 1 and 2 (draining into Goldfield Heights and Goldrush Way) to require approximately 1,420m^3 of onsite attenuation by way of a proposed attenuation storage devices.
- Sub catchment 3 (draining into Silver Creek) to require approximately 1,700 m^3 of additional storage (on top of the existing attenuation of 1,150 m^3 estimated to be available upstream of the stormwater culvert at 634 Frankton Road; assuming a 200mm freeboard from existing road levels). It is assumed that this will be provided via a series of attenuation ponds or wetlands, with existing sediment retention ponds on site reused as permanent devices wherever practically possible.
- Sub catchment 4 (draining into the watercourse at 678 Frankton Road) to require approximately 1,320m^3 of onsite attenuation. Further investigation of site terrain will be required to confirm if attenuation above ground storage will be feasible in this area; alternatively, equivalent storage devices within individual lots may be required (and has been estimated as approximately 3.3m^3 per 100m^2 of developed land).

The above will ensure peak discharge rates from the proposed development do not exceed existing development levels, while also ensuring no increase in flood risk downstream of the site (and in particular, the Alpine Village located at 643 Frankton Road).

4.2. RECOMMENDATIONS

In delivering the stormwater design for the proposed development, overland flow paths (OLFPs) shall also be designed to be directed and managed within proposed roading corridors and existing water courses without imposing any risks of flooding to residential property during storm events of up to a 1% AEP (RCP 8.5). All road runoff should also be treated via biofiltration devices or wetlands to minimise the discharge of sediment and associated pollutants to the downstream receiving environment (i.e., Lake Wakatipu).

APPENDIX A - PROPOSED STORMWATER STRATEGY



LEGEND:

EXISTING

SEDIMENTATION RETENTION POND

FROM QLDC SPATIAL DATA
- CHANNEL
- PIPE/CULVERT
- STRUCTURE
- MANHOLE

SURVEYED BY AURUM SURVEY CONSULTANTS
- PIPE/CULVERT
- STRUCTURE
- MANHOLE

PROPOSED
- PIPE
- MANHOLE
- POND/ WETLAND
- SITE DISCHARGE POINT
- CATCHMENT DISCHARGE POINT

Rev.	Detail	App.	Date
A	FOR CONSENT	NW	22.09.22

Revisions

Status
FOR CONSENT

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Client
MOORELIVING

Project
STORMWATER MANAGEMENT FOR SILVER CREEK DEVELOPMENT

Drawing Title
STORMWATER MANAGEMENT PLAN

Scales
0 60 120 Meters
1:2000 @A1 1:4000 @ A3

Project No.	Drawing No.
J000582	1001

Designed	Checked	Approved	Revision
S.R.S.	J.T.	NW	A
SIGNED	SIGNED	SIGNED	

APPENDIX B - MIKE NETWORK MODEL RESULTS

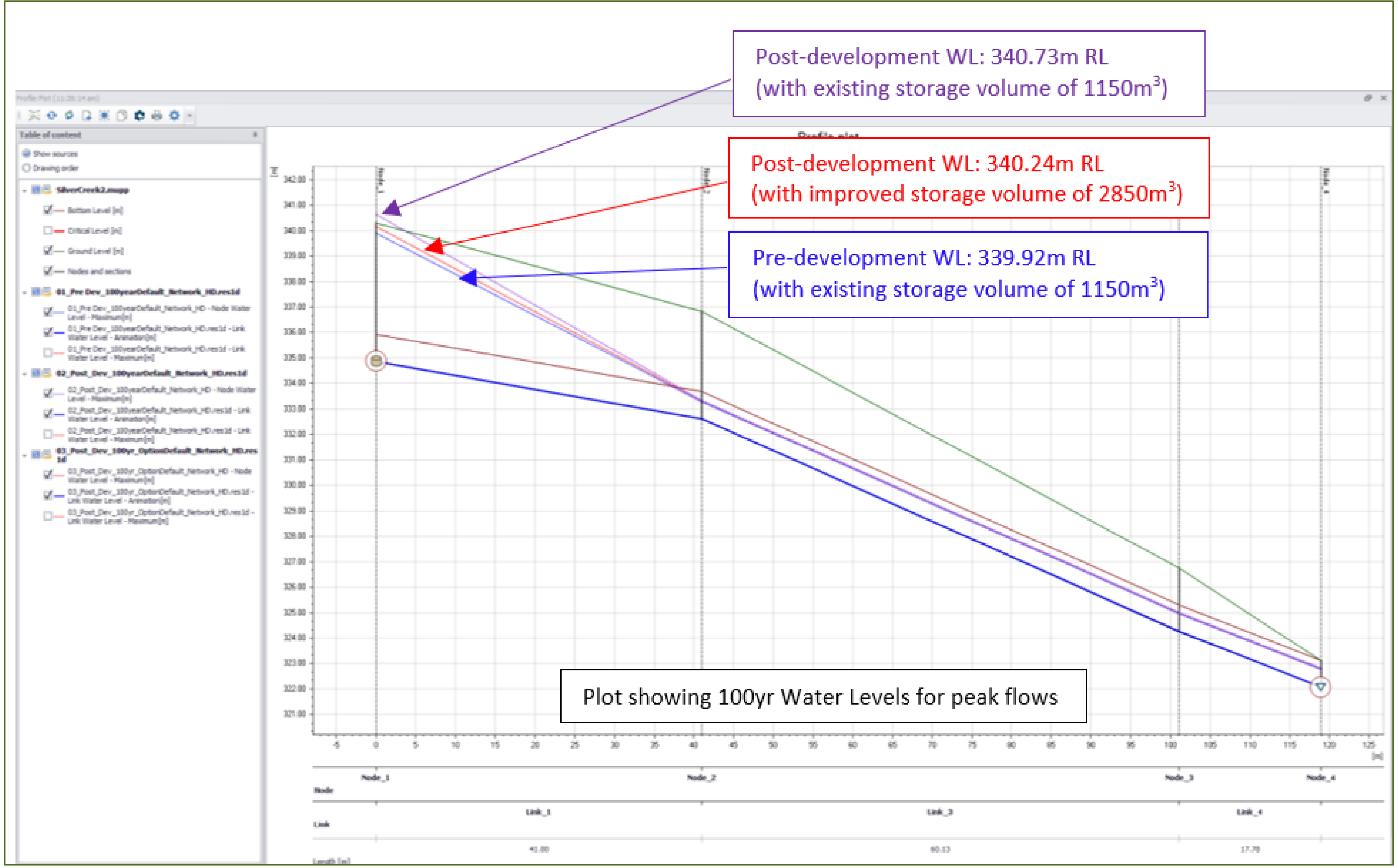


Figure 19. Hydraulic Profile of Stormwater Network Downstream of Culvert 1

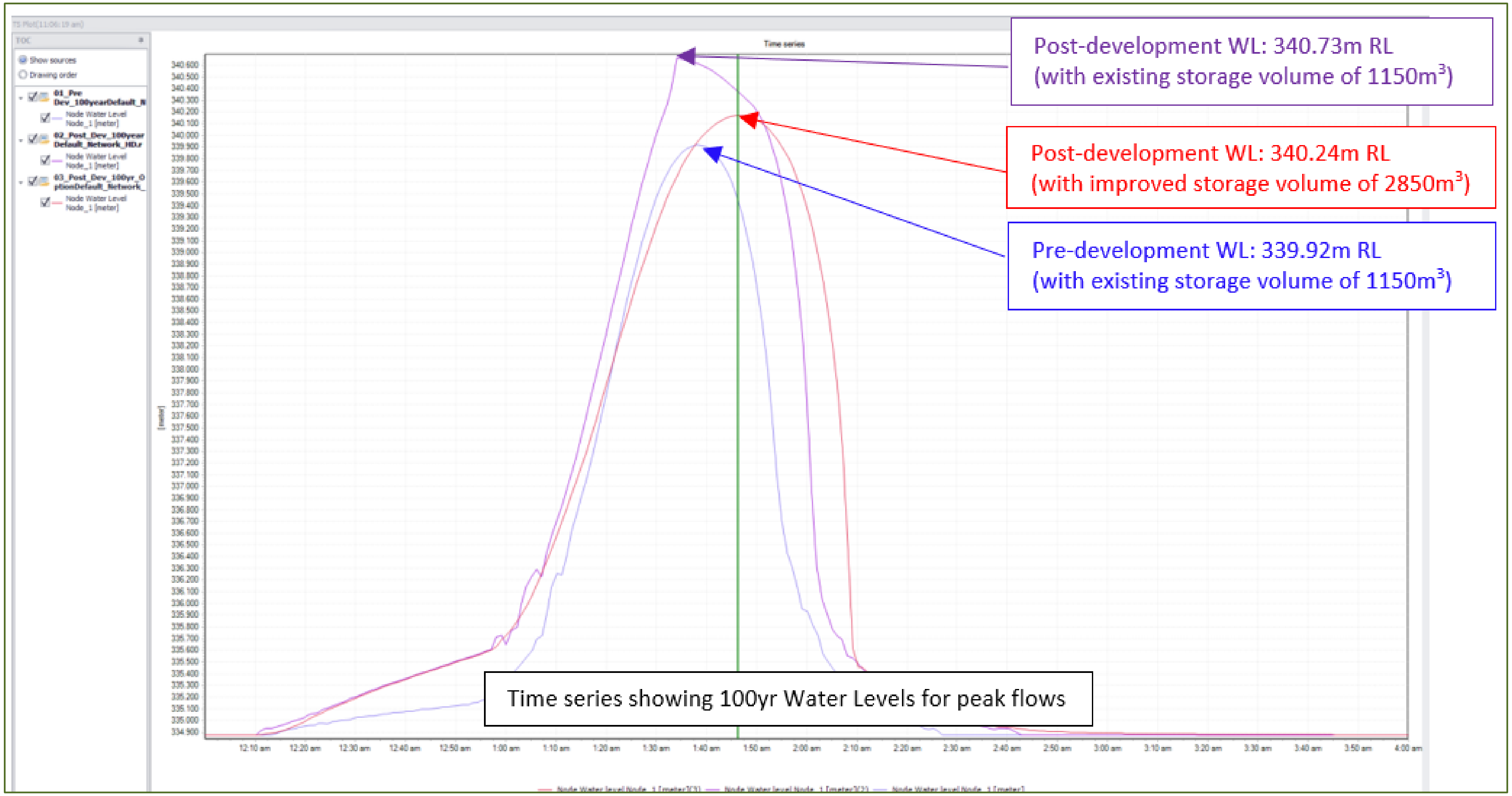
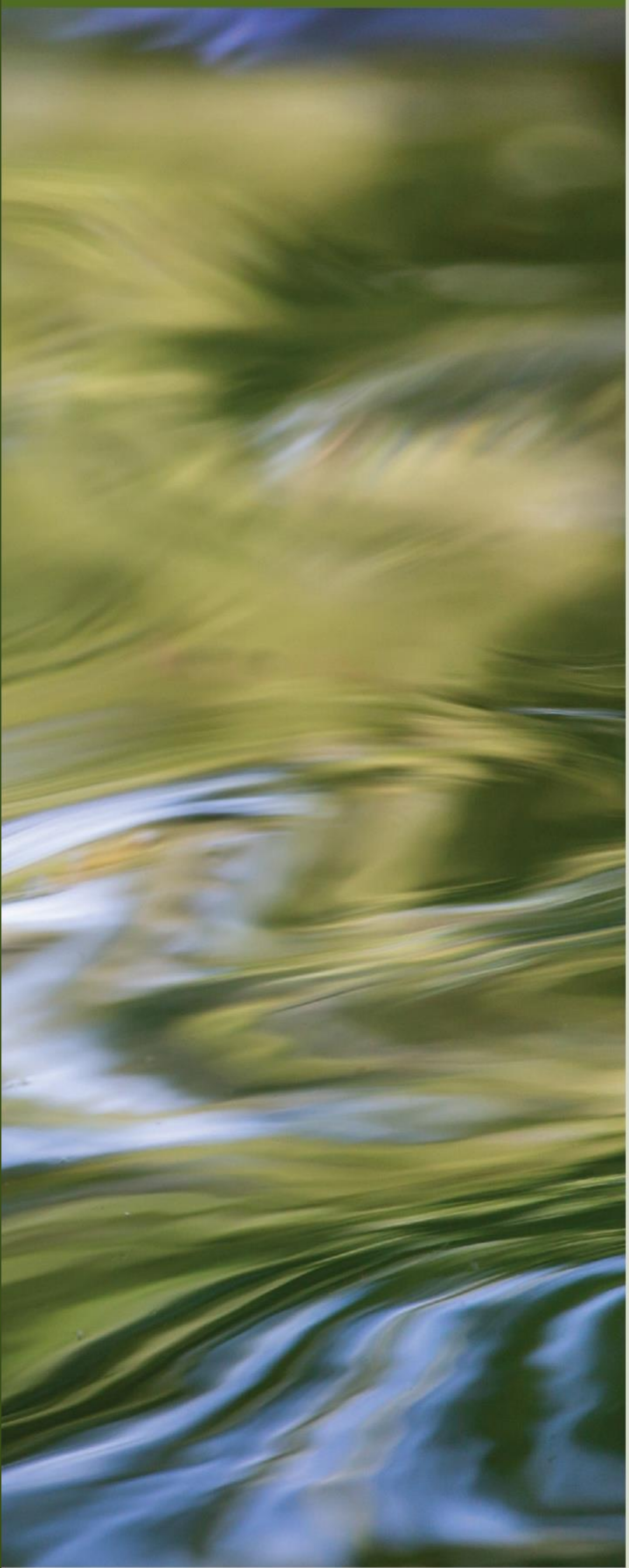


Figure 20. Time Series of Stormwater Network Downstream of Culvert 1



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Appendix C. HAL's Wastewater Reporting

QUEENSTOWN LAKES DISTRICT COUNCIL

SILVER CREEK
DEVELOPMENT IMPACT ASSESSMENT

JULY 2021

HAL HYDRAULIC ANALYSIS LIMITED

HYDRAULIC ANALYSIS LIMITED

QUALITY SECTION

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REVIEWED

Name	Title	Organisation	Signature
Brian Robinson	Director	Hydraulic Analysis Ltd	BDRobinson

REVISION HISTORY

Revision	Publication Date
Draft	21 July 2021

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CONTENTS

1. INTRODUCTION

1.1. OBJECTIVE

The objective of this study is to assess the impact of the proposed Silver Creek development on the QLDC wastewater network. Two different hydraulic models have been utilised for this assessment – the existing hydraulic model (Wakatipu Wastewater Model with HAL updates, 2018) of the Queenstown wastewater network with the current population (2015) scenario was used to assess the impact of the proposed Silver Creek Stage 1 development on the existing wastewater network. Secondly, the Wakatipu Wastewater Master Planning Model (with the Proposed LTP Projects) with the future population (2028) scenario was used to assess the impact of the full development on the network after the completion of the new Frankton Track rising main, that will be running parallel to the existing Frankton Track gravity main.

1.2. BACKGROUND

The Silver Creek development site is located on Goldfield Heights Road, approximately 400m away from the Frankton Track gravity sewer main. The development application seeks approval for subdivision of an existing vacant site into 585 residential dwelling lots, with 3-4 bedrooms per dwelling.

The development proposes three new connection points through which wastewater can enter the QLDC network. The details of the connection points are listed below:

- Goldfield Heights – a gravity connection to the existing 150mm diameter wastewater network along Goldfield Heights to service approximately 150 dwellings which will be stage 1 of the construction phase.
- Potters Hill Drive - a gravity connection to the existing 150mm diameter wastewater network along Potters Hill Drive and it is assumed, approximately 218 dwellings, which is assumed to roughly be half of stage 2 construction phase will be serviced by this connection point.
- Middleton Road - the remaining lots are assumed to be loaded as a gravity connection to the existing 150mm diameter wastewater network at the northern end of Middleton Road.

The network flows southeast via gravity to the Frankton Beach Wastewater Pump Station located at Lake Avenue and from there the flows are pumped to the Frankton Flat Gravity Sewer and eventually to the treatment plant at Shotover Delta Road.

2. SCOPE

The following tasks have been undertaken as part of this assessment:

- Calculation of design flows for the Silver Creek development
- Assessment of the Silver Creek Stage 1 development impact on the existing network for the current (2015) development scenario

- Assessment of the Silver Creek full development impact on the network for the 2028 development scenario with the completion of the proposed new Frankton Track rising main.

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

3. SILVER CREEK DESIGN FLOWS

3.1. OVERVIEW

The Silver Creek development proposal seeks approval for subdivision of an existing vacant site into 585 residential dwelling lots, with 3-4 bedroom dwellings. The location of the proposed development is shown in Figure 3-1 below.

FIGURE 3-1 SILVER CREEK (SILVER CREEK) DEVELOPMENT SITE LOCATION

The development proposes a gravity connection to the existing 150mm diameter wastewater network along Goldfield Heights for stage 1 of the construction phase, which is approximately 150 dwellings and the flows from the remaining 435 dwellings have been split equally between discharge points on Potters Hill and Middleton Road wastewater pipelines, as shown in Figure 3-2 below. The network flows southeast via gravity to the Frankton Beach WWPS.

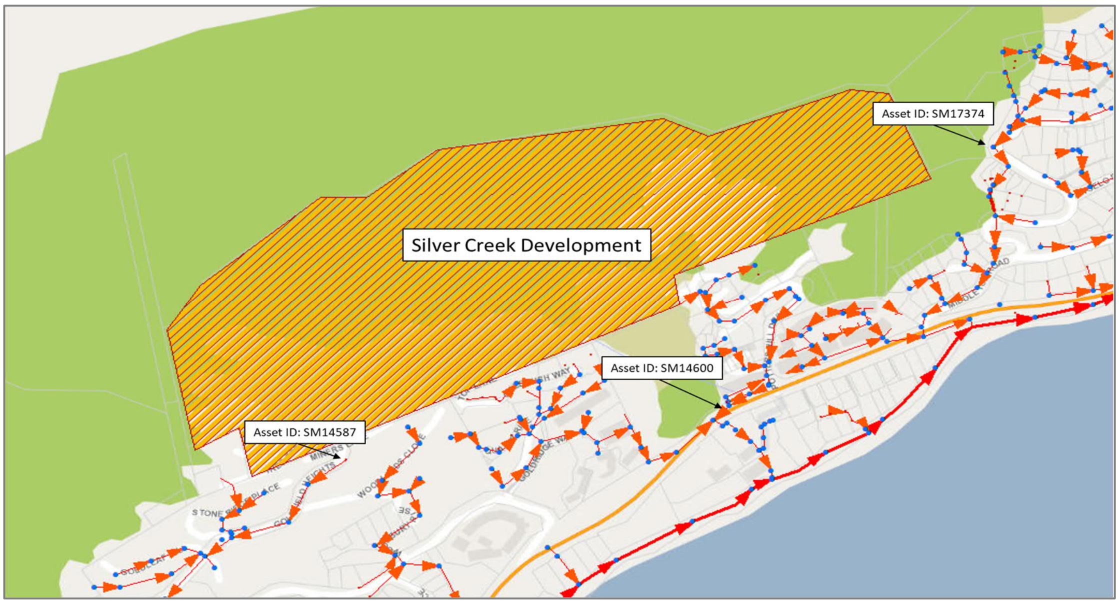


FIGURE 3-2 SILVER CREEK PROPOSED WASTEWATER CONNECTION

3.2. DEVELOPMENT DESIGN FLOWS

The Silver Creek proposal seeks to develop 585 residential dwelling lots, with 3-4 bedroom dwellings proposed.

The design wastewater flows have been calculated using the QLDC 'Land Development and Subdivision Code of Practice', which assumes an average dry weather flow of 250 litres/person/day, a dry weather diurnal peaking factor of 2.5, and a wet weather dilution/infiltration factor of 2 (i.e. a peak wet weather flow (PWWF) of 5x average dry weather flow (ADWF)).

The development proposes subdivision of 585 new residential dwelling lots with a mixture of 3-4 bedrooms, and an assumed occupancy of 3 people per dwelling. This equates to a design PWWF of 25.39 l/s, however, the construction has been phased out into 2 stages as shown in

Table 3-1 below.

TABLE 3-1: SILVER CREEK DEVELOPMENT DESIGN FLOWS

	Residential Lots (Stage 1)	Residential Lots (Stage 2)
No. of Units	150	435
Occupancy	3	3
Population	64	1305
ADWF (l/p/day)	*250*	*250*
ADWF (l/s)	1.30 l/s	3.78 l/s
DWF Peaking Factor	*x2.5*	*x2.5*
PDWF (l/s)	3.26 l/s	9.44 l/s

WWF Peaking Factor	x2	x2
PWWF (l/s)	6.51 l/s	18.88 l/s

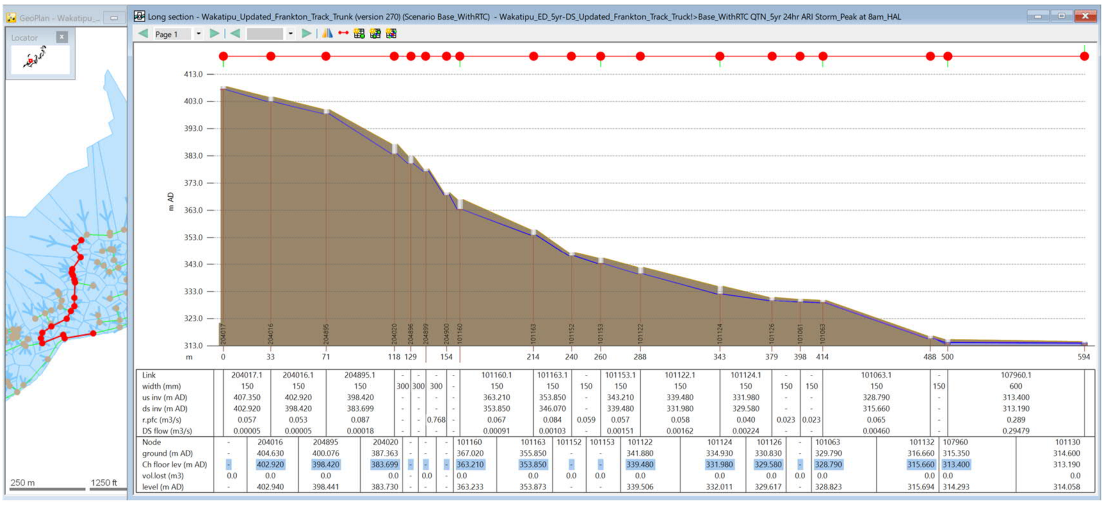
4. SILVER CREEK DEVELOPMENT IMPACT ASSESSMENT

4.1. PRE-DEVELOPMENT SCENARIO

The existing Wakatipu wastewater model (with 2018 HAL updates) was refined in 2019 with new manhole level and pipe invert survey information provided by QLDC from a historical survey. The model was run under the current (2015) population scenario, without the proposed Silver Creek development. A monthly seasonal DWF profile was applied to the model to represent increased visitor numbers during peak periods, with a maximum peaking factor of 1.1x calibrated DWF over the Dec/Jan period. The network was assessed against a 5-year ARI design storm.

