Homestead Bay Development Consent Application

Engineering Feasibility Assessment

VOLUME 1 - REPORT

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RCL Homestead Bay Limited

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Acronyms / Abbreviations

Acronym / Abbreviation	Full Name
ADF	Average Day Flow
ADWF	Average Dry Weather Flow
BGL	Below Ground Level
CoP	Land Development and Subdivision Code of Practice
DUE	Dwelling Unit Equivalents
HB	Homestead Bay Development
DIR	Design Irrigation Rate
DWQAR	Drinking Water Quality Assurance Rules 2022
DWSNZ	Drinking Water Standard for New Zealand 2022
ESCP	Erosion and Sediment Control Plan
GeoSolve	Geosolve Limited
ITA	Integrated Transport Assessment
KSL	Kōmanawa Solution Ltd
LDSCoP	Land Development and Subdivision Code of Practice
LEI	Lowe Environmental Impact
LoS	Level of Service
LPED	Low-Pressure Effluent Disposal
LTA	Land Treatment Area
MABR	Membrane Aerated Biofilm Reactor
MBR	Membrane Bioreactor
QLDC	Queenstown Lakes District Council
PF	Peaking Factor
PDF	Peak Day Flow
PDWF	Peak Dry Weather Flow
PHF	Peak Hour Flow
PWWF	Peak Wet Weather Flow
QSC	Queenstown Southern Corridor
RCL	RCL Homestead Bay Limited
rPBR	Recirculating Textile Packed Bed Reactor
SBR	Sequence Batch Reactor
SH6	State Highway 6
SAF	Submerged Aerated Filter
SQEP	Suitably Qualified and Experienced Person
WSAA	Water Services Association of Australia
WSA	See WSAA
WSS	Water Supply Scheme
WTP	Water Treatment Plant
WWTP	Wastewater Treatment Plant

Glossary

Term	Definition
Average Day Flow	ADF is expressed as an instantaneous flowrate or daily volume that excludes seasonal and diurnal peak flow variations
Average Dry Weather Flow	ADWF is expressed as an instantaneous flowrate or daily volume that includes seasonal peak flow variations — excludes seasonal, daily diurnal and wet weather I&I peak flow variations.
Average Wet Weather Flow	AWWF is expressed as an instantaneous flowrate or daily volume that includes wet weather I&I peak flow variations — excludes seasonal and daily diurnal peak flow variations.
Debris Floods	A debris flood is a rapid flow of water charged with debris. Debris floods are distinguished from clearwater floods when sediment on the bed begins to move together, en-masse, and larger particles become suspended because of bedload destabilisation. Sediment concentration is typically less than 25% by volume.[1]
Design Irrigation Rate	DIR is the application rate to LTAs measured as a depth per day and is usually expressed in mm/day.
Drinking Water	As defined in the Health (Drinking Water) Amendment Act
Dwelling Unit	Any building or group of buildings, or part thereof used, or intended to be used principally for residential purposes and occupied, or intended to be occupied, by not more than one household[2].
Dwelling Unit Equivalent	Dwelling Unit Equivalent (DUE) is the equivalent number of residential dwellings based on the unit flowrate and occupancy as defined by the QLDC LDSCoP, cl. 5.3.5.1 & 5.3.5.3 for wastewater and cl. 6.3.5.6 for water.
Firefighting Water Supply	Supply of water, available to the Fire Service for firefighting, that complies with NZS PAS 4509:2008, where the required flow of water is at a minimum running pressure and of sufficient duration[3]
Operating Storage	Is drinking water storage that "…shall cater for demands exceeding the maximum available inflow rate."[4]
Peak Day Flow	PDF is expressed an instantaneous flowrate or daily volume that incudes seasonal variations.
Peak Dry Weather Flow	PDWF is expressed as an instantaneous flowrate or daily volume that includes seasonal and daily diurnal peak flow variations — excludes wet weather I&I peak flow variations.
Peak Hour Flow	PHF is expressed an instantaneous flowrate that includes seasonal and daily diurnal peak flow variations.
Peak Instantaneous Flow	See Peak Hour Flow.
Peak Wet Weather Flow	PWWF is expressed as an instantaneous flowrate that includes seasonal, daily diurnal, and wet weather I&I peak flow variations.
Queenstown Southern Corridor	Land bordered by the Kawarau River to the north, Remarkables Mountain Range to the east, Peninsula Hill and Lake Wakatipu to the west, and Drift Bay to the south
Reserve Storage	For drinking water storage reservoirs the "Reserve storage shall cater for system component failure. Reserve storage capacity shall be determined from a risk assessment study of the supply zone / system, which considers characteristics of the zone / system to determine the risk (consequences and frequency) to water supply continuity and pressure in the event of a system component failure.