As shown in the Figure 4-1 long section below, the existing 150mm local wastewater network shows some evidence of pipe surcharge near the base of the catchment flowing into the 600mm diameter trunk sewer, with water levels reaching within 500mm of lid level at several different locations along the trunk sewer and predicted manhole overflow volume of 36.6m^3 at MH ID: 101127.

Whilst the existing Frankton Track gravity sewer is known to be an existing constraint, it should be noted that there has been no records of reported overflows at this location. It is considered the scenario modelled is conservative, as it assumes a 5 year ARI storm in conjunction with a peak occupancy scenario, which in reality is unlikely to occur every 5 years (on average) due to the short duration of the peak occupancy reducing the likelihood on coinciding with a 5 year ARI storm. In addition, surveyed lid levels at this manhole appear low compared to surrounding manholes and the pipe invert/soffit (resulting in cover of approximately 200mm). Hence it is recommended the lid level is resurveyed to better quantify the risk of overflows from this location.



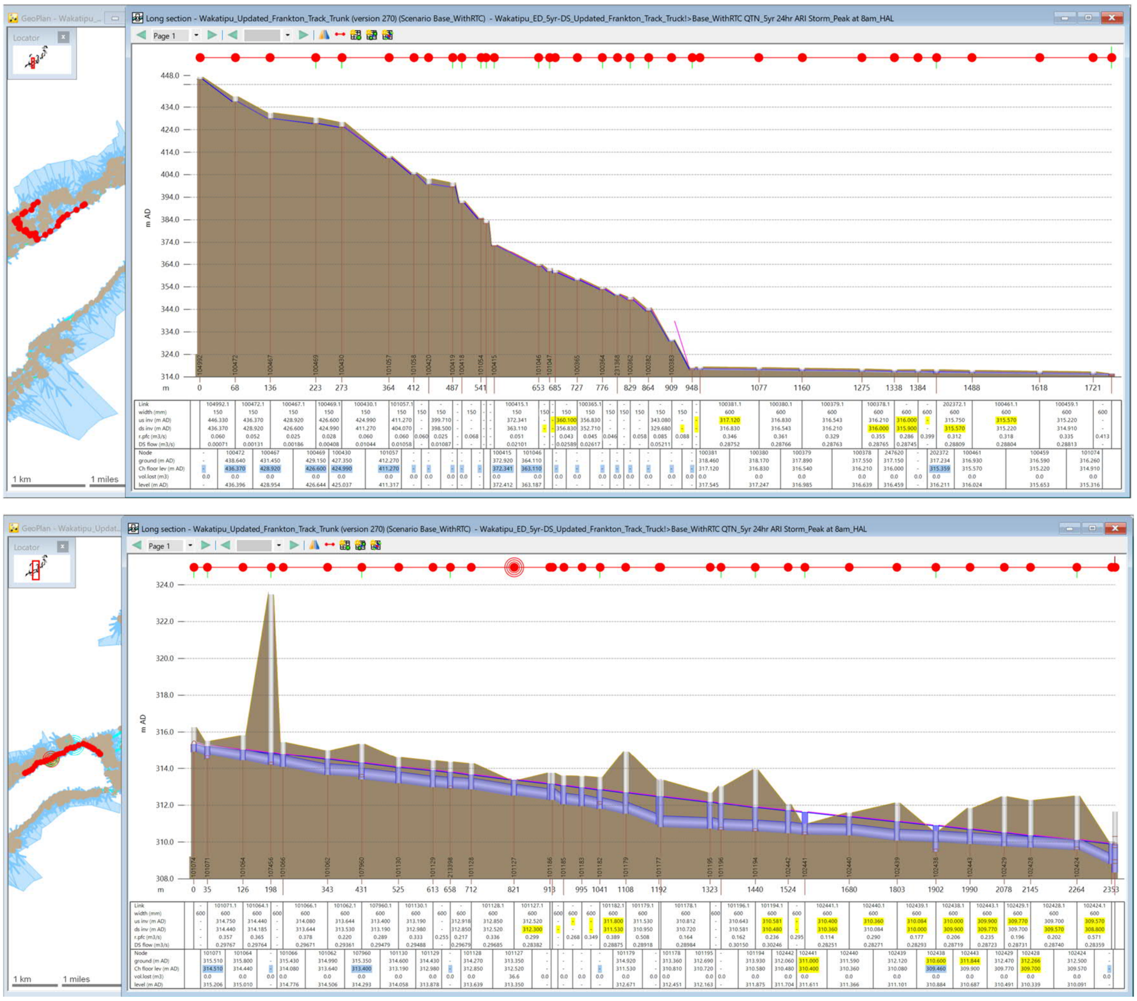


FIGURE 4-1 EXISTING (2015) LONG-SECTION – 5 YEAR ARI DESIGN STORM

4.2. POST-DEVELOPMENT SCENARIO – SILVER CREEK STAGE 1

The Wakatipu wastewater model (with 2018 HAL updates) was run under the current (2015) population scenario, with the additional peak wet weather flow of 6.51 l/s from the proposed Silver Creek, Stage 1 development. The flows were added in as a direct gravity connection to Asset ID:SM14587 on the existing 150mm uPVC wastewater line along Goldfield Heights. The development impact was assessed against a 5-year ARI design storm to understand the performance of the network.

As shown in the Figure 4-2 long-section below, the existing 150mm local network shows evidence of increased pipe surcharge flowing into the 600mm diameter trunk sewer, and increased surcharge in the Frankton Track gravity sewer with water levels reaching within 500mm of lid level at several different locations and an increase in predicted manhole overflow volume from 36.6m^3 to 92.6m^3 at MH ID: 101127. However, there are no additional overflow incidents simulated in the downstream network as a result of the increase in PWWFs from the development.

As noted previously, it is considered the scenario modelled is conservative, as it assumes a 5 year ARI storm in conjunction with a peak occupancy scenario, which in reality is unlikely to occur every 5 years (on average) due to the short duration of the peak occupancy reducing the likelihood on coinciding with a 5 year ARI storm. However as the Frankton Track gravity sewer is already at or close to capacity in a 5 year ARI storm, additional flows resulting from this development will increase the risk of uncontrolled overflows until the proposed Frankton Track rising main (which will receive flows from the proposed Recreation Grounds pump station) is built, reducing flows in the existing Frankton Track gravity sewer.

The details of the overflow volume and surcharge levels are summarised in Table 4-1 below.

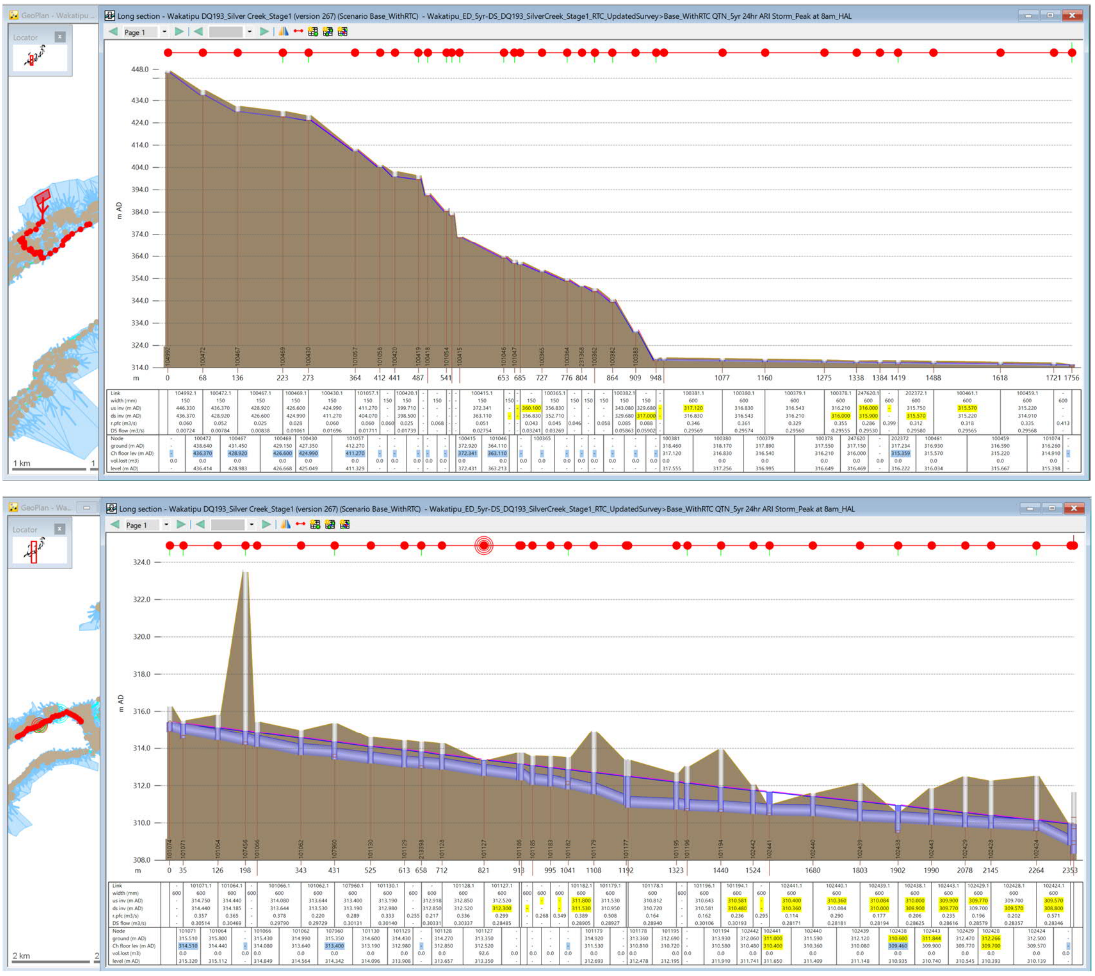


FIGURE 4-2 SILVER CREEK STAGE 1 (6.51 L/S) LONG-SECTION – 5 YEAR ARI DESIGN STORM

TABLE 4-1: SILVER CREEK MANHOLE OVERFLOW VOLUME AND PREDICTED OVERFLOW LOCATIONS

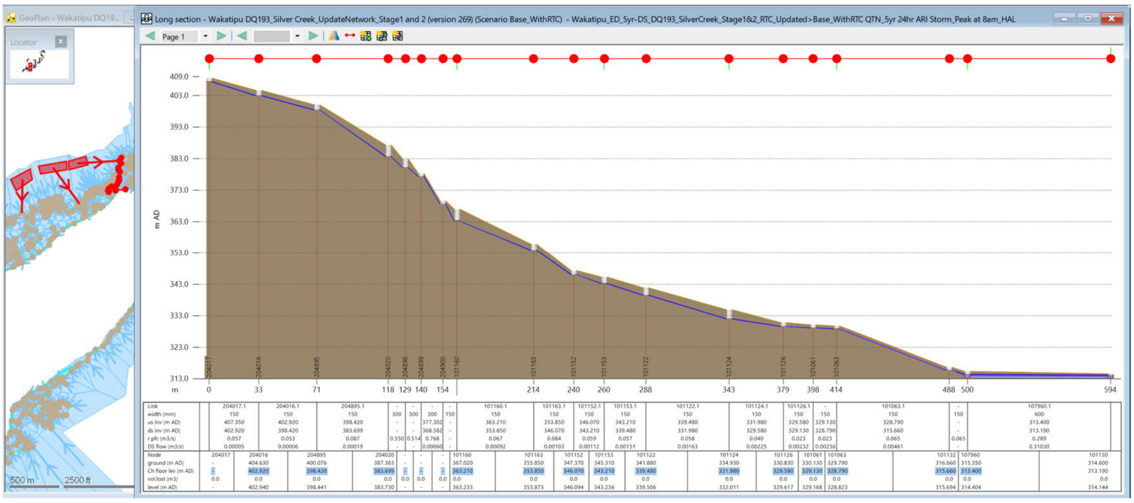
MANHOLE ID	Volume Lost (m3)		Level of Surcharge from Lid Level (mm)	
	Pre-development Scenario	Post Development Scenario – Stage 1	Pre-development Scenario	Post Development Scenario – Stage 1
101127	36.60	92.60		
101130			542	504
100415			508	489
101071			304	190
101062			484	426
101127			At lid level	At lid level
102440			224	181
102420			396	71

4.3. POST-DEVELOPMENT SCENARIO – SILVER CREEK STAGE 1 & 2

The model has been run with the additional PWWFs of 18.88 l/s from the proposed Silver Creek, Stage 2 of the development, added in as a direct gravity connection to Asset ID:SM14600 (9.44 l/s) on the existing 150mm wastewater line adjacent to 658 Potters Hill Drive and to Asset ID: SM17374 (9.44 l/s) which is adjacent to 44 Middleton Road, to assess the capacity of the network to receive the cumulative proposed development flows.

As shown in Figure 4-3 below, the existing 150mm local wastewater network shows evidence of further increase in pipe surcharge near the base of the catchment flowing into the 600mm diameter trunk sewer and water levels reaching within 500mm of lid level at several manholes along the trunk sewer, indicating an increased risk of overflows, with the inclusion of the flows from both stage 1 and 2. Furthermore, predicted manhole overflow volume increased from 36.6m^3 to 172.2m^3 at MH ID: 101127.

The details of the overflow volume and surcharge levels are summarised in Table 4-2 below.



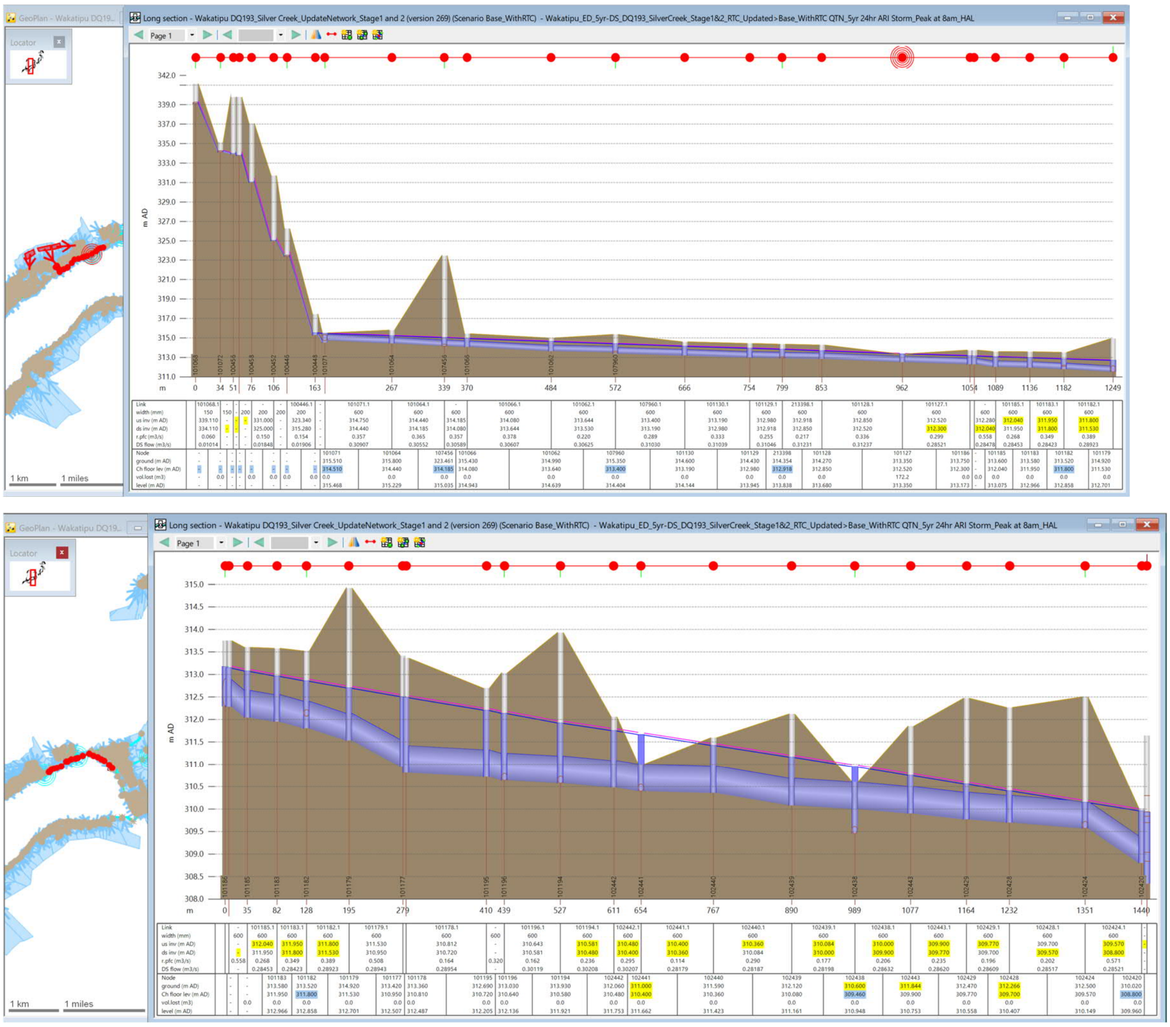


FIGURE 4-3 SILVER CREEK STAGE 1 AND 2 (25.39 L/S) LONG-SECTION – 5 YEAR ARI DESIGN STORM

TABLE 4-2: SILVER CREEK MANHOLE OVERFLOW VOLUME AND PREDICTED OVERFLOW LOCATIONS

MANHOLE ID	Volume Lost (m3)		Level of Surcharge from Lid Level (mm)	
	Pre-development Scenario	Post Development Scenario – Stage 1	Pre-development Scenario	Post Development Scenario – Stage 1 & 2
101127	36.60	172.2		
101130			542	456
100415			508	489
101071			304	42
101062			484	351
101127			At lid level	At lid level
102440			224	167
102420			396	6

As the Frankton Track gravity sewer is already at or close to capacity in a 5 year ARI storm, additional flows resulting from the full development will increase the risk of uncontrolled overflows from the existing Frankton Track gravity sewer until the proposed Frankton Track rising main (which will receive flows from the proposed Recreation Grounds pump station) is built, reducing flows in the existing Frankton Track gravity sewer. However it is understood that the proposed Frankton Track Rising main won't be constructed for at least 3 years, so the timing and staging of the development will need to be carefully considered alongside the timing of this project to avoid an unacceptable risk of overflows.

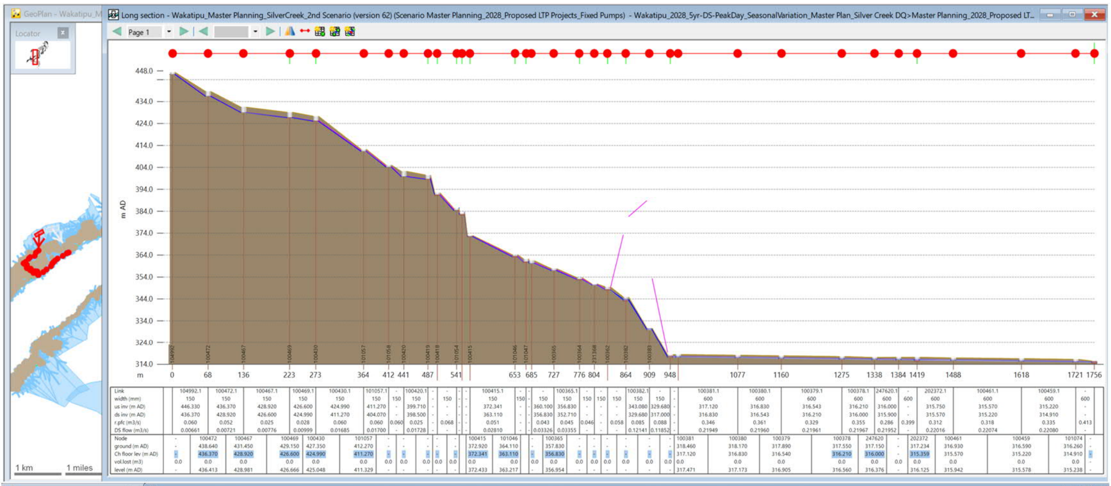
The developer has proposed the implementation of either a low pressure or STEP sewer system to enable wastewater flows to be better controlled and discharged off peak, and likely reduce the wet weather flows, minimising the impact on downstream network constraints. QLDC have advised that they don't consider this to be a desirable long term solution. However it is considered that a similar impact could potentially be achieved in terms of attenuating peak wet weather flows from a traditional gravity system by implementing buffer storage along with an appropriately sized orifice to limit discharge to approximately peak dry weather flow for the various stage of the development, storing wet weather flows above this within the buffer storage facility.