[1] GeoSolve. *Natural Hazard Assessment – Homestead Bay Queenstown*. March 2025. 23.
[2] Standards New Zealand, Land Development and Subdivision Code of Practice, NZS4405:2010 (Standards New Zealand, 2010), https://www.standards.govt.nz/shop/NZS-44042010. 25.
[3] Standards New Zealand, New Zealand Fire Service Firefighting Water Supplies Code of Practice, SNZ PAS 4509:2008. (Standards New Zealand, 2008), https://www.fireandemergency.nz/assets/Documents/Business-and-Landlords/Building-and-designing-for-fire-safety/NZFS-firefighting-water-supplies-code-of-practice.pdf, 14.
[4] Water Services Aussociation Australia. 2022. *Water Supply Code of Australia.* WSA03-2011. Version 3.2. Water Services Association of Australia Ltd. 88-89.

Term	Definition
	Typically, reserve storage capacity should be taken to be equal to ⅓ peak day demand."[5]
Sprinkler Demand	Water supply demand for the Sprinkler System specified, which is in addition to Firefighting Water Supply, where the required flow of water is at a minimum running pressure and of sufficient duration for the sprinkler system hazard type[6]
Subsurface Drip Irrigation	A form of Low-Pressure Effluent Disposal
Wastewater	"Water that has been used and contains unwanted dissolved or suspended substances from communities, including homes, businesses, and industries".[7]
Wastewater Property Connection	A short (typically) wastewater pipe that connects the property to the Wastewater Reticulation Network.
Wastewater Reticulation Network	A network of pipelines that collect wastewater from individual properties via direct consumer connections and conveys it to the Wastewater Transfer System.
Wastewater Servicing Scheme	A combination of elements that together makes up a functioning wastewater collection, conveyance, treatment and disposal system.
Wastewater Transfer System	A combination of bulk infrastructure that together conveys wastewater from the Wastewater Reticulation Network to wastewater treatment facilities.
Water Distribution System	Part of the water supply scheme comprising pipelines, service reservoirs, pumping stations and other assets by which water is distributed to the consumers. It begins at the outlet of the water treatment works and includes the reticulation system.
Water Reticulation Network	A network of pipelines that supplies individual properties with Drinking Water via direct consumer connections. It is connected to the Water Transfer System and generally consists of smaller diameter pipes than Trunk Mains.
Water Service Connection	A short (typically) pipe that connects the property to the Water Reticulation Network.
Water Supply Scheme	A combination of elements that together makes up a functioning water supply from the point of raw water abstraction to consumers.
Water Transfer System	A combination of infrastructure that together conveys water from raw water abstraction sites to water treatment facilities.
Water Trunk Main	A water main that is part of the Water Transfer System, which is designed for bulk transfer e.g., interconnects the borefield, treatment works, reservoirs and/or supply areas, without direct consumer connections.

[5] Water Services Aussociation Australia. 2022. *Water Supply Code of Australia.* WSA03-2011. Version 3.2. Water Services Association of Australia Ltd. 88-89.
[6] SNZ PAS 4509:2008, 33.
[7] Queenstown Lakes District Council. 2020. *Land Development and Subdivision Code of Practice*. Queenstown: Queenstown Lakes District Council. https://www.qldc.govt.nz/media/s21al2gz/2020-qldc-land-development-and-subdivision-code-of-practice.pdf.22.

1 Introduction

1.1 Purpose of the Report

The purpose of this report is to assess the feasibility to service a new housing development on Lot 8 DP 443832 and Lot 12 DP 364700 — called Homestead Bay — in support of RCL Homestead Bay Limited's Housing and Land Development application as part of the New Zealand governments Fast Track Approvals process.

This report shows the viability of providing infrastructure services for the Homestead Bay and, where beneficial, presents bulk servicing options to other developments nearby. It nominally covers the following:

- Provision of infrastructure for the supply of potable water.
- Provision of infrastructure for the collection and disposal of wastewater.
- Provision of infrastructure for the management of stormwater.
- Provision of transport infrastructure.
- Suitability of foundation conditions.
- Natural hazards assessment.
- Earthworks assessment.

1.2 Background

Homestead Bay is located south of Jacks Point and to the west of SH6, see Figure 1-1 and Figure 1-2 below.

Figure 1-1: Locality Plan[8]

[8] Geosolve. *Natural Hazard Assessment*. March 2025. 4.

Figure 1-2: Homestead Bay Masterplan[9]

Lot 8 DP 443832 is currently partially zoned as Rural and as Jacks Point Zone in QLDCs Proposed Operative District Plan and Lot 12 DP 364700 as Jacks Point Zone. However, Homestead Bay is included in Schedule 2 of the New Zealand Government's Fast-track Approvals Bill[10].

1.3 Regulatory Requirements

New Zealand's Fast-track Approvals Bill plans to make additional land development and housing available for thousands of new dwellings.[11] It passed its third reading on 17 December 2024 and passed into law. Homestead Bay was selected by the government as having significant regional benefits and included in Schedule 2 of the Act.[12]

Subpart 1 of the Local Government Act 2002 — Specific obligations to make assessments of drinking water, wastewater, and sanitary services and to ensure communities have access to safe drinking water.[13]

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[9] Pattersons Land Professionals. Drawing No. 001. Lot 8 DP 443832 AND Lot 12 DP 364700 Proposed Subdivision Plan.

[10] Fast-track Approvals Bill. (2024), https://www.legislation.govt.nz/bill/government/2024/0031/39.0/LMS943195.html.

[11] Fast-track Approvals Bill. (2024), https://www.legislation.govt.nz/bill/government/2024/0031/39.0/LMS943195.html.

[12] Fast-track Approvals Bill. (2024), Bill Schedule 2, https://www.legislation.govt.nz/bill/government/2024/0031/39.0/LMS943327.html.

[13] Local Government Act. (2002). https://www.legislation.govt.nz/act/public/2002/0084/latest/DLM170873.html?search=qs_act%40bill%40regulation%40deemedreg_Local+Government+Act+2002+_resel_25_h&p=1

Local Government (Water Services Preliminary Arrangements) Act 2024

1.4 Homestead Bay Development Size

The Homestead Bay development encompasses an area of approximately 205 hectares. It is proposed to consist of low, medium and high-density residential lots, reserves and local centre land uses. A primary school can also be accommodated if the Ministry of Education require. Table 1-1 shows a breakdown of the land-use types and their capacity.

Table 1-1: Development Size Based on Proposed Subdivision Plan[14]

Activity / **Residential**	**Unit Description**[15]	**Number of Units**	**Occupancy Per Unit**[16]	**DUEs**[17]
Standard Density Lots	Dwelling Unit	1438	3 p/Lot	1438
Medium Density Lots	Dwelling Unit	203	3 p/Lot	203
High Density Super Lots	Residential Flat	890	3 p/Lot	890
			TOTAL DUEs	**2531**
Non-residential	**Industry Type**	**Area**		
Commercial	Light[18]	2.50 ha		

[14] See Pattersons Land Professionals drawing 001. *Lot 8 DP 443832 AND Lot 12 DP 364700 Proposed Subdivision Plan* showing the maximum yield of the development.
[15] Queenstown Lakes District Council. 2020. *Land Development and Subdivision Code of Practice*. 19 & 21.
[16] Queenstown Lakes District Council. 2020. *Land Development and Subdivision Code of Practice*. 164.
[17] See Glossary for the definition of DUEs.
[18] Queenstown Lakes District Council. 2020. *Land Development and Subdivision Code of Practice*. 141.