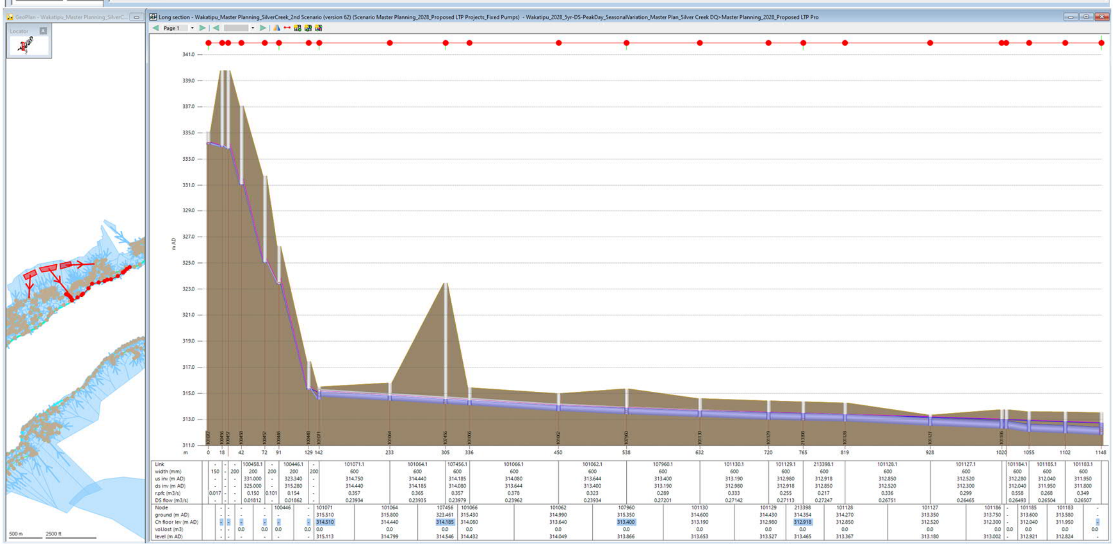
4.4. POST-DEVELOPMENT SCENARIO WITH FRANKTON TRACK RISING MAIN – SILVER CREEK STAGE 1&2

The Wakatipu Wastewater Master Planning Model (with the Proposed LTP Projects) was run under the future (2028) population scenario, with the additional peak wet weather flow of 25.39 l/s from the proposed Silver Creek Stage 1 and 2 developments. It should be noted that the 'Proposed LTP Projects' scenario used for this assessment includes the already committed Res Grounds PS and associated rising main which will eventually be connected to the proposed Frankton Track Rising main, hence significantly reducing the load to the existing Frankton Track Gravity main.

The flows were added in as a direct gravity connection to Asset ID: SM14587 (6.51l/s) on the existing 150mm uPVC wastewater line along Goldfield Heights, Asset ID:SM14600 (9.44 l/s) on the existing 150mm wastewater line adjacent to 658 Potters Hill Drive and to Asset ID: SM17374 (9.44 l/s) which is adjacent to 44 Middleton Road, to assess the capacity of the network to receive the cumulative proposed development flows. The development impact was assessed against a 5-year ARI design storm to understand the performance of the network.

As shown in the Figure 4-4 long-section below, the existing 150mm local network shows evidence of pipe surcharge on the Frankton Track gravity sewer (particularly at the bottom end) but the additional development flows do not result in any uncontrolled manhole overflow events within the downstream local network or along the Frankton Track trunk sewer. However, there is evidence of water levels reaching within 500mm of lid level at several different locations along the trunk sewer, primarily at manholes with limited cover, as detailed in Table 4-3 below.


Long section - Wakatipu_Master Planning_SilverCreek_2nd Scenario (version 62) (Scenario Master Planning_2028_Proposed LTP Projects_Fixed Pumps) - Wakatipu_2028_5yr-DS-PeakDay_SeasonalVariation_Master Plan_Silver Creek DQ>Master Planning_2028_Proposed LT...
m AD
1 km
1 miles



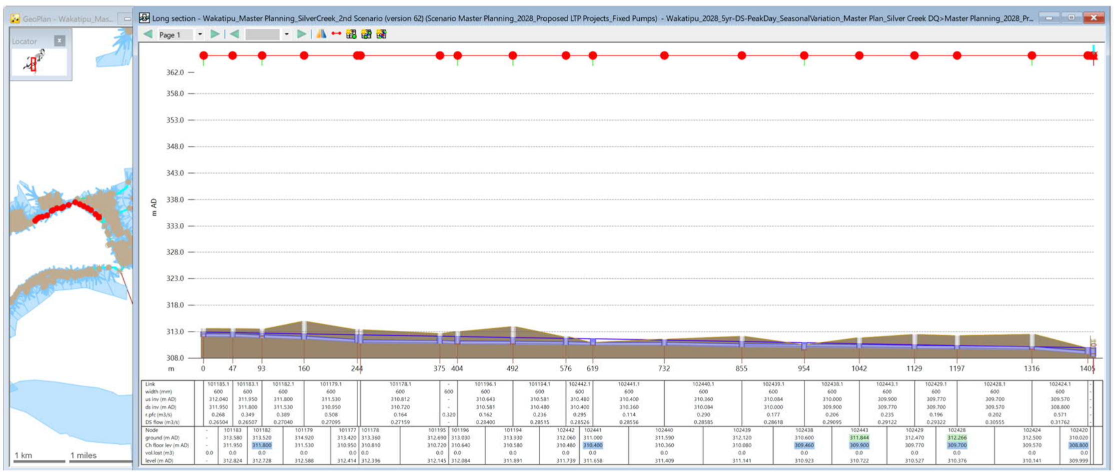

Long section - Wakatipu_Master Planning_SilverCreek_2nd Scenario (version 62) (Scenario Master Planning_2028_Proposed LTP Projects_Fixed Pumps) - Wakatipu_2028_5yr-DS-PeakDay_SeasonalVariation_Master Plan_Silver Creek DQ>Master Planning_2028_Pr...
m AD
1 km
1 miles

FIGURE 4-4 SILVER CREEK STAGE 1 AND 2 (25.39 L/S) LONG-SECTION WITH THE PROPOSED FRANKTON TRACK RISING MAIN – 5 YEAR ARI DESIGN STORM

It should be noted that the predicted surcharge (and corresponding risk of overflow from the below list of manholes) can likely be mitigated by reducing pump capacities at Marine Parade and Park St pump stations to match expected inflows, and/or potentially diverting flows from the Park St PS to the proposed Frankton Track Rising Main, which would reduce the load on the Frankton Track gravity sewer. It is recommended options to reduce peak flows and mitigate the risk of overflows from the existing Frankton Track gravity sewer are investigated by QLDC as part of the design of the proposed Frankton Track Rising Main to ensure spare capacity is available in the existing Frankton Track gravity sewer for future development.

TABLE 4-3: SILVER CREEK MANHOLE OVERFLOW VOLUME AND PREDICTED OVERFLOW LOCATIONS

MANHOLE ID	Level of Surcharge from Lid Level (mm)	
	Pre-development Scenario	Post Development Scenario – Stage 1&2
100415	508	487
101071	406	397
101127	244	170
102420	161	20

4.5. FRANKTON BEACH WASTEWATER PUMP STATION ASSESSMENT

The Frankton Track trunk sewer flows southeast, discharging via gravity to the Frankton Beach Wastewater Pump Station B located in Lake Avenue, approximately 2.70km from the development site. There is another Pump Station located adjacent (Frankton Beach Wastewater Pump Station A) which currently receives flows from the Kelvin Heights and Frankton Flats area, with an interconnection between the two pump stations.

The Frankton Road WWPS B has a maximum capacity of 220 l/s with one pump operating and 330 l/s with both pumps (based on QLDC records). The pre-development scenario simulates a peak inflow of approximately 275 l/s during the 5-year design storm indicating that the pump station and associated rising main have ample pass forward capacity.

When considering the cumulative PWWF of 25.35 l/s from both stage 1 and 2 of the proposed development, this results in an increase in peak inflow to the WWPS B of 285 l/s under a 5-year ARI design storm indicating that the modelled flows are less than the maximum pump capacity. Hence, the existing pump station is considered to have sufficient capacity to receive the additional cumulative development flows from the proposed site under the current population (2015) scenario.

During the 2028 population scenario (with the Proposed LTP Projects), the Frankton Road WWPS B has a maximum capacity of 330 l/s with both pumps in operation (pump 1 – 220 l/s and pump 2 – 110 l/s), in which case the pump station and rising main has sufficient capacity to pass forward the peak inflow of 314 l/s which includes the flows from both stage 1 and 2 of the proposed development. However, it should be noted that even with the additional pump in operation, the pump station is nearing capacity and is likely to experience capacity constraints with further significant developments in the contributing catchment.

According to the QLDC Wastewater Master Plan 2020, it is assumed that the “flows from the Frankton Track rising main would discharge into the southern of the Frankton Beach PS’s wet wells (Frankton Beach Pump Station A) and that the northern pump station (Frankton Beach Pump Station B) will continue to receive flows from existing Frankton Track gravity sewer”. Hence, as already recommended in section 4-4, it will be beneficial to reduce PS capacities at Marine Parade and Park St pump stations to match expected inflows, and/or divert the Park St PS to the proposed Frankton Track rising main, reducing the load on both the Frankton Track gravity sewer and Frankton Beach Pump Station B.

Modelled inflows and outflows for the post-development, 2015 and 2028 population scenarios are shown in Figure 4-5 and 4-6 below.

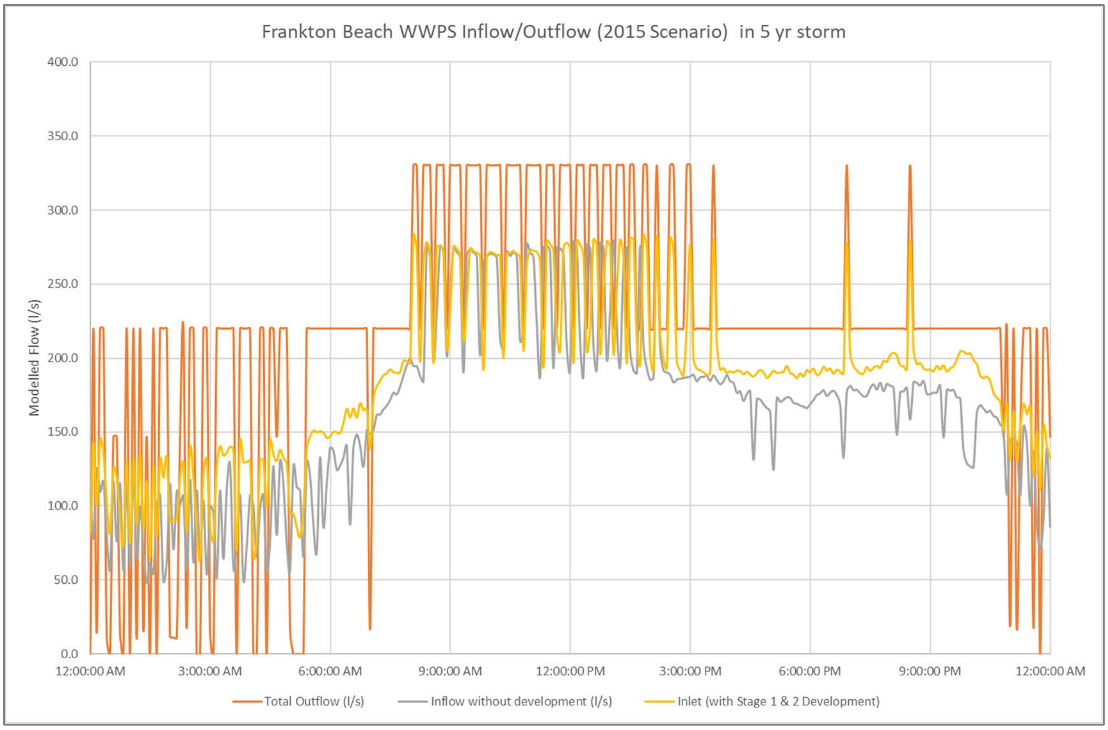


FIGURE 4-5 MODELLED SILVER CREEK WWPS FLOWS – 5 YEAR ARI DESIGN STORM WITH 2015 POPULATION SCENARIO

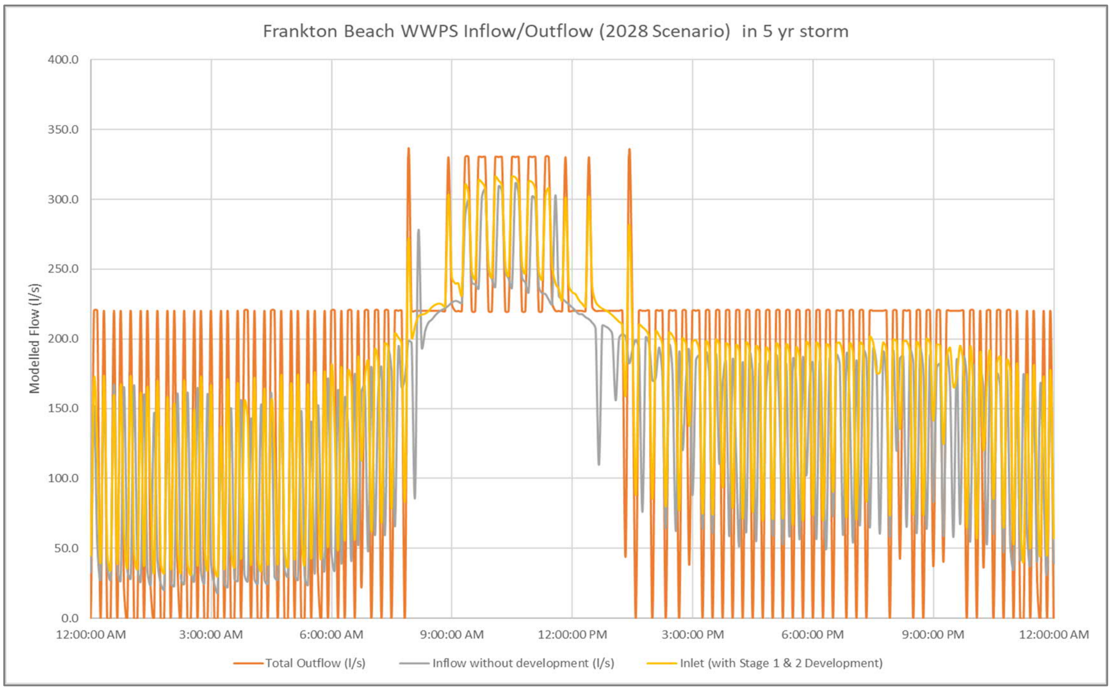


FIGURE 4-6 MODELLED SILVER CREEK WWPS FLOWS – 5 YEAR ARI DESIGN STORM WITH 2028 POPULATION SCENARIO

5. MODEL ASSUMPTIONS AND LIMITATIONS

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

- 'Wakatipu Wastewater Model Build & Calibration Report' (Beca, August 2016)
- 'Wakatipu Wastewater Network Future System Performance Report' (Beca, August 2017)
- 'Wakatipu Wastewater Model Review & Update – High & Medium Priority Fixes Memo' (HAL, 2018)
- Wastewater Master Plan report (Morphum, 2020)

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

- The model was originally calibrated against flows developed from field data collected in 2015 and supplemented by QLDC pump station SCADA data. The 2018 model review undertaken by HAL has determined only a medium degree of confidence in the accuracy of the model. Additional flow gauging and model re-calibration is proposed for 2019.
- The distribution of the modelled population is an approximation based on the 2013 census residential population, factored up for a high population scenario. No allowance has been made for additional growth since 2013, other than known development areas.
- Modelled network asset data for manholes and pipes is generally as provided in the BECA calibration model, and its origin is not clear. Manhole and pipe level data has not been validated against QLDC's GIS, as-builts or survey data as part of this assessment, or as part of the HAL model review/update. Where potential network constraints are identified, it is recommended asset data in these areas is confirmed through manhole survey.
- Pump station model parameters have been determined based on information provided by the QLDC planning team, SCADA data (where available) and pump station manuals, and the accuracy has not been validated as part of these studies.
- This assessment excludes information on any additional recently consented neighbouring developments in the contributing catchment.
- This assessment focuses on the wastewater network downstream of the site, and does not consider sizing of infrastructure within the proposed site to service future development upstream of the site.
- It has been assumed that no existing overarching structure plan has been developed by QLDC for servicing this area.

6. CONCLUSION

The objective of this study is to assess the impact of the proposed Silver Creek development on the QLDC wastewater network. The existing hydraulic model (Wakatipu Wastewater Model with HAL updates, 2018) with the 2015 population scenario and the Wakatipu Wastewater Master Planning Model (with the Proposed LTP Projects) with the 2028 population scenario were used to assess the impact of the development on the network.

The development proposes subdivision of 585 new residential dwelling lots with a mixture of 3-4 bedrooms, and an assumed conservative occupancy of 3 people per dwelling. This equates to a design PWWF of 25.39 l/s.

6.1. PRE-DEVELOPMENT SCENARIO

In the pre-development scenario, the existing downstream network shows evidence of some pipe surcharge near the base of the catchment flowing into the 600mm diameter trunk sewer, with water levels reaching within 500mm of lid level at several different locations along the trunk sewer and predicted manhole overflow volume of 36.60m^3 at MH ID: 101127.

Whilst the existing Frankton Track gravity sewer is known to be an existing constraint, it should be noted that there has been no records of reported wet weather overflows at this location. It is considered the scenario modelled is conservative, as it assumes a 5 year ARI storm in conjunction with a peak occupancy scenario, which in reality is unlikely to occur every 5 years (on average) due to the short duration of the peak occupancy reducing the likelihood on coinciding with a 5 year ARI storm. In addition, surveyed lid levels at this manhole appear low compared to surrounding manholes and the pipe invert/soffit (resulting in cover of approximately 200mm). Hence it is recommended the lid level is resurveyed to better quantify the risk of overflows from this location.

6.2. POST-DEVELOPMENT SCENARIO – STAGE 1 & 2 WITH EXISITNG NETWORK

In the post-development scenario, with the inclusion of the Silver Creek Stage 1 discharge, there is evidence of increased pipe surcharge flowing to the trunk sewer and an increase in predicted manhole overflow volume from 36.6m^3 to 92.6m^3 at MH ID: 101127. After the addition of flows from both Stage 1 and 2, there is evidence of further pipe surcharge and an increase in surcharge levels at several manholes along the network, with an increase in predicted manhole overflow volume to 172.2m^3 at MH ID: 101127. However, there are no additional overflow incidents simulated in the downstream network as a result of the increase in PWWFs from the development.

As the Frankton Track gravity sewer is already at or close to capacity in a 5 year ARI storm, additional flows resulting from the full development will increase the risk of uncontrolled overflows from the existing Frankton Track gravity sewer until the proposed Frankton Track rising main (which will receive flows from the proposed Recreation Grounds pump station) is built, reducing flows in the existing Frankton Track gravity sewer.

It is understood that the proposed Frankton Track Rising main won't be constructed for at least 3 years, so the timing and staging of the development will need to be carefully considered alongside the timing of that project to avoid an unacceptable risk of overflows.

The developer has proposed the implementation of either a low pressure or STEP sewer system to enable wastewater flows to be better controlled and discharged off peak, and likely reduce the wet weather flows, minimising the impact on downstream network constraints. QLDC have advised that they don't consider this to be a desirable long term solution. However it is considered that a similar impact could potentially be achieved in the short term in terms of attenuating peak wet weather flows from a traditional gravity system by implementing buffer storage along with an appropriately sized orifice to limit discharge to approximately peak dry weather flow for the various stage of the development, storing wet weather flows above this within the buffer storage facility.

6.3. POST-DEVELOPMENT SCENARIO – STAGE 2 WITH PROPOSED NETWORK

Finally, with the inclusion of the additional PWWFs in the future (2028) population scenario, the 150mm local network shows evidence of pipe surcharge flowing to the trunk sewer but the additional development flows do not result in any uncontrolled manhole overflow events within the downstream local network or along the Frankton Track trunk sewer. However, there is evidence of water levels reaching within 500mm of lid level at few different locations along the trunk sewer.

It should be noted that the predicted surcharge (and corresponding risk of overflow) can likely be mitigated by reducing pump capacities at Marine Parade and Park St pump stations to match expected inflows, and/or potentially diverting flows from the Park St PS to the proposed Frankton Track Rising Main, which would reduce the load on the Frankton Track gravity sewer.

It is recommended options to reduce peak flows and mitigate the risk of overflows from the existing Frankton Track gravity sewer are investigated by QLDC as part of the design of the proposed Frankton Track Rising Main to ensure spare capacity is available in the existing Frankton Track gravity sewer for future development.

The Frankton Beach WWPS is shown to have sufficient capacity for expected flows from the full development, assuming that flows from the proposed Frankton Track rising main would discharge into the southern of the two Frankton Beach PS's wet wells (Frankton Beach Pump Station A) and that the northern pump station (Frankton Beach Pump Station B) will receive flows from existing Frankton Track gravity sewer, and downstream constraints are resolved allowing both pump stations to operate simultaneously, as envisaged in the QLDC Wastewater Master Plan (2020).

7. RECOMMENDATIONS

The following recommendations are made in relation to this proposed development:

- Undertaken manhole survey on manholes on Frankton Track predicted to overflow (in particular Manhole 101127) or close to overflowing (within 500mm of lid level), to better quantify risk of overflows.
- Confirm timing and staging of development in conjunction with expected timing of proposed Frankton Track Rising main to ensure risk of overflows in minimised.
- The developer should investigate options to attenuate flows from the first stage of the development to approximately expected peak dry weather flows through the implementation of an appropriately sized orifice and buffer storage facility to store excess wet weather flows to minimise the impact on the existing downstream Frankton Track gravity sewer until such time as the Frankton Track Rising Main is constructed.
- QLDC should investigate options to reduce peak flows and mitigate the risk of overflows from the existing Frankton Track gravity sewer as part of the design of the proposed Frankton Track Rising Main to ensure spare capacity is available in the existing Frankton Track gravity sewer for future development.

HAL HYDRAULIC ANALYSIS LIMITED

J0516 Silver Creek Development Impact Assessment – Stage 1 Attenuated Flows (Draft B)

To:	Brandon Ducharme	QLDC
Distribution:	Morgana Zanotto Later, Richard Powell	QLDC
From:	Brian Robinson (HAL), Michelle Mak (HAL)	
Subject:	Silver Creek Proposed Development – Stage 1 Attenuated Flows	
Date:	Thursday 14th March 2023	

1 Introduction

1.1 Objective

The objective of this study is to re-assess the impact of the attenuated flows resulting from Stage 1 of the proposed Silver Creek development has on the existing Frankton Track gravity sewer using the refined hydraulic model (Wakatipu Wastewater Model with updates, 2018). The scope of works include:

1) Update the 2018 hydraulic model following on from the manhole surveys completed for the Frankton Track gravity sewer and the inclusion of the newly built Rec Grounds Pump Station (PS);
2) Further refined the validated wet weather parameters at flow monitored locations contributing to Marine Parade PS and Park Street PS.
3) Validate the modelled flows against the measured flows in 2020/21 at the previously monitored location on the Frankton track gravity sewer (QT15 at manhole 102428) for four calibration events.
4) Re-assess the impact of the Stage 1 proposed Silver Creek development has on the existing Frankton Track gravity sewer by attenuating the flows to ~Peak Dry Weather Flow, interlocking the Rec Grounds PS and provision of 90m^3 storage at Park Street PS.