2 Water Supply

2.1 Introduction

A key element of a healthy community is access to safe, secure and uninterrupted supply of potable water delivered at flows and pressures appropriate for the activities being served. Consequently, potential sources that would meet these fundamental requirements for Homestead Bay were investigated and are reported in the subsequent sections.

2.2 Regulatory Requirements

As discussed in Section 1.3, New Zealand's Fast-track Approvals Bill plans to make additional land development and housing available for thousands of new dwellings in New Zealand, and Homestead Bay is one of them and is therefore included in Schedule 2 of the Act.[19]

The Water Supply Scheme (WSS) serving Homestead Bay will comply with the following guidance and regulatory documents:

- Drinking Water Standard for New Zealand 2022, Revised 2024 (DWSNZ)
- Drinking Water Quality Assurance Rules 2022 (DWQAR)[20]
- Queenstown Lakes District Council Land Development and Subdivision Code of Practice 2020 (QLDC CoP)
- New Zealand Standard 4404:2010 Land Development and Subdivision Infrastructure (NZS 4404:2010)
- New Zealand Fire Service Firefighting Water Supplies Code of Practice (SNZ PAS 4509:2008)

2.3 Servicing Strategy

The concept to service Homestead Bay with potable water will be to establish a new water scheme called the Homestead Bay Water Supply Scheme (WSS), via a new borefield and treatment system within the development.

The WSS will include the following key components:

- New borefield with adequate capacity adjacent to Lake Wakatipu (direct lake-take is not favoured because of previous problems with lake-snow), including possible inline booster pump station to convey flows from the borefield to the WTP.
- Treatment of raw water to the New Zealand Drinking Water Standard. Based on current available raw water quality testing, the following treatment process is proposed:[21;22]
    - Pre-oxidation.
    - Greensand Plus® filtration, or equal.
    - UV disinfection.
    - Finished water chlorination.
    - pH adjustment and alkalinity control.
    - Chlorine contact is achieved in the storage reservoir to establish free available chlorine residual in the downstream distribution network which is effective against most bacteria.

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[19] Fast-track Approvals Bill. (2024), https://www.legislation.govt.nz/bill/government/2024/0031/39.0/LMS943195.html. Schedule 2.
[20] Taumata Arowai. *Drinking Water Quality Assurance Rules (Revised 2024)*. 2022.
[21] See 310104425-00-000-C0404 for indicative water treatment schematic.
[22] Further details of the water treatment process are discussed in Section 2.7.

- Provision of storage reservoir to maintain the required residual pressure Levels of Service (LoS).

- Booster pump station to service lots at higher elevations.

The preferred location of the storage reservoirs, borefield and treatment plant are shown in Figure 2-1.