1.2 Background

An assessment was previously undertaken in 2021 to understand the impact of the proposed Silver Creek development has on the existing network using the existing hydraulic model that was refined in 2018 however the Rec Grounds PS was excluded from this assessment. The outcomes of this assessment showed risk of overflows from the existing Frankton Track gravity sewer based on the previous model parameters and asset information. Asset surveys were recommended following on from this assessment to confirm the lid levels of several low-lying manholes located on the Frankton Track gravity sewer.

Further to this, QLDC is currently upgrading Park Street PS to increase its capacity and provide additional emergency storage tanks with a provisional storage volume of 90m^3. A new connection from Marine Parade PS to a redundant storage tank on Park St was also included as part of this upgrade to provide remote storage for Marine Parade PS in the event of the PS being unable to cope with the incoming flows.

Flow gauging was implemented by AML in June 2020 and was completed in August 2021. Measured flows were available for a number of significant wet weather events which can be used to validate the model outputs to give a better understanding of the existing hydrology and ultimately determine the accuracy and reliability of the model in predicting issues in the network under future development scenarios.

In addition, the construction of the Rec Grounds PS was completed in late 2022 which will intercept flows from much of the existing Marine Parade PS catchment, and pump directly to the Frankton Track gravity sewer through a new rising main. QLDC have identified an urgent need to understand the performance of the existing Frankton Track gravity sewer with Rec Grounds PS contributing 165l/s downstream.

This memo summarises the model validation outcomes, update process and should be read in conjunction with the previously submitted Silver Creek Development Query Assessment memo (HAL, 2021).

2 Model Validation

2.1 Overview

The existing hydraulic model was updated with 2022 average day population and validation of the model outputs against the recently completed flow monitoring in 2021 was undertaken to achieve a model that is sufficiently representative of the existing flows in the wastewater catchments contributing to the Frankton Track gravity sewer.

The figure below shows the monitored locations relevant to this assessment. While previous model validation was completed for the catchments contributing to Marine Parade and Park Street PS under a different work package, they were further refined as part of this assessment to give a better representation of flows in the Frankton Track gravity sewer (Flow Monitored location QT15). This is achieved by comparing the modelled versus observed flow hydrographs at this location and adjustment of wet weather flow parameters to achieve reasonable match for the selected validation events.

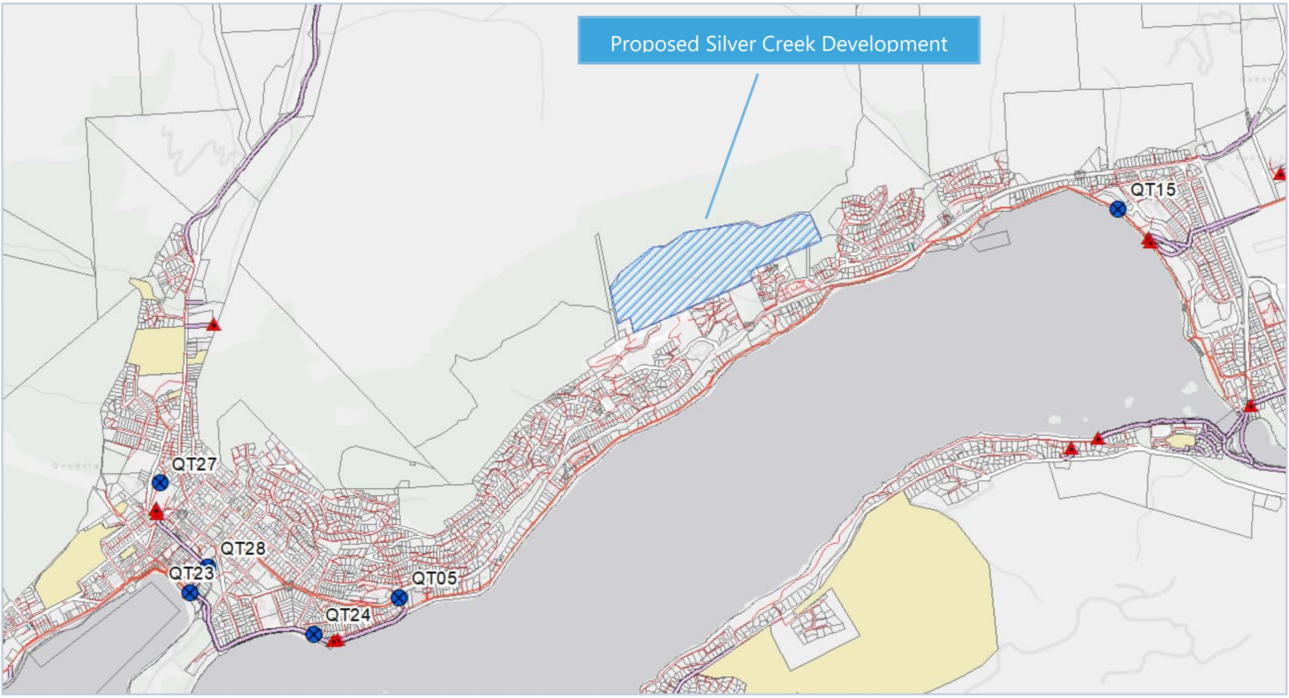


Figure 2-1: Map showing the monitored locations relevant to this assessment

2.2 Model Validation

2.2.1 Dry Weather Validation

The catchments contributing to Marine Parade PS and Park Street PS were previously validated and therefore this assessment focus on adjusting the DWF parameters for QT15 subtract catchments. For this assessment, the Code of Practice per capita flow of 250 l/person/day adopted and it was considered sensible given that some areas in the current model has higher per capita flows).

2.2.2 Wet Weather Validation

A high-level review of the modelled flows vs observed flows were undertaken for the above flow monitored locations for four wet weather events.

The wet weather events selected for use in the model validation process vary from a 3 Month to an 8 year ARI and are typically of moderate intensity (noting the rainfall data is only currently available in hourly increments as outlined above). See below Table 2-1 for adopted wet weather validation periods.

Table 2-1: Identified WWF Validation Events

Event ID	Start Date	Duration (Days)	Depth (mm)	Peak Intensity (mm/Hr)	ARI (24 Hours)	Primary Usage
WWF_1	20/07/2020	1.04	61	11.60	4 Year	Validation
WWF_2	31/08/2020	0.5	10.20	6.80	3 Month	Validation
WWF_3	25/10/2020	0.75	30.2	5.2	4 Month	Not used
WWF_4	01/01/2021	2.58	76.60	8.40	8 Month	Validation
WWF_5	06/07/2021	1.38	65	10.20	8 Year	Validation

Note: In total 5 Wet Weather Events were considered for the validation process and 4 were chosen based on the quality of flow data available at the six validation locations.

Once a reasonable match between modelled and observed flows was achieved for the dry weather flow events at all five key locations, the wet weather flow parameters for each catchment under consideration was adjusted following a comparison of the observed wet weather flow for each gauge with the corresponding modelled wet weather flow. During the process, multiple scenarios were plotted and compared against the observed flow data.

2.2.3 WWF Model Validation Criteria

The targeted WWF calibration tolerances are as follows:

- Wet weather Peak Flow Rate - < +30% to –30%
- Wet Weather Volume - < +30% to –30%
- Peak Depth +/- 20%

Table 2-2 below summarises the adjusted wet weather parameters for the previously validated catchments and QT15 subtract catchment. Note that this validation exercise was undertaken without Rec Grounds PS as this PS was not commissioned until late 2022.

Table 2-2: Refined WWF Parameters for Various Flow Monitored Locations

Gauge ID	Manhole Compkey	Location	Landuse ID	Original WWF Parameters*			Refined WWF Parameters		
				Fast Response (%)	Slow Response (%)	Impervious runoff Routing Value	Fast Response (%)	Slow Response (%)	Impervious runoff Routing Value
QT05	100109	133 Frankton Road	7	1.5%	0%	1	Unchanged	Unchanged	Create a new Runoff Surface ID 8

									with a routing value of 5
QT15	102428	17 Yewlette Crescent	9	1.25%-2.5%	0%	70	1.25%**	Unchanged	Unchanged
QT23	100134	22 Earl Street	6	2%	0%	1	1.5%	Unchanged	Unchanged
QT24	216761	104 Park Street	9	2%	2%	1	Unchanged	Unchanged	Unchanged
QT27	100265	58 Camp Street	6	2%	0%	1	1.5%	Unchanged	Unchanged
QT28	178292	3 Camp Street	6	2%	0%	1	1.5%	Unchanged	Unchanged

*Previous validation was undertaken using existing 2018 Wakatipu hydraulic model with HAL updates.

** A wet weather fast response connected area of 1.25% of the subtract catchment contributing to QT15 was adopted as 1.25% is the figure that is used for the majority of the current model (with some areas having a lower % than that) and was considered to result in approximately 5 x ADWF (i.e. equivalent to COP design flows) from the local catchment in a 5 year design storm and was considered to provide a reasonable match with the observed peak flows at QT15 flow gauge location though it was considered conservative and it is recommended this is refined as part of the model recalibration work package.

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Table 2-3: WWF Validation Statistics

Location	Gauge	Gauge Type	Catchment Type	Event	Volume				Peak Discharge				Peak Depth				Comments
					Observed (m3)	Simulated (m3)	Volume Difference (m3)	Error (%)	Observed (l/s)	Simulated (l/s)	Peak Flow Difference (l/s)	Error (%)	Observed (m)	Simulated (m)	Peak Depth Difference (m)	Error (%)	
375mm gravity to Frankton Track	QT05	HVQ	Subtract	Validation 1	1,680	1,673	-7	-0.5%	27	27	0.0	0%	0.10	0.09	0.01	5%	A good representative calibration between the observed and predicted flows at this site for Events 1 and 2. However, for Event 4 (01/01/2021) the simulated flow volume is 25% higher, which may be due to the minor periods of velocity "drop-outs" or "ragging", noted in the contractor's Flow Monitoring Report, for the same period. And as for Event 5 (06/07/2021), the simulated flow volume is 22% lower and the simulated peak discharge is 29% lower, this is potentially due to variation in rainfall distribution across the short-term rain gauges.
	QT05	HVQ	Subtract	Validation 2	1,033	1,226	-193	19%	15	15	0.0	0%	0.08	0.07	-0.01	-8%	
	QT05	HVQ	Subtract	Validation 4	1,294	1,621	327	25%	31	23	-8.0	-26%	0.11	0.09	-0.02	-20%	
	QT05	HVQ	Subtract	Validation 5	2,331	1,813	-518	-22%	41	29	-12	-29%	0.12	0.09	-0.03	-25%	
600mm Frankton Track Gravity Sewer	QT15	HVQ	Subtract	Validation 1	21,801	22,156	355	2%	194	258	64	33%	0.49	0.54	0.05	10%	A good representative calibration between the observed and predicted volumes at this site for all events. However, the peak discharge calibration tolerance cannot be met for Events 1, 2 and 5. The model consistently overpredicts the peak discharge by 32-45% for these events which may be due to under reading of observed data, potentially due to velocity 'drop-outs' during peak of the storms. This could also because of the flow spikes from upstream pump stations attenuating slower in the model resulting in higher peak discharges.
	QT15	HVQ	Subtract	Validation 2	17,570	18,602	1,032	6%	129	148	19	15%	0.4	0.36	-0.1	-10%	
	QT15	HVQ	Subtract	Validation 4	21,803	19,259	2,544	-12%	157	207	50	32%	0.44	0.47	0.03	7%	
	QT15	HVQ	Subtract	Validation 5	22,186	23,538	1,352	6%	193	279	86	45%	0.42	0.72	0.30	71%	
U/S of Marine Parade PS	QT23	HVQ	Subtract	Validation 1	5,613	5,506	-107	-2%	89	82	-7	-7%	0.49	0.2	-0.29	-59%	A good representative calibration between the observed and predicted volumes at this site for all events except for Event 5. However, the validation criteria could not be met for the peak discharge for all the events except for Event 1 without compromising the calibration criteria for the other locations and events.
	QT23	HVQ	Subtract	Validation 2	3,500	4,427	924	26%	35	56	21	60%	0.4	0.16	-0.24	-60%	
	QT23	HVQ	Subtract	Validation 4	3,403	4,370	967	28%	43	72	29	67%	0.44	0.16	-0.28	-63%	
	QT23	HVQ	Subtract	Validation 5	3,826	5,905	2,079	54%	63	91	28	44%	0.42	0.19	-0.23	-55%	
U/S of Park St PS	QT24	HVQ	Leaf	Validation 1	1,046	948	-98	-9%	21	21	0	0%	0.35	0.13	-0.22	-62%	Low confidence is placed in the data measured at this site. In spite of the low-quality data, a reasonable representative calibration between the observed and predicted flows at this site for events 1, 2 and 4. However, for Event 5 (06/07/2021) the simulated volume is 49% higher, which may be due to silt and debris build up in the pipe invert, as noted in the contractor's Flow Monitoring Report, for the same period.
	QT24	HVQ	Leaf	Validation 2	804	747	-57	-7%	17	12	-5	-29%	0.13	0.10	-0.03	-23%	
	QT24	HVQ	Leaf	Validation 4	967	813	-154	-16%	18	18	0	0%	0.12	0.13	0.01	6%	
	QT24	HVQ	Leaf	Validation 5	722	1,081	359	49%	18	22	-4	22%	0.15	0.14	-0.01	-6%	
U/S of proposed Rec Grounds PS	QT27	HVQ	Subtract	Validation 1	2,249	2,310	61	3%	41	40	-1	2%	0.17	0.19	0.02	12%	A good representative calibration between the observed and predicted flows at this site for Events 1, 2 and 4. However, the validation criteria could not be met for the flow volumes for Event 5 without compromising the calibration criteria for the other locations and events.
	QT27	HVQ	Subtract	Validation 2	1,895	1,921	26	1%	28	29	1	4%	0.14	0.15	0.01	7%	
	QT27	HVQ	Subtract	Validation 4	1,949	2,131	182	9%	30	32	2	7%	0.15	0.17	0.02	13%	
	QT27	HVQ	Subtract	Validation 5	1,286	2,571	1,285	100%	35	42	7	20%	0.15	0.19	0.04	27%	
150mm on Camp St	QT28	HVQ	Leaf	Validation 1	1,134	1,374	240	21%	21	17	-4	-19%	0.14	0.12	0.02	14%	A good representative calibration between the observed and predicted flows at this site for Event 1 but validation criteria could not be met for the flow volumes for Event 4 and 5 without compromising the calibration criteria for the other locations and events.
	QT28	HVQ	Leaf	Validation 2	810	1,178	368	45%	10	12	2	20%	0.10	0.10	0	0%	
	QT28	HVQ	Leaf	Validation 4	627	1,307	680	108%	20	15	-5	-25%	0.14	0.11	-0.03	-21%	
	QT28	HVQ	Leaf	Validation 5	985	1,441	456	46%	17	19	2	12%	0.12	0.19	0.07	58%	

Key

Failure of Calibration Tolerance
Achieved Calibration Tolerance

3 Model Update

3.1 Summary of Updates

This study has utilised the existing hydraulic model (Wakatipu Wastewater Model with HAL updates, 2018) of the Queenstown wastewater network and the population was updated to reflect the projected current population.

In addition to the model validation undertaken as described in the above section, a number of other model updates have been made specific to this study to improve the modelled representation of the critical assets including:

- Updated the model to incorporate recent manhole survey undertaken by the Silver Creek developer, noting that one of the critical low-lying manhole (ID 101127) has a surveyed lid level lower than previously adopted in the model and also suggested the pipe soffit as modelled would be above ground level. The previously adopted lid level was used for this reason.
- Updated the model doe the development assessment to reflect recent (or soon to be implemented) PS upgrades at Marine Parade & Park St PS's, as well as latest model representation of Rec Grounds PS.
- Represent latest understanding of proposed pump station interlocking philosophy at Rec Grounds PS as provided by QLDC.

The model updates for the key pump stations and which model scenarios these updates were applied to are summarised below:

Table 3-1: Modelled Pump Capacities

PS Name	No of Modelled Pumps	Maximum Capacity (l/s)	Emergency Storage Volume (m3)	Interlocking Philosophy	Development Scenario
Marine Parade PS	1	165	0 (however an overflow gravity pipe is connected from this PS to the re-purposed storage tank at Park St PS)	None	Existing (2022) Pre-Development Scenario and Post Development Scenario
Rec Grounds PS	1	165	550	This PS can be inhibited when Marine Parade PS operates	Existing (2022) Pre-Development Scenario and Post Development Scenario
Park St PS	1	52	90	None	Existing (2022) Post Development Scenario

4 Development Impact Assessment

4.1 Silver Creek Attenuated Flows

As previously assessed, the Frankton Track gravity sewer was predicted to overflow from the low-lying manholes as a result of the downstream hydraulic constraints under the existing development scenario. Therefore, the Silver Creek development proposal seeks approval for Stage 1 development only with a proposed attenuation flow of 3x Average Dry Weather Flow through an on-site balance tank. The location of the proposed development is shown in Figure 4-1 below.

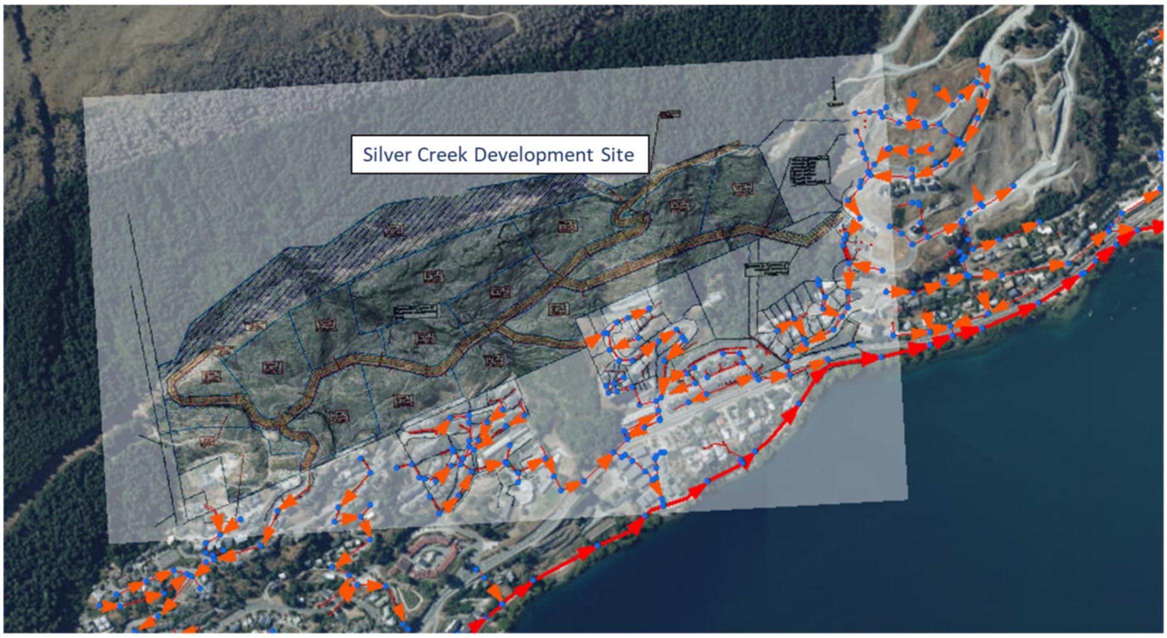

Figure 4-1 Silver Creek (SILVER CREEK) DEVELOPMENT SITE location

The Stage 1 development proposes a gravity connection to the existing 150mm diameter wastewater network along Goldfield Heights, which is approximately 150 dwellings. It is understood that the Stage 2 development which comprises approximately 435 dwellings is likely to be assessed at a later stage when the new rising main from Rec Grounds PS is built to divert its flows to the network downstream of Frankton Track gravity sewer.

The proposed Stage 1 development has been modelled dynamically using the wastewater profile adopted for the neighbouring catchment and the peak flows were attenuated through a 20m^3 balance tank and an orifice restricting a flow of 3.5l/s into the 150mm gravity network along Goldfield Heights. See below the design flows for the Stage 1 development.

Table 4-1: Silver Creek Development Design Flows

Item	Residential Lots (Stage 1)
No. of Units	150
Occupancy	3
Population	64
ADWF (l/p/day)	*250*
ADWF (l/s)	1.30 l/s
DWF Peaking Factor	*x2.5*

PDWF (l/s)	3.26 l/s
WWF Peaking Factor	*x2*
PWWF (l/s)	6.51 l/s
Attenuated Flow (l/s)	3.5 l/s

Real time control (RTC) rules previously adopted for the two Frankton Beach PS's to prevent both pump stations operating at the same time have been adopted for all scenarios (however 2 pumps within a given wet well can pump simultaneously). No other committed or proposed projects (other than that as described in Section 3 of this memo) have been included in the existing scenarios.

The sections below summarise the analysis of the model results for the pre-development and post development scenarios under a 5-year synthetic (nested) design storm.

4.2 Pre-Development Scenario

The existing Wakatipu wastewater model (with 2018 HAL updates) was validated against the recently completed flow monitored measured data as outlined in the above sections and updated with new manhole lid level survey information provided by the developer.

The model was run under the current (2022) population scenario, without the proposed Silver Creek Stage 1 development. A monthly seasonal DWF profile was applied to the model to represent increased visitor numbers during peak periods, with a maximum peaking factor of 1.1x calibrated DWF over the Dec/Jan periods.