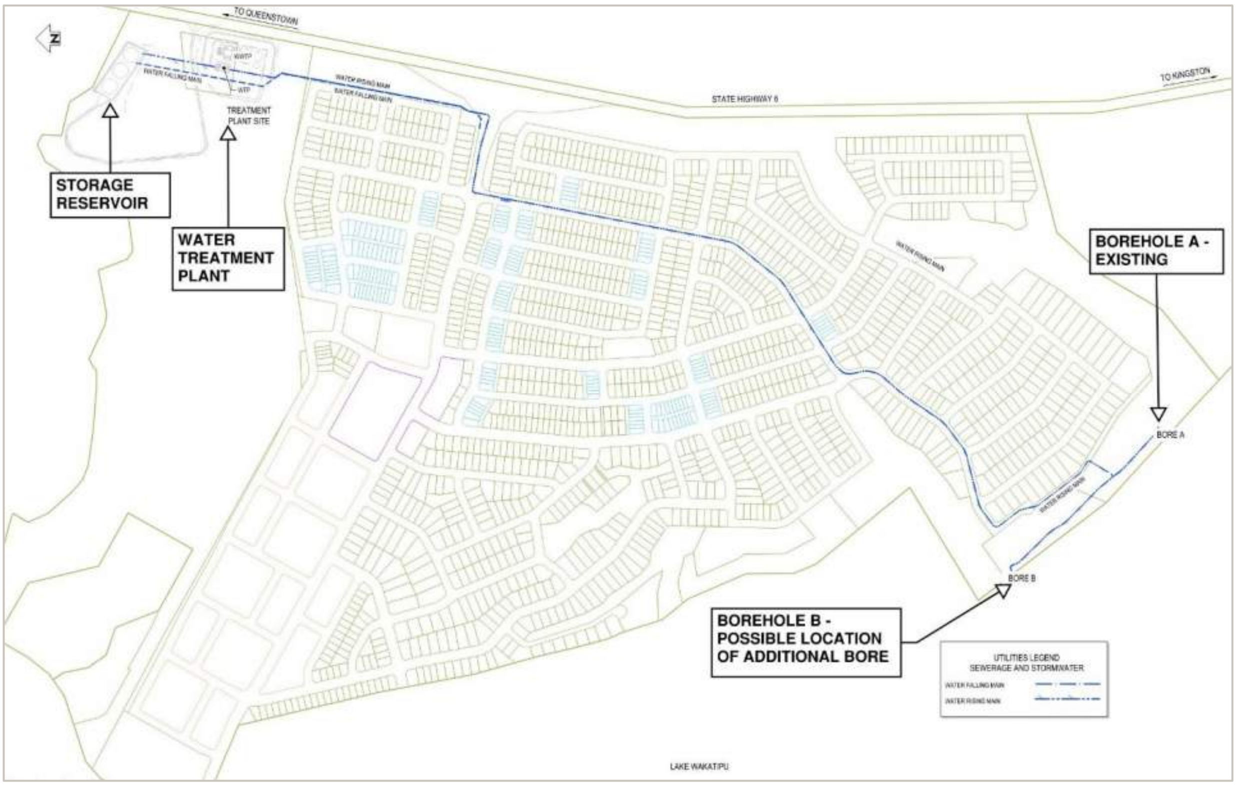

Figure 2-1: Indicative Location of Borefield and Storage Reservoirs

The key infrastructure is discussed further in Section 2.6 to Section 2.11.

2.4 Water Demands

The demand criteria for Homestead Bay are based on QLDCs Land Development and Subdivision Code of Practice (CoP) and agreed practice. A summary follows:

- The demand rate per DUE is proposed to be the same as Hanley's Farm as land uses, lot sizes and densities are similar. The demand units proposed are 1,000 litres per lot per day for the Average Daily Flow (ADF) in litres per second. Ulrich Glasner (the former QLDC Chief Engineer in 2016) approved this figure for Hanley's Farm based on a flow metering study that showed that 1,000 litres per lot per day is representative of actual flows.
- The Peak Daily Flow (PDF) in litres per second is calculated by multiplying the ADF by a peaking factor of 2.0. This peaking factor was approved by QLDC and is shown in Mott MacDonalds report titled "Coneburn and Hanley's Farm Water Supply Infrastructure Assessment 2021 (v5), see Appendix A in Volume 2.[23]
- The Peak Hour Flow (PHF) in litres per second is calculated by multiplying the ADF by a peaking factor of 4.92. This peaking factor was approved by QLDC in the same report, see Appendix A in

[23] Mott MacDonald, *Coneburn and Hanley's Farm Water Supply Infrastructure Assessment 2021 (v5)*, Technical Memorandum Revision: Final-E, (Queenstown: 15 October 2021), 2.

Volume 2.[24]

Fire flow requirements are based on the New Zealand Fire Service Firefighting Water Supplies Code of Practice, SNZ PAS 4509:2008.[25] A summary follows:

- FW2 Fire Class Firefighting Demand is appropriate for all residential land-uses without a sprinkler system.[26]
- FW2 Fire Class Firefighting Demand plus Sprinkler Demand is appropriate for commercial land-uses and multi-level apartments.[27] The total water demand to combat a fire is equivalent to FW3 for Homestead Bay.[28]
- Pressures during ⅔ of Peak Day Demand plus Firefighting Demand plus Sprinkler Demand must be at least 100kPa for the fire flow class selected for Homestead Bay.[29]

QLDC's Level of Service (LoS) for the supply of potable water is as follows:

- Residual pressure during the PHF shall be between 300 kPa and 900 kPa.[30]
- Headlosses through pipes must be no greater than:[31]
    - 5m/km in pipes smaller than 150 mm NB.
    - 3m/km in pipes greater than 200 mm NB.