As shown in the Figure 4-2 long section overleaf, the existing 150mm local wastewater network shows some evidence of pipe surcharge near the base of the catchment flowing into the 600mm diameter Frankton Track gravity sewer trunk sewer however no overflows were predicted from this local network and the Frankton Track gravity sewer, which is backed up by the no reported incidents for this area. Although it is also noted that there is significant surcharge at the bottom section of the Frankton Track gravity sewer almost to the point of overflow predicted at manhole 101127.

It is considered the scenario modelled is conservative given that the simulated peak flow was higher than the observed flow during the validation process, and that it assumes a 5-year ARI storm in conjunction with a peak occupancy scenario, which in reality is unlikely to occur every 5 years (on average) due to the short duration of the peak occupancy reducing the likelihood on coinciding with a 5 year ARI storm.

The model results differ to that from the previous assessment in that the model was predicting overflows at manhole 101127 because the previous model was overpredicting the peak flows from Hill catchments contributing to the flow gauge location QT15. The WWF parameters were refined to give a better presentation of the flows at this location under this study.

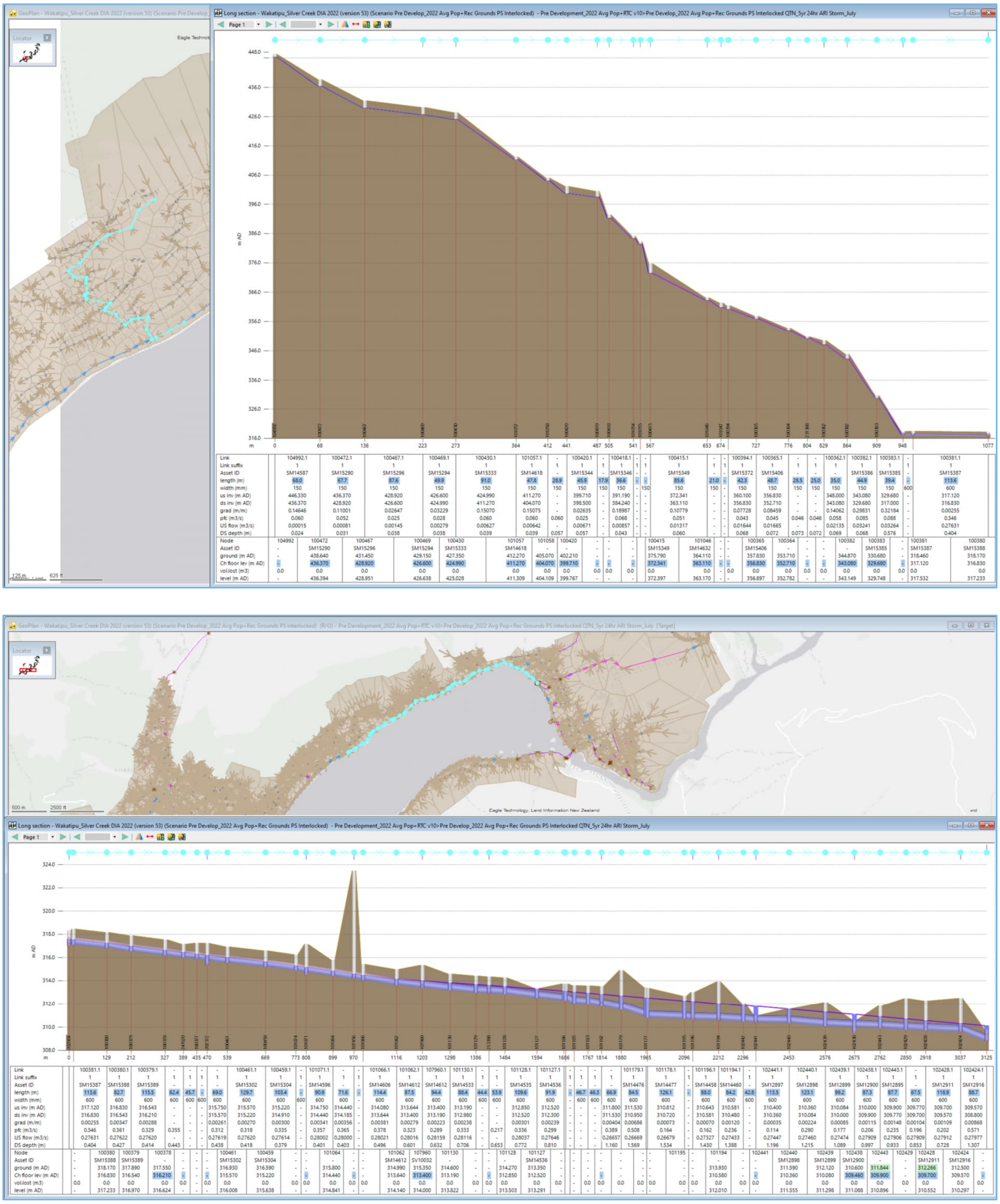

Figure 4-2 Existing (2022) Pre-Development Scenario LONG-SECTION – 5 year ARI design storm

4.3 Post-Development Scenario – Silver Creek Stage 1 Attenuated Flow

The Wakatipu wastewater model (with 2018 HAL updates) was run under the current (2022) population scenario, with the additional attenuated flow of 3.5 l/s from the proposed Silver Creek, Stage 1 development.

As described above, the flows were added in using an orifice connected to Asset ID:SM14587 on the existing 150mm uPVC wastewater line along Goldfield Heights and a 20m^3 balance tank on site to store the wet weather flows.

Two different post development scenarios were assessed against a 5-year ARI design storm to understand the performance of the network with and without the 90m3 emergency storage at Park Street PS.

4.3.1 Post Development Scenario – Existing Park Street PS Capacity of 30l/s and no storage at Park Street PS

This scenario was assessed to understand the performance of the network in particular the Frankton Track gravity sewer with the addition of the Stage 1 development attenuated flow without the emergency storage of 90m^3 currently being constructed at Park Street PS and Park Street PS with its existing (i.e. pre upgrade) of 30l/s.

As shown in the Figure 4-3 long-section overleaf the existing 150mm local network shows evidence of minimal increase in pipe surcharge flowing into the 600mm diameter trunk sewer, and minimal increase in surcharge in the Frankton Track gravity sewer almost to the point of overflow predicted at manhole 101127. This is expected given that this was predicted in the existing pre-development scenario where the significant surcharge caused by the hydraulic constraints at the bottom section of the Frankton Track gravity sewer and Frankton Beach Pump Stations.

There are no other overflow incidents simulated in the downstream network as a result of the increase in attenuated flows from the Stage 1 development. The proposed balance tank of 20m^3 storage was sufficient to provide buffer for the wet weather flows under the 5-year Design Storm.

As noted previously, it is considered the scenario modelled is conservative, as it assumes a 5 year ARI storm in conjunction with a peak occupancy scenario, which in reality is unlikely to occur every 5 years (on average) due to the short duration of the peak occupancy reducing the likelihood on coinciding with a 5 year ARI storm. However, as the Frankton Track gravity sewer is shown to be already at or close to capacity in a 5 year ARI storm, additional flows resulting from this development will increase the risk of uncontrolled overflows.

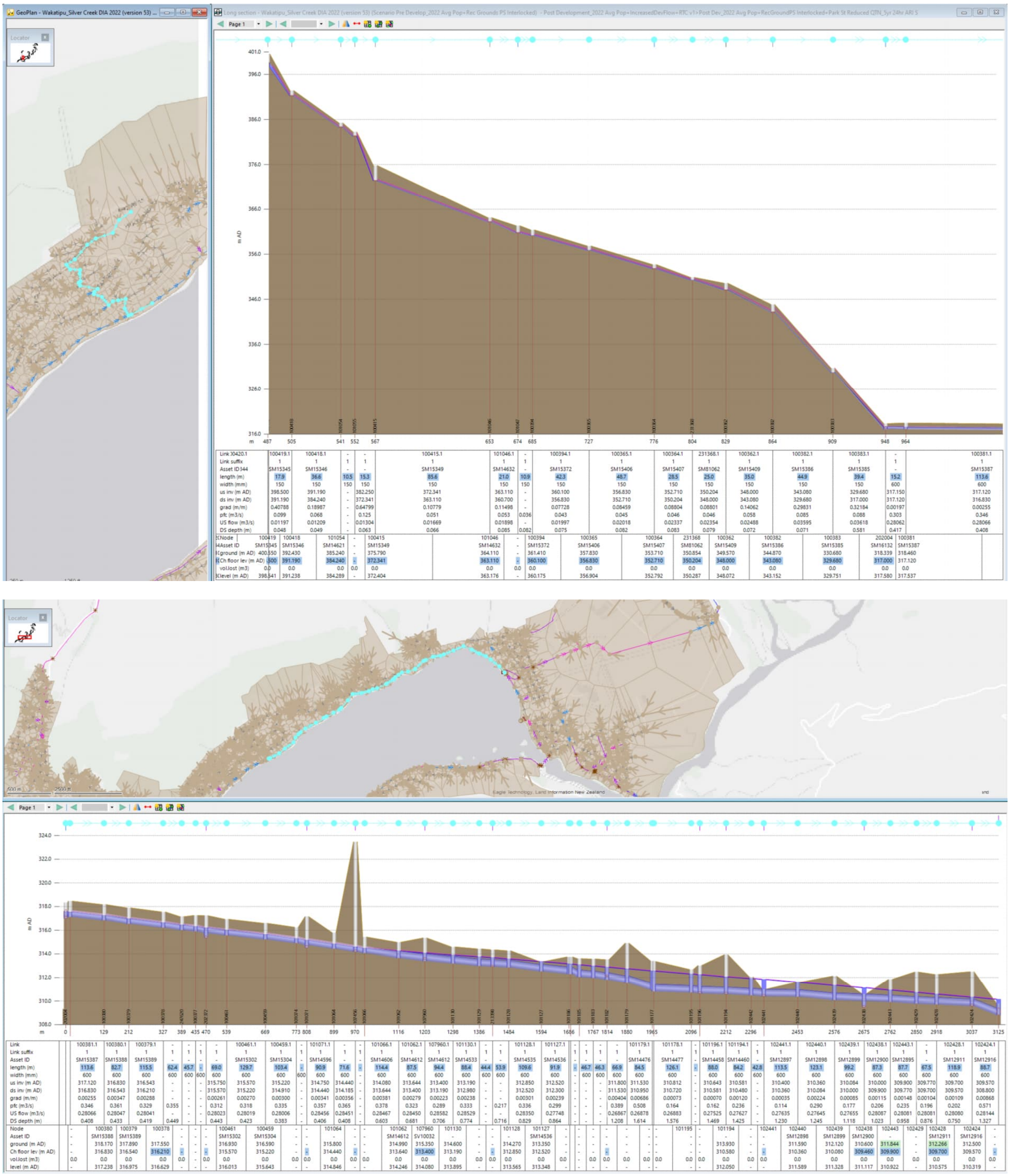

Figure 4-3 Existing (2022) Post-Development Scenario LONG-SECTION With Park Street PS existing capacity of 30l/s and without Storage at Park Street PS – 5 year ARI design storm

4.3.2 Post Development Scenario – Upgraded Park Street PS Capacity of 52l/s and 90m^3 emergency storage at Park Street PS

It was our understanding that Park Street PS is currently being upgraded to convey flows of 52l/s into the Frankton Track gravity sewer with an emergency storage of 90m^3 to be constructed.. As previously noted, both Park Street and Marine Parade PS are not interlocked, and they were given the priority over Rec Grounds PS to operate during a storm.

Therefore, this scenario was assessed to understand the performance of the network in particular the Frankton Track gravity sewer with the addition of the Stage 1 development attenuated flow and the above upgrades applied to the model.

As shown in the Figure 4-4 long-section overleaf the existing 150mm local network shows evidence of minimal increase in pipe surcharge flowing into the 600mm diameter trunk sewer, and significant surcharge in the Frankton Track gravity sewer, resulting in uncontrolled overflows predicted from manholes 101127 (~26m^3) and 102440 (<1m^3).

The additional ~20l/s of flow from Park Street PS has surcharged the Frankton Track gravity sewer further and caused the overflows to occur at these 2 locations. Furthermore, the storage at Park Street PS was barely utilised because both Park Street and Marine Parade PS have the capacity to passforward the expected peak flows contributing to this PS (see bottom figure showing the peak flows arriving at Park Street PS).

There are no other overflow incidents simulated in the downstream network as a result of the increase in attenuated flows from the Stage 1 development. The proposed balance tank of 20m^3 storage was sufficient to provide buffer for the wet weather flows under the 5-year Design Storm.

As noted previously, it is considered the scenario modelled is conservative, as it assumes a 5 year ARI storm in conjunction with a peak occupancy scenario, which in reality is unlikely to occur every 5 years (on average) due to the short duration of the peak occupancy reducing the likelihood on coinciding with a 5 year ARI storm. However, as the Frankton Track gravity sewer is already at or close to capacity in a 5 year ARI storm and can only passforward a flow of ~280 l/s before overflow occurs, the additional capacity at Park Street PS (noting that Rec Grounds PS only operated intermittently over a very short period of time when Marine Parade PS is not pumping) will increase the risk of uncontrolled overflows.

Based on the current model parameters, the operation of the 3 key pump stations (Park Street PS, Marine Parade PS and Rec Grounds PS) should be investigated further to optimise the interlocking philosophy to fully utilise the available storage in the network until such time the wider network strategy including the proposed Frankton Track rising main (which will receive flows from the proposed Recreation Grounds pump station) is implemented.

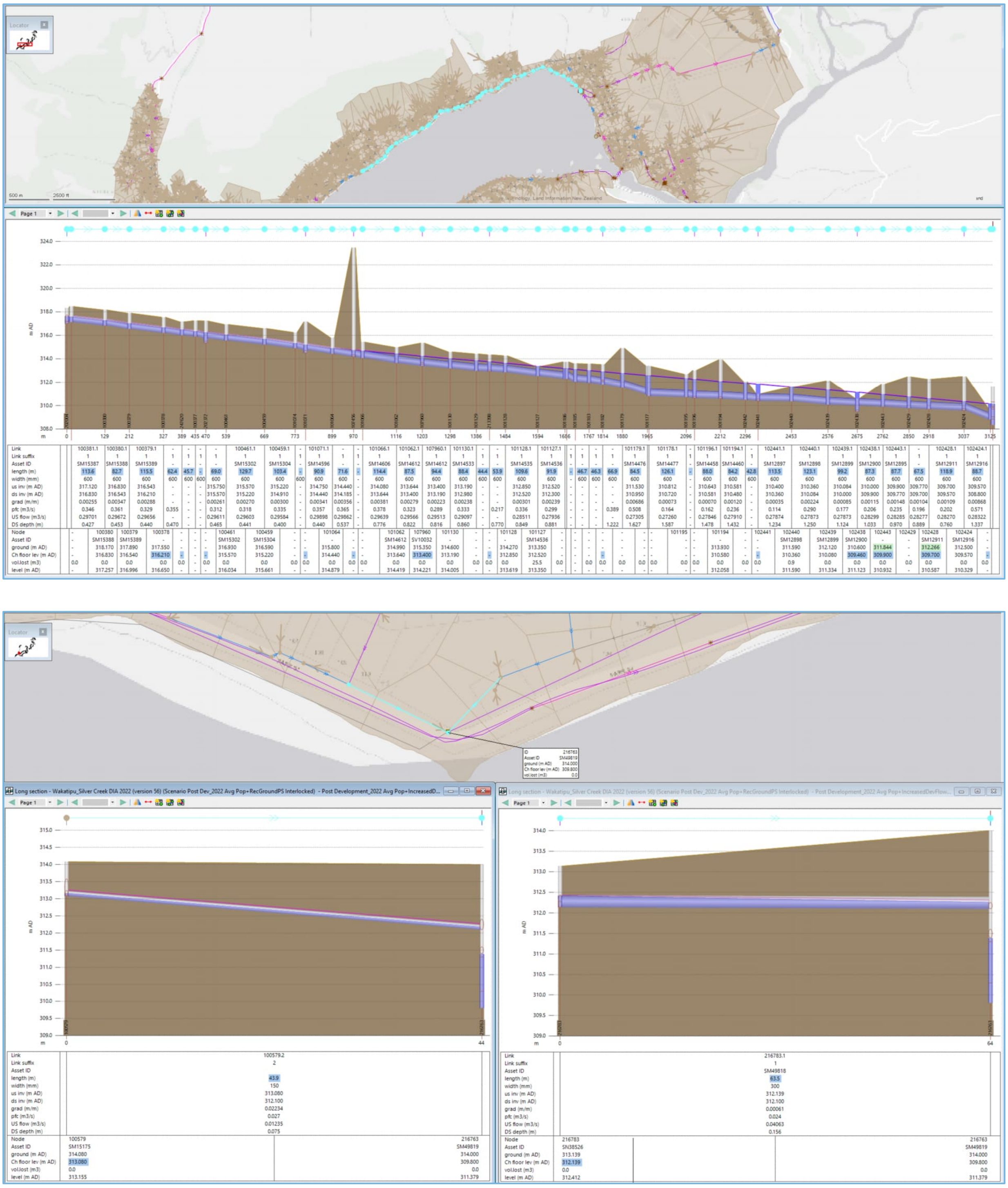

Figure 4-4 Existing (2022) Post-Development Scenario LONG-SECTION With Park Street PS conveying 52l/s and 90m³ Storage at Park Street PS – 5 year ARI design storm

5 Model Assumptions and Limitations

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

- 'Wakatipu Wastewater Model Build & Calibration Report' (Beca, August 2016)
- 'Wakatipu Wastewater Network Future System Performance Report' (Beca, August 2017)
- 'Wakatipu Wastewater Model Review & Update – High & Medium Priority Fixes Memo' (HAL, 2018)
- Wastewater Master Plan report (Morphum, 2020)
- Silver Creek Development Query Memo (HAL, 2021)

The following limitations apply to the modelling undertaken as part of this study:

- The model was originally calibrated against flows developed from field data collected in 2015 supplemented by QLDC pump station SCADA data. The 2018 model review undertaken by HAL has determined only a medium degree of confidence in the accuracy of the model. Additional flow gauging was undertaken in 2020/21 which will be used to recalibrate the model for wider network purposes.
- Interim flow gauging data from the 2020/21 flow gauging has been utilised to validate the existing model at 6 key locations relevant to this study, which affect flows at the top end of the Frankton Track gravity sewer.
- A high-level model refinement of DWF and WWF parameters has been undertaken at these gauge locations to improve the match with gauged data, but this does not represent a full model recalibration.
- Hourly rainfall data from the short term rain gauges deployed as part of the 2020/2021 flow gauging has been utilised for the validation and calibration refinement which is of lower resolution than what would normally be utilised (5 minute data) which has required the use of fast routing values to obtain the same response as the flow gauge data. It is possible that this may overestimate modelled peak flows in predominantly gravity catchments when used with higher resolution rainfall data such as the 5 year design storm used for this assessment.
- Modelled network asset data for manholes and pipes is generally as provided in the BECA calibration model, and its origin is not clear. Manhole and pipe level data has not been validated against QLDC's GIS, as-builts or survey data as part of this assessment, or as part of the HAL model review/update. Where potential network constraints are identified, it is recommended asset data in these areas is confirmed through manhole survey.
- No specific allowance for population growth was included in this study except for QT15 subtract catchment where the population was distributed evenly using the 2022 projected population as provided by QLDC. For the previously validated catchments upstream of Marine Parade and Park Street PS, additional DWF flows required to improve the DWF calibration at the selected gauges as part of this study have been added as 'additional foul flows' and hence can't be easily reconciled against actual populations, but are considered to provide a good representation of DWF during the flow survey period.
- This assessment excludes information on any additional recently consented neighbouring developments in the contributing catchment.
- This assessment focuses on the wastewater network downstream of the site, and does not consider sizing of infrastructure within the proposed site to service future development upstream of the site.
- It has been assumed that no existing overarching structure plan has been developed by QLDC for servicing this area.

- The RTC rules applied to the Frankton Beach pump stations were developed as part of the '2019 Interim System Performance' model and based on the best understanding of QLDC staff at that time.
- This study isn't a detailed option assessment and focuses on the impact the attenuated flows from the proposed Stage 1 Silver Creek development has on the existing network with the additional upgrades as advised by QLDC, to manage the risk of overflows from the Frankton Track gravity sewer and are limited to interlocking of Rec Grounds PS, and additional capacity and storage at Park Street PS.

6 Conclusion

The objective of this study is to assess the impact of the proposed Silver Creek development Stage 1 attenuated flow has on the QLDC wastewater network. The existing hydraulic model (Wakatipu Wastewater Model with HAL updates, 2018) with the 2022 average day population scenario and refined against the flow gauging data (2020/2021) was used to assess the impact of the attenuated flows on the network.

The development proposes subdivision of 585 new residential dwelling lots with a mixture of 3-4 bedrooms, and an assumed conservative occupancy of 3 people per dwelling. This equates to a design PWWF of 25.4 l/s. An assessment was undertaken in 2021 to understand the impact the full development has on the downstream network and the model results have shown that the Frankton Track gravity sewer which was already nearing capacity under a 5-year ARI Design Storm was predicted to spill from several low-lying manholes.

Therefore this assessment is undertaken to re-assess the impact of the Stage 1 proposed development with flows attenuated to Peak Dry Weather Flow by utilising a balance tank on site to buffer the wet weather flows during the storm as a measure to mitigate risk of overflows from the Frankton Track gravity sewer.

6.1 PRE-DEVELOPMENT SCENARIO

In the pre-development scenario, the existing downstream network shows evidence of some pipe surcharge near the base of the catchment flowing into the 600mm diameter trunk sewer and significant surcharge in the bottom section of the Frankton Track gravity sewer almost to a point of overflowing at manhole ID 101127. However, no overflows were predicted under this scenario.

Whilst the existing Frankton Track gravity sewer is known to be an existing constraint, it should be noted that there has been no records of reported wet weather overflows at this location. It is considered the scenario modelled is conservative because the simulated peak flows at the QT15 flow gauge location (in Frankton Track gravity sewer) was higher than the observed flows during the validation process and also it assumes a 5 year ARI storm in conjunction with a peak occupancy scenario, which in reality is unlikely to occur every 5 years (on average) due to the short duration of the peak occupancy reducing the likelihood on coinciding with a 5 year ARI storm.

6.2 POST-DEVELOPMENT SCENARIO – STAGE 1 ATTENUATED FLOWS

Two post development scenarios were simulated in the model, one with Park Street's capacity remain unchanged (30 l/s) and no emergency storage was allowed for at Park Street PS and the other scenario includes the current upgrade at Park Street PS where the capacity was increased to 52 l/s and an additional emergency storage of 90m^3 was allowed for at this PS. Note that in both post development scenarios, Rec Grounds PS was interlocked to give priority to Marine Parade PS and Park Street PS to operate during a storm.

In the first post-development scenario, with the inclusion of the Silver Creek Stage 1 attenuated flow of 3.5 l/s and a 20m^3 balance tank to buffer wet weather flows during the storm, there is evidence of slight increased pipe surcharge in the local 150mm dia sewer flowing to the trunk sewer and minimal increase in pipe surcharge in the Frankton Track gravity sewer. However, no overflows were predicted downstream of this proposed development under this scenario.

In the second post-development scenario, it was evident from the analysis that the additional flow from the upgraded Park Street PS (an increase of ~20l/s) surcharged the Frankton Track gravity sewer further resulting in uncontrolled overflows predicted from manholes 1011027 (~26m^3) and 102440 (<1m^3).

As the Frankton Track gravity sewer is already at or close to capacity in a 5 year ARI storm, while the Stage 1 attenuated flow from the proposed Silver Creek development did not result in any overflows from the downstream network, the increased capacity at Park Street PS will increase the risk of uncontrolled overflows from the existing Frankton Track gravity sewer.

It is understood that the proposed Frankton Track Rising main won't be constructed for at least 3 years and that Park Street upgrade is almost complete, and so the operation of the 3 key pump stations will need to be carefully considered (this could potentially be inhibiting Park Street PS or reducing flows at Park Street in the mean-time) while fully utilising the available storage in the network to minimise the risk of overflows from Frankton Track gravity sewer.

The developer has proposed the implementation of a balance tank on site to attenuate the flows to ~3.5 l/s to enable wastewater flows to be better controlled, and it was evident from the above assessment that this proposed short-term mitigation measure has minimal impact on downstream network constraints. While QLDC does not consider this as a desirable long-term solution, this may be a suitable short-term solution until such time the Frankton Track rising main (to divert flows from Rec Grounds PS) and the RTC is optimised.

7 RECOMMENDATIONS

The following recommendations are made in relation to this proposed development:

- QLDC should investigate options to optimise the operation of the 3 key pump stations (Marine Parade PS, Park Street PS and Rec Grounds PS) to reduce peak flows and mitigate the risk of overflows from the existing Frankton Track gravity sewer.
- Confirm timing of the proposed Stage 2 development in conjunction with expected timing of proposed Frankton Track Rising main to ensure risk of overflows is minimised.
- To consider, as part of the design of the proposed Frankton Track Rising Main, the latent capacity in the existing Frankton Track gravity sewer to service flows from the existing and future development.
- To progress the recalibration of the model to give better representation of the flows in the existing network to understand the impact of the future development has on the existing network and if required, validate the previously developed wider network servicing strategy.

SILVER CREEK DEVELOPMENT – STAGE 1
MEMO: PROPOSED ATTENUATION

To:	Brandon Ducharme	Queenstown Lakes District Council (QLDC)
	Richard Powell	Queenstown Lakes District Council (QLDC)
Distribution:	Michelle Mak	HAL

From:	Brian Robinson (HAL)
Subject:	Silver Creek Development – Stage 1: Proposed Attenuation
Date:	31st January 2023

1 Background

HAL was engaged by QLDC in July 2021 to assess the impact on the wastewater network of a proposed 585 lot residential development (Stage 1 = 150 lots) on Goldfield Heights Road in Queenstown. The assessment concluded that based on the current hydraulic model, overflows from a low lying manhole on the Frankton track gravity sewer were predicted to occur in the pre development scenario in a 1 in 5 year design storm, which would be exacerbated by additional flows from the proposed development.

Key conclusions and recommendations from this study were:

- Wastewater flows from Stage 1 of the proposed development (indicatively 150 lots) were predicted to have a relatively minor impact on the performance of the downstream wastewater network, increasing predicted overflows from a low lying manhole on the Frankton Track gravity sewer, however was acknowledged that modelled flows in the Frankton Track gravity sewer were likely to be conservative.
- Wastewater flows from future stages of the development (Stage 2 and beyond) will have a bigger impact on the risk of overflows, and shouldn't be accepted until QLDC have built the proposed extension of the rec grounds rising main to the Frankton Beach pump station, which will free up capacity in the Frankton track gravity sewer.
- Flows from Stage 1 of the development should be attenuated to approximately peak dry weather flow (~ 3.3 l/s) in the short term to minimise the impact on the risk of overflows from the Frankton Track gravity sewer
- Additional manhole survey was recommended to confirm lid levels (and better quantify the risk of overflows) for a number of low lying manholes on the Frankton track gravity sewer.

However it is believed that the existing model is over predicting wet weather flows in the Frankton track gravity sewer, as evidenced by over estimation of flows compared to those measured during a number of significant storms captured during the recent 2020/21 flow survey commissioned by QLDC, and due to the fact that uncontrolled overflows haven't been reported on the Frankton Track gravity sewer, including during a number of recent large rainfall events.

2 2023 Assessment

It should be noted that the existing hydraulic model of the Queenstown wastewater network is currently being updated and recalibrated against the 2020/21 flow survey to improve confidence in model outputs, but this project wasn't completed at the time of this study. However, the developer requires recommendations on proposed balance tank volumes and allowable discharge before those model updates will be available. In parallel to this memo, some high level model refinements are being undertaken in advance of the full model update and recalibration to improve the match of modelled flows with gauged flows in the Frankton track gravity sewer, but this work is ongoing at this time.

It should be noted that the maximum gauged flow in the Frankton track gravity sewer during the flow survey period (which included 2 rainfall events of approximately 2 year ARI) was approximately 200 l/s, significantly less than the theoretical capacity of approximately 270-280 l/s, suggesting sufficient spare capacity for the proposed attenuated flow of 3 l/s (~ 1 % of total sewer capacity) from Stage 1 of the proposed development, and a relatively low risk of causing overflows from the Frankton track gravity sewer.

3 Proposed Attenuation

Due to the potential constraint and the uncertainty associated with existing flows in the Frankton Track gravity sewer, to minimise the impact of the additional flow, it is recommended that a balance tank is provided with an orifice restricting discharge from Stage 1 of the proposed development to 3.5 l/s (approximately peak dry weather flow).

It is also recommended that the orifice is designed to minimise the risk of blockage, with a high level overflow, and to give flexibility to remove (or replace with a larger/smaller orifice plate) if necessary.

The recommended operational storage volume of the proposed balance tank is 20 m^3. Based on an assessment of the indicative wet weather flows from Stage 1 of the development in a 1 in 5 year design storm, as shown below.

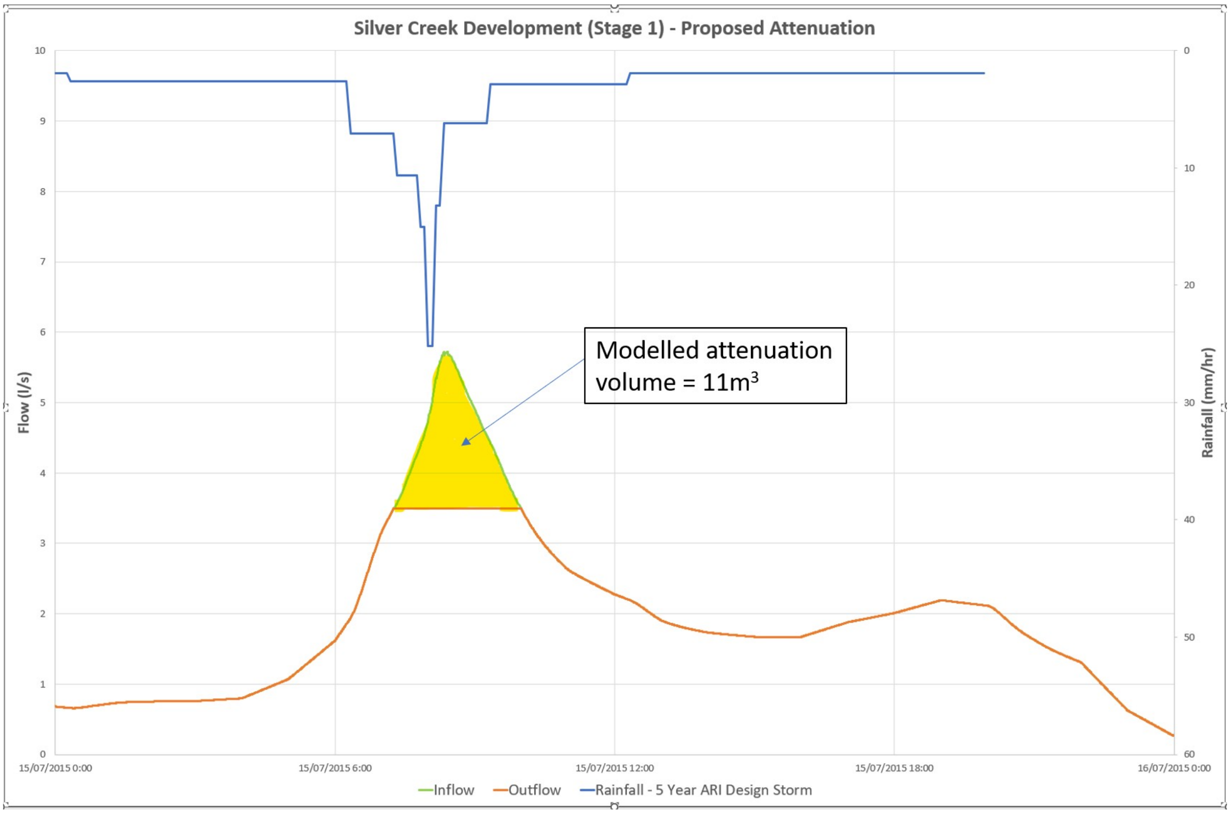

Figure 1- 1: Proposed attenuation

The following assumptions apply to this assessment:

- Stage 1 – Number of Dwellings = 150
- Stage 1 – Population = 450
- Per Capita Flow = 250 l/person/day
- Average Dry Weather Flow = 1.3 l/s
- Peak Dry Weather Flow = 3.3 l/s
- Peak wet Weather Flow (unattenuated) = 6.5 l/s
- Maximum discharge (attenuated) = 3.5 l/s
- Modelled storage volume required = 11 m^3
- Storage Tank safety factor = 25%
- Recommended operational storage volume for balance tank = 14 m^3

Appendix D. Watershed Engineering Ltd's Potable Water Reports

TECHNICAL MEMO

Water Supply Network Modelling

Queenstown – Silver Creek Development Assessment

Prepared for Queenstown lakes District Council
Prepared by: Watershed Engineering Limited

20 February 2023

1 INTRODUCTION

Watershed Engineering Limited (WSE) have been engaged to undertake hydraulic modelling to assess the impacts of the proposed Silver Creek development on the existing Queenstown water supply network and to ensure the proposed infrastructure achieves the levels of service required by Queenstown Lakes District Council (QLDC).

The Silver Creek development proposes up to 585 new residential dwellings, with 3-4 bedrooms per dwelling. The development is to be delivered in two stages, with Stage 1 delivering 150 units, followed by Stage 2 that will deliver an additional 435 Units.

A site Location Plan is provided below in Figure 1. The 33.71Ha site is legally identified as Lot 2 DP 409336 as held in the Record of Title 434296.

Figure 1 Silver Creek Development Location

2 PROPOSED RETICULATION

Mooreliving have provided the following plans, shown in Figure 2 and Figure 3.

The development is separated into 16 different lots with staging shown in Figure 4.

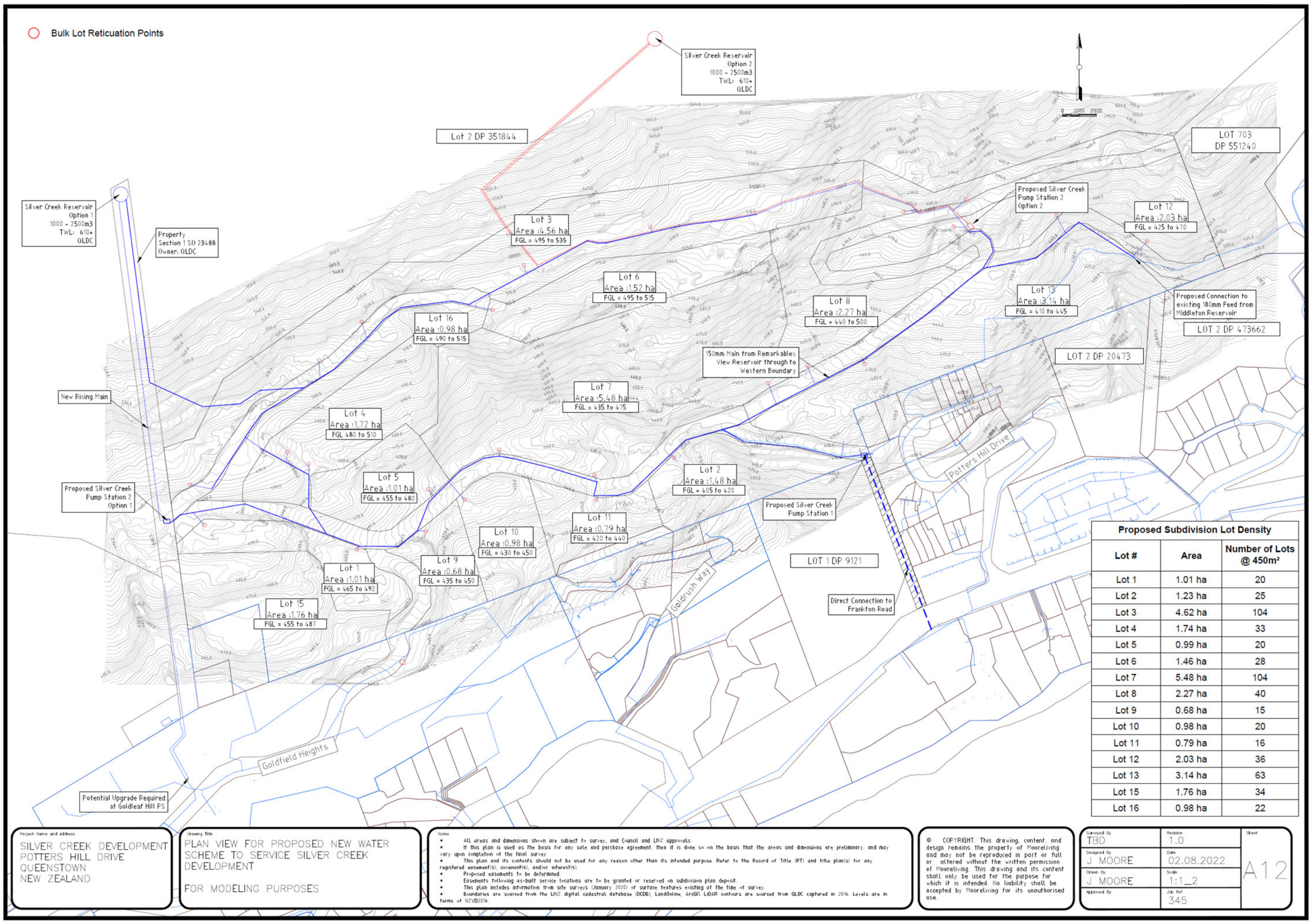

Figure 2 Silver Creek Reticulation Plan

The development area experiences a significant change in elevation as it rises up Queenstown Hill. Servicing will require different pressure zones to meet levels of service. Figure 3 overleaf shows the servicing concepts proposed by Mooreliving. This indicates three separate zones, Lower Silver Creek 1, Lower Silver Creek 2 and Upper Silver Creek.

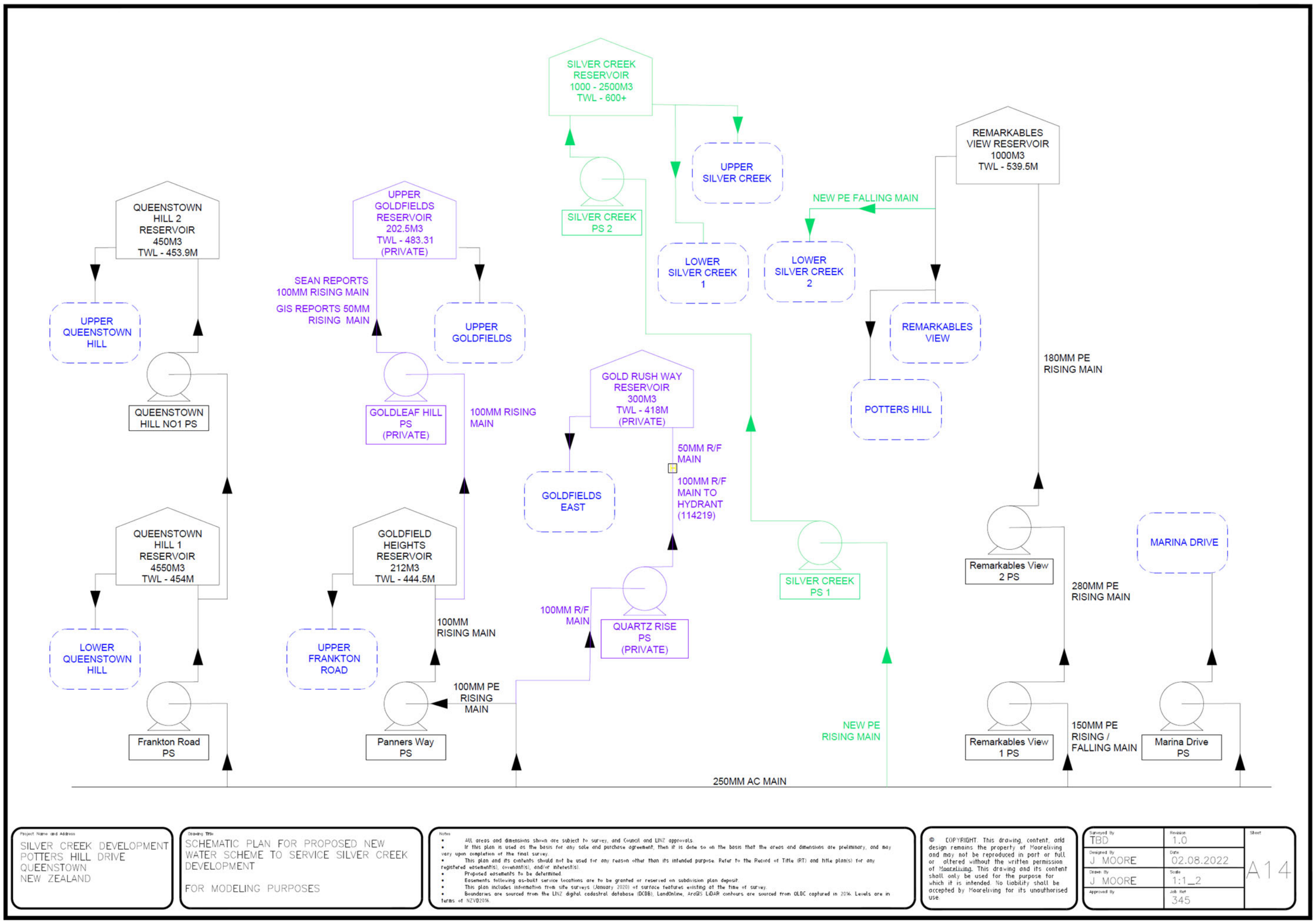

Figure 3 Silver Creek Conceptual Servicing

Lower Silver Creek 2 is to be supplied by the existing Middleton Road Reservoir (Incorrectly labelled in Figure 3 as Remarkables View Reservoir).

Upper Silver Creek and Lower Silver Creek 1 are to be supplied from a new reservoir, with Lower Silver Creek 1 pressure reduced. The proposed connection point to feed up to the new reservoir is from the existing 250mm Asbestos Cement (AC) Watermain on Frankton Road.

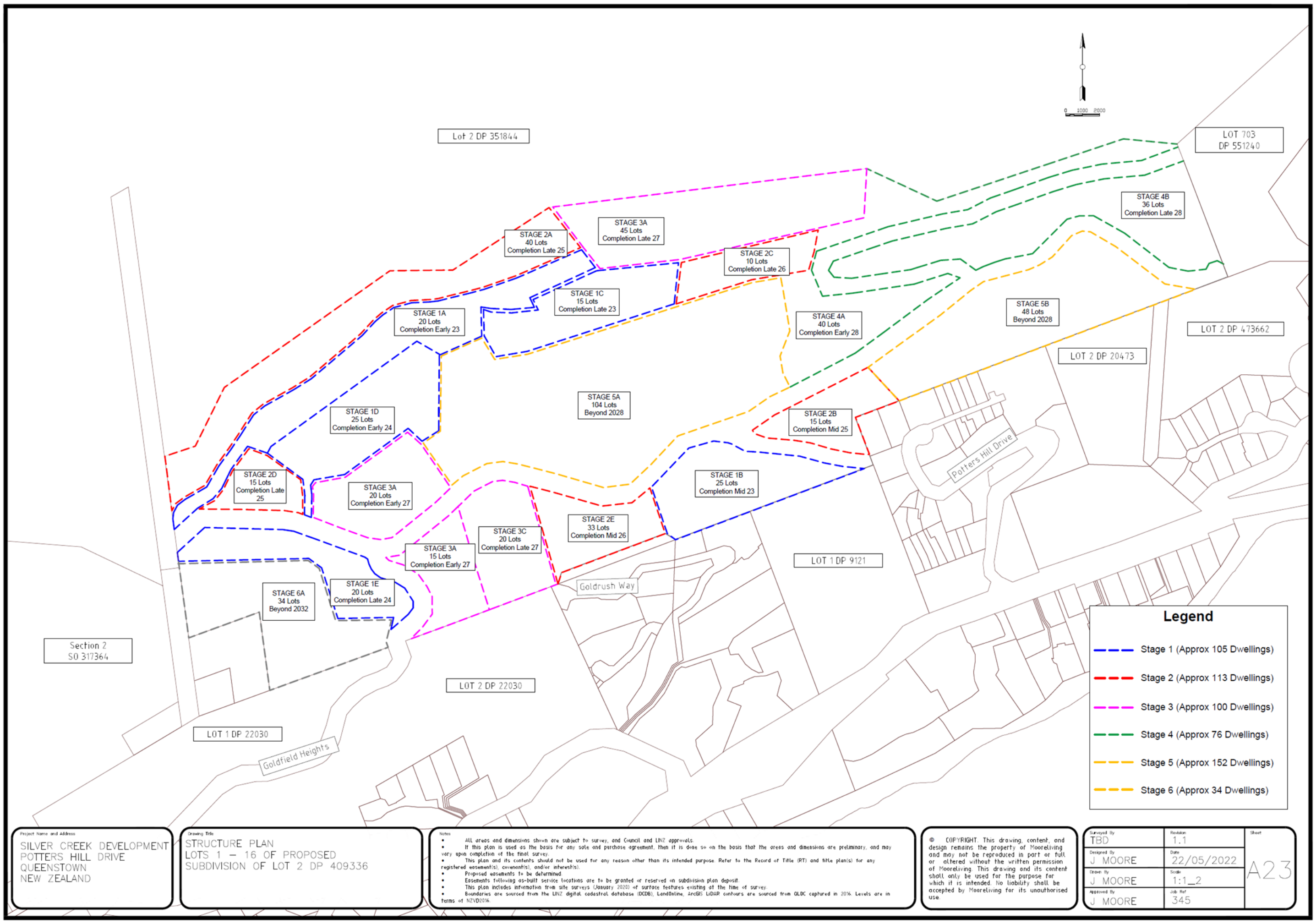

Figure 4 Proposed Staging

3 EXISTING SYSTEM PERFORMANCE

The performance of the existing network in the vicinity of the proposed Silver Creek Development has been assessed with the intention of providing a baseline for the analysis of the proposed subdivision and to identify if there are any existing system performance issues.

The existing network relates to the capacity in the trunk main on Frankton Road and the Middleton Road (Remarkables View) Scheme.

The Frankton Road watermain is part of the Wakatipu Supply Zone, supplied from the Two-Mile water intake and Fernhill #1 Reservoir. The watermain also supplies into the Frankton Water Supply Zone. A pressure sustaining valve on Frankton Road enables occasional flow back into Wakatipu from Frankton, which is also supplied from the

Kelvin Heights water intake and Kelvin Heights Reservoir. The Middleton Road Scheme is discussed below in Section 3.2.

3.1.1 Levels of Service

The levels of service historically agreed upon with QLDC for the current system performance assessment (as part of the previous model development and calibration projects) are outlined below:

- The minimum service pressure is 200-300kpa,
- The maximum service pressure is 700-800kpa,
- Reservoir Storage should provide for 12 hours Peak Day demand or 24 hours Average Day demand,
- Where pumping to a reservoir, a target maximum pump run time of 18hrs was defined for the maximum forecast peak day.

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

Queenstown Lakes District Council does not prescribe any level of service criteria relating to pipe head loss, generally speaking pipe head loss per unit length for new pipes should ideally be < 3 m/km, or 3 - 5 m/km for existing pipes during normal operation.

The values for the system performance assessment vary slight to the QLDC Land Development and Subdivision Code of Practice (2020) which is focused on system design.

- The design pressure shall be between 300 kPa and 900 kPa (30 m to 90 m).
- The head loss through pipes and fittings at the design flow rate shall be less than 5 m/km for DN ≤150; 3 m/km for DN ≥200.
- Pipelines shall be designed for flow velocities within the range of 0.5 to 2.0 m/s. In special circumstances, velocities of up to 3.0 m/s may be acceptable.
- Fire flows as specified in SNZ PAS 4509.

The project brief for the Silver Creek Development has also asked for assessment of specific assets which are included in the following assessment.

3.2 MIDDLETON ROAD SCHEME

The Middleton Road water supply scheme (also known as the Remarkables View) comprises of two pump stations, a buffer storage tank and a storage reservoir.

The Middleton Road No. 1 Pump Station draws water from the trunk main on Frankton Road and operates to fill a 25m^3 tank on Florence Close. This tank is referred to a Remarkables View Buffer Tank. The Middleton Road No. 2 Pump Station draws water from the buffer tank and operates to fill the 1000m^3 reservoir on Middleton Road.

3.2.1 Middleton Road Pump Station No.1

The Middleton Road No. 1 Pump station contains four ASP Mono pumps (Model ASP 620 5.5kW), three of which were installed originally (2006) with a fourth pump installed in 2017. At commissioning, the 3 original pumps had a recorded duty of 4.5L/s.

The elevation of the pump station is approximately RL359m pumping to the Remarkables View Tank at an approximate elevation of RL453.9m.The modelled suction pressure to the pump station ranges from 32-44m (pumps off). The model appears to be uncalibrated for the Middleton Road Scheme where neither pump station operated during the field test period.

QLDC requested Veolia to undertake testing of the pumping station in January 2023 which confirmed the capacity as follows:

- 1 Pump operating 4.4 L/s
- 2 Pumps operating 8.9 L/s
- 3 Pumps operating 12.9 L/s
- 4 Pumps operating 17 L/s.

Low pressure on suction side was noted with four pumps in operation, suggesting the capacity should be limited to three pumps operating at 12.9 L/s.

3.2.2 Remarkables View Buffer Tank (Florence Close)

The Remarkables View Tank is 25m^3 at an approximate elevation of RL453.9m. The Operation & Maintenance manual indicates the purpose of the tank is to provide buffer storage for Middleton Road No. 2 Pump Station and is not intended to provide any static storage.

3.2.3 Middelton Road Pump Station No.2 (Florence Close)

The Middleton Road No. 2 Pump station contains a Grundfos Pump set comprising of three Hydro MPC-E-3 CRE 32-4-2 pumps. The Operation and Maintenance manual suggests each pump is capable of delivering approximately 15L/s against a static head of approximately 70m. The pump set can produce a maximum flowrate of 40L/s. The pumps are controlled with variable speed drives. Information from the Grundfos website suggests the capacity can be limited to reasonably low flows.

Field testing of this pump station has not been carried out.

The model has been updated, with a single pump station link and three pumps modelled based on the individual pump curves. The pump station operated to maintain the level in the Middleton Road reservoir between 75% and 95% full.

3.2.4 Middleton Road Reservoir

The Middleton Road Reservoir has a gross volume of 1000m^3, a Top water level (TWL) of 540.47m, an elevation of 535.47m with a corresponding depth of 5m.

The reservoir can be filled from the Middleton Road No. 2 pump station; however it is noted the full capacity of this pumping station is substantially greater than the capacity of the Middleton Road No. 1 pump station and the buffer tank of 25m^3 is reasonably small when considering filling a 1000m^3 reservoir.

The reservoir currently services a small demand area. The previous system performance assessment report produced by Mott MacDonald (Queenstown Water Supply Model Partial model update and system performance, June 2022) notes Middleton Road Reservoir has excessively high storage hours available and a high number of turnover days (low turnover).

The Operation and Maintenance manual references a historical modelling analysis undertaken by Tonkin & Taylor suggesting the “Remarkables View” network was sized for 555 dwellings. The area is still under development.

3.2.5 Middleton Road Scheme Performance Issues

As noted above the capacity of the Middleton Road No. 2 pump station is substantially greater than the Middleton Road No. 1 pump station, and the buffer storage is limited to 25m^3. However, Middleton Road No. 1 pump station has been confirmed as able to produce 12.9 L/s with 3 pumps running, and the fourth pump on standby bring the total capacity to 17.9L/s if the upstream network can provide this.

It is noted the scheme is uncalibrated in the model, and the full control regime and pumping station flowrates should be better understood to ensure the 25m^3 tank is not drained.

The Middleton Road reservoir is noted as having good capacity to supply a wider number of customers.

3.3 WATERMAIN CAPACITIES

Watermain capacities have been assessed against the design standards set out in the QLDC Land Development and Subdivision Code of Practice for Subdivision and Development (2020).

- The head loss through pipes and fittings at the design flow rate shall be less than 5 m/km for DN ≤150; 3 m/km for DN ≥200.
- Pipelines shall be designed for flow velocities within the range of 0.5 to 2.0 m/s. In special circumstances, velocities of up to 3.0 m/s may be acceptable.

It should be noted that while the design criteria are helpful in understanding the pipeline capacities, typically, with an existing network pipes would not be replaced solely on headloss, unless it were a main contributing factor to system performance issues such as the ability to deliver minimum pressures.

3.3.1 Middleton Road Pump Station No.1 Rising Main

The Middleton Road Pump Station No.1 Rising Main is a 280mm PE100 watermain PN16 resulting in an internal diameter of ID 227.8mm. To meet the level of service criteria of < 3m/km the maximum flow rate or capacity would be 36L/s. Typically for a dedicated rising main a higher head loss may be acceptable depending on the system curve and pumping cost efficiency. Keeping head loss at <5m/km would allow a flow rate of approximately 48L/s.

Maximum pressures within the watermain occur at the lowest elevation (pump station discharge) and are approximately 95m, which is within the pressure rating of the pipeline.

3.3.2 Middleton Road Pump Station No.2 Rising Main

The Middleton Road Pump Station No.2 Rising Main is a 250mm PE100 watermain PN16. This would result in an ID 203.4mm pipeline. To meet the level of service criteria of < 3m/km the maximum flow rate or capacity would be 27L/s. Typically for a dedicated rising main a higher head loss may be acceptable depending on the system curve and pumping cost efficiency. Keeping head loss at <5m/km would allow a flow rate of approximately 35L/s.

Maximum pressures within the watermain occur at the lowest elevation (pump station discharge) and are approximately 86m, which is within the pressure rating of the pipeline.

3.3.3 Middleton Road Falling Main

There is a 180mm PE100 watermain leaving the Middleton Road Reservoir to supply domestic customers in the Middleton Road (Remarkables View) scheme.

Assuming a 180mm PE100 PN 12.5 and therefore ID 152.8mm pipeline, to meet the level of service criteria of < 3m/km the flow rate or capacity would be 12.5L/s.

The existing modelled flowrate under the current peak day scenario is 0.99 L/s. Pressures within the line are governed by the hydraulic grade of the Middleton Road Reservoir. At the lowest elevation the pressure is 86m.

3.3.4 Watermain Supplying Woods Lane

There is a 180mm PE100 watermain supplying Woods Lane from the falling main of the Middleton Road Reservoir to supply domestic customers. Assuming a 180mm PE100 PN 12.5 and therefore ID 152.8mm pipeline, to meet the level of service criteria of < 3m/km the flow rate or capacity would be 12.5L/s.

3.3.5 150mm UPVC Watermain on Middleton Road

Assuming a 150mm UPVC watermain with an assumed ID 150mm pipeline, to meet the level of service criteria of < 3m/km the flow rate or capacity would be 12L/s. The maximum flowrate in the current peak day scenario is 6L/s, based on the modelled operation of the Middleton Road No.1 Pump Station. The maximum pressure in the line is 75m.

3.3.6 250mm AC Watermain on Frankton Road

The 250mm AC watermain, based on an assumed internal diameter of 250mm, to meet the level of service criteria of < 3m/km the flow rate or capacity would be 47L/s. The maximum flowrate in the current peak day scenario is 21L/s. The maximum pressure in the line at the proposed connection point (Node ID MOD_V_3464567) is 63m, the minimum pressure is 51m. It should be noted the highest elevation point on the Frankton Road watermain is on Andrews Road, where the model indicates the existing minimum pressure is as low as 7m. It is recommended pressures in the watermain in this vicinity are verified in the field. Table 1 provides a summary of the pipeline capacities.

Table 1 Summary of Specific Existing Pipeline Capacities

Asset	GIS Comp Key	Current Peak Day Flow (L/s) [1]	Max Flow for Headloss < 3m/km	Maximum Pressure	Minimum Pressure
280mm PE100 watermain supplying the Remarkables View Tank	301917	6 L/s	36 L/s	95m	2.8m (By tank)
250mm PE100 watermain supplying the Middleton Road Reservoir	301848	10 L/s	27 L/s	86m	3.75m (By reservoir)
180mm PE100 watermain leaving the Middleton Road Reservoir	301852	0.99 L/s	12.5 L/s	86m	3.75m (By reservoir)
180mm PE100 Watermain supplying Woods Lane	269278	0.92 L/s	12.5 L/s	60m	17m
150mm UPVC Watermain on Middleton Road	147792	6 L/s	12 L/s	75m	31m
250mm AC Watermain on Frankton Road	146754	21 L/s	47 L/s	63m	51m

[1] This area of the model appears to be uncalibrated and therefore the existing flows could be different.

3.4 MAXIMUM AND MINIMUM PRESSURES

Minimum pressures within the study area are greater than 30m or 300kPa, with the exception of some nodes immediately adjacent to the reservoirs. Typically, these do not represent actual customers points of supply and no demand is assigned on these nodes.

There are areas within the Middleton Road scheme where the maximum pressures are in excess of 90m or 900kPa.

3.5 FIREFLOW ASSESSMENT

An assessment of the available hydrant flow for hydrants in the study area, specifically the Middleton Road scheme has been undertaken. The available flow for all hydrants is in excess of 50L/s, and up to 100L/s as a result of high pressures.

3.6 EXISTING SYSTEM PERFORMANCE SUMMARY

The capacity of the system is governed by the ability to fill the Middleton Road Reservoir from the two pump stations without compromising the Remarkables View Buffer Tank. As noted above the capacity of the Middleton Road No. 1 pump station has been confirmed as able to produce 12.9 L/s with 3 pumps running, and the fourth pump on standby.

The Middleton Road Reservoir is currently oversized for the area it supplies and has very low turnover.

There are no minimum pressure issues, some high pressures in excess of 900kPa, and available fire flows greater than 50L/s.

4 SILVER CREEK DEVELOPMENT DEMAND ASSESSMENT

The demand has been assessed based on information provided by QLDC as referenced below in Table 2. Staging of the development has been provided by Mooreliving and the demand assessment by Staging lot shown in Table 3. *It is noted there is a 5-unit difference in the total development units, however this in not likely to have any significant impacts on the assessment*.

Figure 2 shows the Lot numbers and Figure 4 indicates proposed staging.

For the purposes of this assessment QLDC has adopted the following key design parameters from the QLDC Land Development and Subdivision Code of Practice (2020):

- Daily consumption of 700 L/p/day
- Occupancy per Residence = 3
- Peak hour factor of up to 4.0 (Queenstown), 6.6 (Rest of District);
- Firefighting demands as specified in SNZ PAS 4509.
- The network should be designed to maintain appropriate nominated pressures for both peak demand (average daily demand in L/s x peak hour factor) and firefighting demand scenarios. These figures should be applied to mains of 100 mm diameter or greater. Mains less than 100 mm in diameter can be sized using the multiple dwellings provisions of AS/NZS 3500.1.

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

Table 2 Average and Peak Day Demand Calculations

Development	Number of Residential Lots	Population	Average Demand (L/s)	Peak Hour Demand (L/s)	Peak Day Demand (L/s)[1]
Stage 1	150	450	3.65	14.58	6.92
Stage 2	435	1305	10.57	42.29	20.07
Total Additional Lots	585	1755	14.22	56.88	26.99

[1] Peak Day Demand has been calculated based on applying a standard domestic equivalent profile with a peak hour factor of 2.1, and the specified total peak hour factor of 4. This results in an average to peak day factor of 1.9.

Table 3 Average and Peak Day Demand Calculations by Lot

Development	Area (Ha)	Ground Level Range (RL m)	Number of Residential Lots	Population	Average Demand (L/s)	Peak Hour Demand (L/s)	Peak Day Demand (L/s)[1]
Lot 1	1.01	465-492	20	60	0.49	1.94	0.92
Lot 2	1.23	405-420	25	75	0.61	2.43	1.15
Lot 3	4.62	495-535	104	312	2.53	10.11	4.80
Lot 4	1.74	480-510	33	99	0.80	3.21	1.52
Lot 5	0.99	455-480	20	60	0.49	1.94	0.92
Lot 6	1.46	495-515	28	84	0.68	2.72	1.29
Lot 7	5.48	435-475	104	312	2.53	10.11	4.80
Lot 8	2.27	440-500	40	120	0.97	3.89	1.85
Lot 9	0.68	435-450	15	45	0.36	1.46	0.69
Lot 10	0.98	430-450	20	60	0.49	1.94	0.92
Lot 11	0.79	420-440	16	48	0.39	1.56	0.74
Lot 12	2.03	425-470	36	108	0.88	3.50	1.66
Lot 13	3.14	410-445	63	189	1.53	6.13	2.91
Lot 14	-	-			-	-	-
Lot 15	1.76	455-487	34	102	0.83	3.31	1.57
Lot 16	0.98	490-515	22	66	0.53	2.14	1.02
Total	-	-	580	1740	14.10	56.39	26.76

[1] Peak Day Demand has been calculated based on applying a standard domestic equivalent profile with a peak hour factor of 2.1, and the specified total peak hour factor of 4. This results in an average to peak day factor of 1.9.

5 IMPACTS OF PROPOSED DEVELOPMENT ON EXISTING NETWORK

The impacts of the proposed Silver Creek Development have been considered in this section in simplified terms. Demands of the full development have been calculated and assigned to the proposed connection point or points on the existing network. The intention is to assess the impacts of the additional demand on the existing network and assess the system performance. This will provide good base information on the level of infrastructure required for the proposed development.

This high-level assessment considers two key flowrates, the calculated peak hour flowrate of the full development at 56.59 L/s and the calculated peak daily demand at 26.76 L/s.

If the development was connected directly to the 250mm AC Watermain on Frankton Road, the peak hour flowrate 56.59 L/s exceeds the capacity of this main. The existing minimum pressure at the point of supply is 51.4m, this would in theory fall to a minimum of just 16.5m, although pressure at the high point in the on St Andrews Road would be below zero (no water).

Considering the use of reservoir storage to mitigate peak hour flows, and pumping the daily flow rate of 26.76L/s to a reservoir over 18hrs, results in approximately 35.7L/s drawn from the network. This flowrate would still put substantial strain on the capacity of the 250mm AC watermain, with the minimum pressure falling to approximately 30m. The pressure at the high point on St Andrews Road would still be zero (no water).

If the subdivision were to be supplied in part, say half, from the Middleton Road Reservoir, the Middleton Road Pump Station No.1 would need to be upgraded. The result on the Frankton Road watermain would be the same, whereby the same peak day volume would need to be pumped to reservoirs, just via two supply points.

This high-level assessment indicates upgrades would be required outside of the infrastructure proposed for the development. This will likely include Frankton Road, and the Middleton Road No.1 Pump Station.

6 REDUCED DEMAND SCENARIO ASSESSMENT

Consideration has been given to a reduced demand scenario using an average daily consumption value of 250 L/p/day. The other design parameters remain the same as for the initial assessment.

Table 4 shows the reduced demand by stage and Table 5 shows the demands by Lot numbers.

Table 4 Reduced Demand Scenario - Average and Peak Day Demand Calculations

Development	Number of Residential Lots	Population	Average Demand (L/s)	Peak Hour Demand (L/s)	Peak Day Demand (L/s)[1]
Stage 1	150	450	1.30	5.21	2.47
Stage 2	435	1305	3.78	15.10	7.17
Total Additional Lots	585	1755	5.08	20.31	9.64

[1] Peak Day Demand has been calculated based on applying a standard domestic equivalent profile with a peak hour factor of 2.1, and the specified total peak hour factor of 4. This results in an average to peak day factor of 1.9.

Table 5 Reduced Demand Scenario - Average and Peak Day Demand Calculations by Lot

Development	Area (Ha)	Ground Level Range (RL m)	Number of Residential Lots	Population	Average Demand (L/s)	Peak Hour Demand (L/s)	Peak Day Demand (L/s)[1]
Lot 1	1.01	465-492	20	60	0.17	0.69	0.33
Lot 2	1.23	405-420	25	75	0.22	0.87	0.41
Lot 3	4.62	495-535	104	312	0.90	3.61	1.71
Lot 4	1.74	480-510	33	99	0.29	1.15	0.54

Development	Area (Ha)	Ground Level Range (RL m)	Number of Residential Lots	Population	Average Demand (L/s)	Peak Hour Demand (L/s)	Peak Day Demand (L/s)[1]
Lot 5	0.99	455-480	20	60	0.17	0.69	0.33
Lot 6	1.46	495-515	28	84	0.24	0.97	0.46
Lot 7	5.48	435-475	104	312	0.90	3.61	1.71
Lot 8	2.27	440-500	40	120	0.35	1.39	0.66
Lot 9	0.68	435-450	15	45	0.13	0.52	0.25
Lot 10	0.98	430-450	20	60	0.17	0.69	0.33
Lot 11	0.79	420-440	16	48	0.14	0.56	0.26
Lot 12	2.03	425-470	36	108	0.31	1.25	0.59
Lot 13	3.14	410-445	63	189	0.55	2.19	1.04
Lot 14	-	-			0.30	1.18	0.56
Lot 15	1.76	455-487	34	102	0.19	0.76	0.36
Lot 16	0.98	490-515	22	66	0.17	0.69	0.33
Total	-	-	580	1740	5.03	20.14	9.56

[1] Peak Day Demand has been calculated based on applying a standard domestic equivalent profile with a peak hour factor of 2.1, and the specified total peak hour factor of 4. This results in an average to peak day factor of 1.9.

6.1 IMPACTS OF PROPOSED DEVELOPMENT ON EXISTING NETWORK (REDUCED DEMAND SCANERIO)

The same approach has been taken as outlined in Section 5.

This two key flowrates considered are the calculated peak hour flowrate of the full development at 20.14 L/s and the calculated peak daily demand at 9.56 L/s.

If the development was connected directly to the 250mm AC Watermain on Frankton Road, the peak hour flowrate 20.14 L/s exceeds the capacity of this main. The existing minimum pressure at the point of supply is 51.4m, this would fall to a minimum of 41.5m. It should be noted the highest elevation point on the Frankton Road watermain is on Andrews Road, where the model indicates the existing minimum pressure is as low as 7m. Pressure here falls to near zero (no water) for the peak flowrate of 20.14L/s.

Considering the use of reservoir storage to mitigate peak hour flows, and pumping the daily flow rate of 9.56L/s to a reservoir over 18hrs, results in approximately 12.74L/s drawn from the network. This flowrate is just below the capacity of the Middleton Road No.1 Pump Station with 3 pumps operating. This flowrate reduces pressure in the 250mm AC watermain by approximately 5m, with the minimum pressure falling to approximately 46.2m. For the high point on Andrews Road, the model indicates the minimum pressure falls to 3.3m for this scenario.

7 INFRASTRUCTURE OPTIONS FOR SUPPLYING SILVER CREEK

Figure 3 above shows the servicing concepts proposed by Mooreliving. This indicates three separate zones, Lower Silver Creek 1, Lower Silver Creek 2 and Upper Silver Creek.

The reticulation plan shows some options for reservoirs and pumping station sites, but it is not specific about the breakdown of pressure zones and the lots they might service. Scheme design is not included in the scope of this project, however a scenario has been developed in the model which creates the three pressure zones. A single new pumping station has been used to supply the proposed new reservoir through a dedicated rising main. The new reservoir supplies the upper-level zone by gravity. A pressure reducing valve supplies into the lower level zone. These two zones equate to approximately two thirds of the development, or 380 Lots. The mid-zone, is supplied from the Middleton Road reservoir, approximately 200 Lots.

Figure 5 below shows the assessed servicing concept.

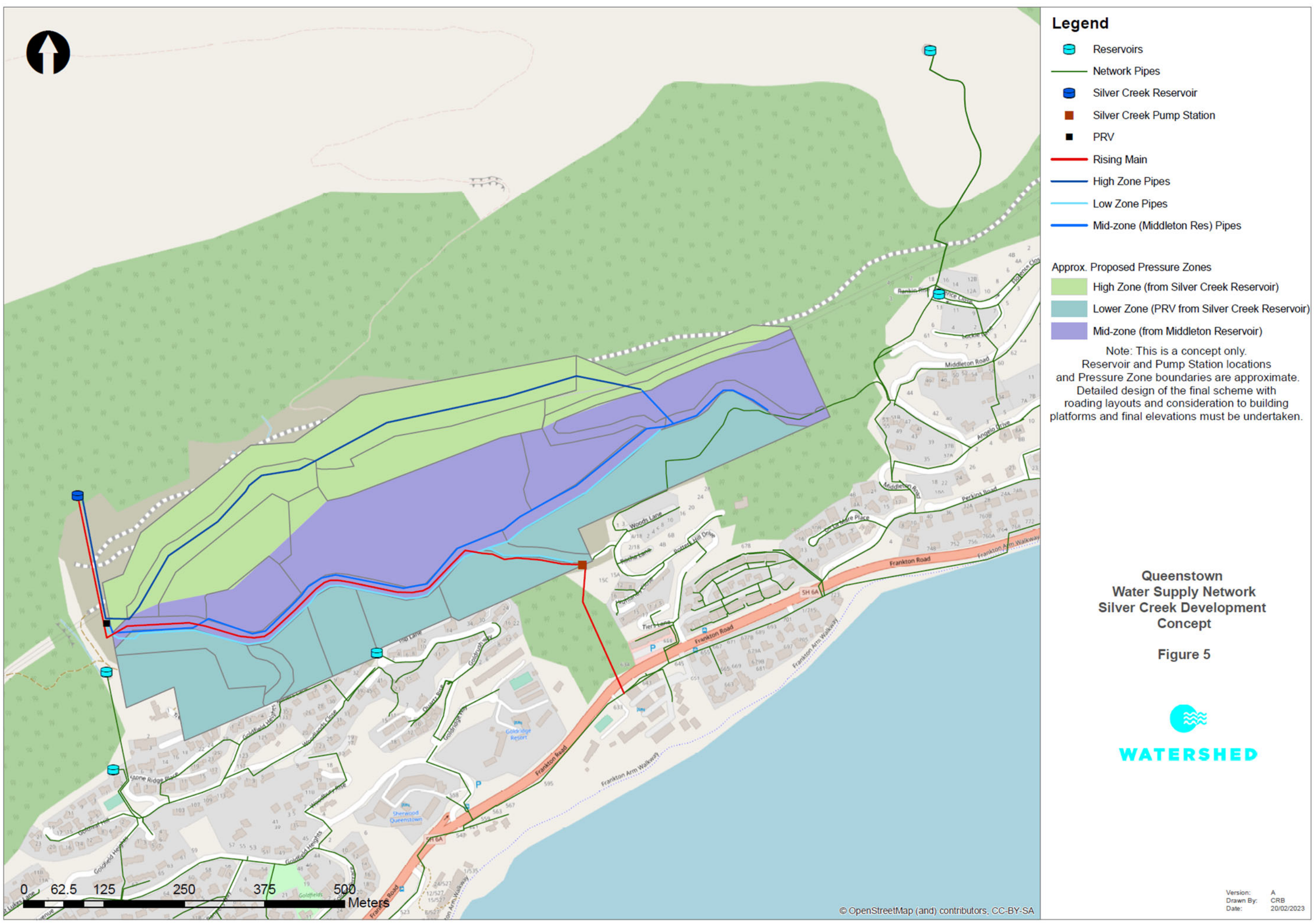

Figure 5 Sliver Creek Servicing Scenario

The new pumping station servicing the new reservoir and 380 lots would have a flow rate of around 8.35 L/s.

Middleton Road would still be required to operate to fill the Middleton Road reservoir. Based on the current demands this flowrate can be limited to 1 pump operating at 4.4L/s. However, this area is under-development and if the station operated with 3 pumps at 12.9 L/s and the new pump station operated (i.e. 21.25L/s both stations) pressures in the Frankton Watermain would be zero (no water) at the high point.

Therefore, even with reduced demands it appears the system would likely have limitations in supplying the full development.

Currently, the advice from Veolia, based on field testing, is that operating 4 pumps at Middleton Road Pump Station reduces the suction pressure and is not recommended. Therefore, adding additional load to the Frankton Watermain beyond the capacity of 3 pumps at 12.9L/s is not advisable.

The recommended development should be limited to the allocation from the existing Middleton Road scheme, unless wider network upgrades are considered.

8 SUMMARY

It is important for QLDC to ensure the best outcomes for the community and future property owners, and to ensure the delivery of safe reliable drinking water.

The Silver Creek development proposes 580- 585 new residential dwellings up steep terrain along the Frankton Arm.

Based on the design demands in the QLDC Land Development and Subdivision Code of Practice 2020 there is insufficient capacity to supply the development without upgrades to the wider network.

Further assessment was undertaken with reduced demands assuming 250L/person/day. While the subdivision could be supplied in part, there are still concerns with the capacity of the Frankton Road watermain, and with approved developments already underway. The recommended development should be limited to the allocation from the existing Middleton Road scheme, unless wider network upgrades are considered.

It is recommended further investigation of the network is carried out to confirm the capacity of the Frankton Road watermain and calibration/validation of the hydraulic model. This should include assessing pressure at the high point in the watermain, and further investigation into the possibility of operating the Frankton Road Pump Station.

The future integration of the new Shotover Country water treatment plant is also on-going, and some changes to the system may occur on Frankton Road.

The hydraulic model is a representation of the physical water supply system and as such, has some limitations which should be noted in the model development and calibration report. Watershed Engineering Limited were not involved in the original development of the Queenstown hydraulic model, nor the conversion of the EPA models to Infoworks WS pro. Updates have been made to the model by Watershed to enable it to be used for this project brief. The demands and peaking factors used to assess the development are based on assumptions and the actual final water demands may vary.

We trust this report meet your requirements. Please contact Charlotte Broadbent on s 9(2)(a)
s 9(2)(a) if you wish to discuss any aspects of this report further.

Revision	Name	Signed	Date
A – Interim Draft	Charlotte Broadbent	Charlotte Broadbent	15 November 2022
B - Final	Charlotte Broadbent	Charlotte Broadbent	20 February 2023

WATERSHED

TECHNICAL MEMO

Water Supply Network Modelling

Queenstown – Silver Creek Development Assessment – Additional Analysis

Prepared for Queenstown lakes District Council
Prepared by: Watershed Engineering Limited

17 March 2023

1 INTRODUCTION

Watershed Engineering Limited (WSE) was engaged to undertake hydraulic modelling to assess the impacts of the proposed Silver Creek development on the existing Queenstown water supply network and to ensure the proposed infrastructure achieves the levels of service required by Queenstown Lakes District Council (QLDC).

A technical memorandum of the outcomes of the assessment was produced in February 2023 and concluded, based on the design demands in the QLDC Land Development and Subdivision Code of Practice 2020, there is insufficient capacity to supply the development without upgrades to the wider network. A further assessment was undertaken with reduced demands assuming 250L/person/day. While the subdivision could be supplied in part, there are still concerns with the capacity of the Frankton Road watermain, and approved developments already underway. It was recommended the Silver Creek development should be limited to the allocation from the existing Middleton Road scheme, unless wider network upgrades were considered.

WSE have been asked to undertake further analyses to explore additional options to address servicing the full development and to assess required storage and upgrades based on residential dwelling or DUEs (Development Unit Equivalent).

The Silver Creek development proposes up to 585 new residential dwellings, with 3-4 bedrooms per dwelling. The development is to be delivered in two stages, with Stage 1 delivering 150 units, followed by Stage 2 that will deliver an additional 435 Units.

A site Location Plan is provided below in Figure 1. The 33.71Ha site is legally identified as Lot 2 DP 409336 as held in the Record of Title 434296.

This memorandum should be read in conjunction with the February 2023 memorandum.

Figure 1 Silver Creek Development Location

2 SILVER CREEK DEVELOPMENT DEMAND ASSESSMENT

For the purposes of this assessment QLDC has adopted the following key design parameters from the QLDC Land Development and Subdivision Code of Practice (2020):

- Daily consumption of 700 L/p/day
- Occupancy per Residence = 3
- Peak hour factor of up to 4.0 (Queenstown), 6.6 (Rest of District);
- Firefighting demands as specified in SNZ PAS 4509.
- The network should be designed to maintain appropriate nominated pressures for both peak demand (average daily demand in L/s x peak hour factor) and firefighting demand scenarios. These figures should be applied to mains of 100 mm diameter or greater. Mains less than 100 mm in diameter can be sized using the multiple dwellings provisions of AS/NZS 3500.1.

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

A reduced demand analysis has also been assessed at 250L/p/day, and a further scenario considering 1000L/connection per day or 333.33L/p/day.

Table 1 Average and Peak Day Demand Calculations

Stage	No. of Lots	Pop.	700 L/ p / day			250 L/p/day			1000 L/conn (333.33 L/p/day)		
			Average (L/s)	Peak Hour (L/s)	Peak Day (L/s)[1]	Average (L/s)	Peak Hour (L/s)	Peak Day (L/s)[1]	Average (L/s)	Peak Hour (L/s)	Peak Day (L/s)[1]
Stage 1	150	450	3.65	14.58	6.92	1.30	5.21	2.47	1.74	6.94	3.30
Stage 2	435	1305	10.57	42.29	20.07	3.78	15.10	7.17	5.03	20.14	9.56
Total Additional Lots	585	1755	14.22	56.88	26.99	5.08	20.31	9.64	6.77	27.08	12.85

[1] Peak Day Demand has been calculated based on applying a standard domestic equivalent profile with a peak hour factor of 2.1, and the specified total peak hour factor of 4. This results in an average to peak day factor of 1.9.

2.1.1 Levels of Service

The levels of service historically agreed upon with QLDC for the current system performance assessment (as part of the previous model development and calibration projects) are outlined below:

- The minimum service pressure is 200-300kpa,
- The maximum service pressure is 700-800kpa,
- Reservoir Storage should provide for 12 hours Peak Day demand or 24 hours Average Day demand which ever is greater,
- Where pumping to a reservoir, a target maximum pump run time of 18hrs was defined for the maximum forecast peak day.

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

Queenstown Lakes District Council does not prescribe any level of service criteria relating to pipe head loss, generally speaking pipe head loss per unit length for new pipes should ideally be < 3 m/km, or 3 - 5 m/km for existing pipes during normal operation.

3 REQUIRED STORAGE & STAGING

Reservoirs operate on the network and provide a buffer for diurnal changes in demands. Typically, when accounting for emergency storage it is assumed 20% of the volume is required for the operational volume and the bottom 5% in the reservoir is assumed unusable dead storage. Therefore, the only 75% of the gross volume is effectively emergency storage. These values may vary between reservoirs with different operating levels and philosophies, and the approach may also vary between Local Authorities. For the purposes of this study, QLDC has specified that the gross storage volume will be taken as the emergency storage volume.

Based on the demand assessment Table 2 overleaf outlines the required storage for full development under each demand scenario, and also presents storage per residential lot to enable staging to be considered.

Table 2 Silver Creek Reservoir Storage

Demand Scenario	24hr Average Demand – Full Development	Required Storage - Full Development (m^3)	Required Storage per DUE (m^3 / DUE)
700 L/p/day	14.22	1229	2.1
250 L/p/day	5.08	439	0.75
1000 L/conn/day	6.77	585	1.0

4 SUPPLY FROM FRANKTON

The Frankton Road watermain currently has a non-return valve and pressure sustaining valve on the Frankton Road Watermain which controls flows between the Whakatipu and Frankton zone. The non-return valve allows flow from Whakatipu into Frankton, while the pressure sustaining valve maintains pressures in Frankton only allowing flow back to Whakatipu if pressures allow. The current peak day model indicates limited flows either way through this location.

For this analysis, it has been proposed to move the PSV/NRV arrangement further west and enable the Middleton Road Reservoir and proposed Silver Creek Development to be supplied from the Frankton area.

A closed valve has been modelled on the Frankton Road watermain between Goldfield Heights and Potters Hill Drive.

4.1 CHANGES TO EXISTING SYSTEM PERFORMANCE WITH CLOSED VALVE

For the existing system, the model results indicate similar minimum pressures in the network under normal peak day demands to the east of the new valve (Frankton), and increased minimum pressures by approximately 5m west of the new closed valve (Whakatipu). Fireflows on Frankton Road are slightly improved both sides the new valve.

However, with the additional demand being supplied from Kelvin Heights, all three pumps are required to operate to keep the Kelvin Heights Reservoir full.

In the future the Shotover Country Bores and Water treatment Plant will supply into the Frankton area. Therefore, it is assumed supply of water will not be an issue.

4.2 SILVER CREEK SERVICING

The impacts of the proposed Silver Creek Development has been considered in simplified terms. Demands of the full development have been calculated and assigned to the proposed connection point on the existing network with the new change in operation on Frankton Road. The intention is to assess the impacts of the additional demand on the network and assess the system performance.

Demands are considered in terms of pumping to a reservoir over 18 hours. The three demand scenarios have been modelled with the impacts on the minimum pressure at the point of supply and available fire flows presented in Table 3. The assessment has been undertaken *without* the Middleton Road Pump Station operating.

Table 3 Silver Creek Servicing Analysis

	Average (L/s)	Peak Hour (L/s)	Peak Day (L/s)	Pumped Flowrate over 18hrs (L/s)	Reduction in Minimum Pressure at Point of Supply (m)	Adverse Impact on Network LOS for Minimum Pressures	Adverse Impact on Network LOS for Fireflow
700 L/ p / day	14.22	56.88	26.99	36	18.5	Yes	Yes
250 L/p/day	5.08	20.31	9.64	12.85	5	No	No
1000 L/conn (333.33 L/p/day)	6.77	27.08	12.85	17.14	7	No	No

These results indicate a higher flowrate can be achieved with the new zone valve on Frankton Road and supply to Silver Creek from Frankton than the previous analysis. Impacts on minimum pressures are starting to show with 17.14 L/s, however levels of service are still met. It should be noted again, this assessment is without the Middleton Road Pump Station operating, therefore the total flow would need to include the Middleton Road pumped flowrate.

Based on the low flow scenario, Silver Creek would draw 12.85 L/s to service the full development, which enables 4.5 L/s for the Middleton Road pump Station or 1 pump operating.

4.2.1 Middleton Road Scheme

The approved Middleton Road Scheme allows for 555 Lots, where 200 Lots could be allocated to the Silver Creek development.

Based on the same low demand calculation (250L/p/day), to service 555 Lots, the peak day demand would be 9.15L/s, or 12.2L/s pumped the reservoir over 18 hours. This would equate to 3 pumps operating in the existing station at a flowrate of 12.9L/s.

The demand from Silver Creek would reduce by 200 Lots (approximately two thirds), to 6.34L/s or 8.45L/s pumped over 18 hours.

The total flowrate required from the Frankton Road Watermain to service the two development areas results in 21.35L/s, with 12.9L/s through Middleton Road Pump Station and a further 8.45L/s through a new Silver Creek pump station.

With these two flows applied in the model, a reduction in pressure at the connection point of 8.7m was observed, however levels of service for minimum pressure (above 20m) are still met. Areas of Potters Hill Drive and Teirs Lane begin to be impacted.

4.2.2 Connection through Frankton Roundabout

An assessment has also been undertaken with a new connection between the two 355mmm diameter mains on Frankton Road across the Frankton roundabout. The results of this assessment indicate the flowrate of 21.35L/s could be achieved while still maintaining levels of service above 25m on Tiers Lane and Potters Hill Drive. Fire flows are also improved.

5 SUMMARY

It is important for QLDC to ensure the best outcomes for the community and future property owners, and to ensure the delivery of safe reliable drinking water.

The Silver Creek development proposes 580- 585 new residential dwellings up steep terrain along the Frankton Arm.

WSE were engaged to undertake further analyses to explore additional options to address servicing the full development and to assess required storage and upgrades based on residential dwelling or DUEs (Development Unit Equivalent). This assessment should be read in conjunction with the technical memorandum produced in February 2023 which provides details of the analysis previously undertaken.

Storage was assessed for each of the demand scenarios (700L/p/day, 250L/p/day and 1000L/conn/day) and tabulated a per dwelling volume to enable simplified calculation of development stages.

A change to the operation of the network was considered to service the Silver Creek full development. This change involves moving (or removing) the existing PSV/NRV arrangement on Frankton Road further to the west and creating a zone boundary valve. This would enable the Middleton Road Reservoir and proposed Silver Creek Development to be supplied from the Frankton area. The results of the analysis are favourable and allow greater flows to be drawn from the Frankton watermain than the existing system operation.

This analysis assumes an existing connection, not currently shown as active in the GIS, on the corner of McBride Street and Frankton Road connecting the 355mm diameter HDPE watermain on Frankton Road to the 250mm AC watermain on McBride Street.

Further consideration was given to an upgrade to reduce head loss through the Frankton Beach area by constructing a connection between the two 355mm diameter watermains on Frankton Road. This connection improves system pressures and fire flow to the Frankton Arm.

5.1 RECOMENDATIONS

The previous analysis recommended, under the existing system operation, that the development should be limited to the allocation from the existing Middleton Road scheme, unless wider network upgrades are considered. This recommendation remains as an initial development stage of up to 200 Lots.

Based on this analysis, the removal of the existing non-return valve and pressure sustaining valve arrangement on the Frankton Road and the construction of a new a closed valve / bypass arrangement between Goldfield Heights and Potters Hill Drive, could enable the full Silver Creek development to proceed under the low demand scenario conditions.

It is still recommended further investigation of the network is carried out to confirm the capacity of the Frankton Road watermain and more comprehensive calibration/validation of the hydraulic model in this area.

It is also noted, the future integration of the new Shotover Country Water Treatment Plant is also on-going, and changes to the system operation and system pressures may occur which could alter this analysis.

WATERSHED

Construction of the connection between the two 355mm diameter HDPE watermains on Frankton Road is recommended as it does improve network pressures ensuring customers remain over 25m, however levels of service are shown in the model to be meet with minimum pressure above 20m.

The hydraulic model is a representation of the physical water supply system and as such, has some limitations which should be noted in the model development and calibration report. Watershed Engineering Limited were not involved in the original development of the Queenstown hydraulic model, nor the conversion of the EPA models to Infoworks WS pro. Updates have been made to the model by Watershed to enable it to be used for this project brief. The demands and peaking factors used to assess the development are based on assumptions and the actual final water demands may vary.

We trust this report meet your requirements. Please contact Charlotte Broadbent on s 9(2)(a) s 9(2)(a) if you wish to discuss any aspects of this report further.

Revision	Name	Signed	Date
A – Final	Charlotte Broadbent	Charlotte Broadbent	17 March 2023

Disclaimer: This report has been prepared solely for the benefit of for Queenstown Lakes District Council with respect of the particular brief and it may not be relied upon in other contexts for any other purpose without Watershed Engineering Limited's prior review and agreement. Watershed Engineering Limited accepts no responsibility with respect to its use, either in full or in part, by any other person or entity.

Appendix E. PowerNet Confirmation Letter

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Invercargill 9840, New Zealand
P: 03 211 1899
F: 03 211 1880
E: enquiries@powernet.co.nz

Ref: 409650 - Silver Creek

6th March 2023

Mooreliving
820 Frankton Road
Queenstown 9300

Attention: Gavin Moore

Dear Gavin

Provision of Electrical Supply – Silver Creek development, Frankton Road, Queenstown

Please accept this letter as notification that PowerNet Limited on behalf of Lakeland Network, can confirm there is satisfactory electricity network reticulation in the area (via Frankton Road and BP roundabout), for development of the Silver Creek development.

This will allow for connections at 15kVA single phase and larger three phase supplies as required, of the new development once designed.

Lakeland design and reticulate at a distribution voltage of 22kV and 415VAC this provides for an infrastructure resilient for the future market and growth possibilities. We have extensive network within the Wakatipu Basin which is exclusively underground or ground mounted. This ensures a reliable network in extreme weather events.

Additionally, Lakeland network encourage Solar or renewable energy installations, which provide a path toward zero carbon emissions 2030-2050. We design our networks to provide capability for energy export and smart metering for visibility of loads and generation. The technology and application are current if you wish to discuss further.

Yours faithfully

Phil Chittock
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