Potable water demand (domestic and commercial land-uses) for Homestead Bay is shown in Table 2-1.

Table 2-1: Homestead Bay Potable Water Demand

Activity	**Number of Units**	**Demand Rate per Unit**	**ADF[32] (L/s)**	**PDF[33,34] (L/s) PF= 2**	**PHF[35,36] (L/s) PF= 4.92**
Standard Density Lots	1,438 DUE	1,000 L/lot/d	16.64	33.29	81.89
Medium Density Lots	203 DUE	1,000 L/lot/d	2.35	4.70	11.56
High Density Super Lots	890 DUE	1,000 L/lot/d	10.30	20.60	50.68

[24] *Coneburn and Hanley's Farm Water Supply Infrastructure Assessment 2021 (v5)*, 3.
[25] Standards New Zealand, *New Zealand Fire Service Firefighting Water Supplies Code of Practice*, SNZ PAS 4509:2008. (Standards New Zealand, 2008), https://www.fireandemergency.nz/assets/Documents/Business-and-Landlords/Building-and-designing-for-fire-safety/NZFS-firefighting-water-supplies-code-of-practice.pdf.
[26] SNZ PAS 4509:2008. 19-20, Table 1 and Table 2.
[27] SNZ PAS 4509:2008. 33, Table C1 Appendix C.
[28] As defined by the NZS PAS4509:2008 Code of Practice.
[29] SNZ PAS 4509:2008. 22.
[30] Queenstown Lakes District Council. *Land Development and Subdivision Code of Practice*. 2020. https://www.qldc.govt.nz/media/s21al2gz/2020-qldc-land-development-and-subdivision-code-of-practice.pdf. 166
[31] QLDC, *Land Development and Subdivision Code of Practice*, 2020, 163.
[32] QLDC, Land Development and Subdivision Code of Practice. 2020, 164.
[33] QLDC, Land Development and Subdivision Code of Practice, 2020, 164.
[34] Mott MacDonald, *Coneburn and Hanley's Farm Water Supply Infrastructure Assessment 2021 (v5)*, Technical Memorandum Revision: Final-E, (Queenstown: 15 October 2021), 2.
[35] QLDC, Land Development and Subdivision Code of Practice. 2020, 164.
[36] Mott MacDonald, *Coneburn and Hanley's Farm Water Supply Infrastructure Assessment 2021 (v5)*, Technical Memorandum Revision: Final-E, (Queenstown: 15 October 2021), 3.

Activity	**Number of Units**	**Demand Rate per Unit**	**ADF[32] (L/s)**	**PDF[33,34] (L/s) PF= 2**	**PHF[35,36] (L/s) PF= 4.92**
Commercial[37,38]	2.50 ha	0.44 L/s/ha	1.10	2.20	5.41
		Total	**30.39**	**60.79**	**149.54**

Firefighting water demands for Homestead Bay are shown in Table 2-2.

Table 2-2: Demand on the Reticulation System from the Fire Sprinkler System and Firefighting Plus an Allowance for Domestic Demand for Homestead Bay[39]

Firefighting Demand - FW2[40] (L/s)	**Sprinkler Demand[41, 42] (L/s)**	**Peak Annual Domestic Demand (L/s)**	**⅔ Peak Annual Domestic Demand (L/s)**	**Firefighting + Sprinkler + Domestic Demand (L/s)**
25	25	149.54	99.69	149.69
	Total Firefighting + Sprinkler + Domestic Water Demand			**149.69**

FW2 fire classification requirements apply to residential properties without sprinkler systems. It also applies to commercial properties and multi-storey residential apartments with sprinkler systems. However, additional demand for sprinkler systems will be catered for in the infrastructure serving these land-use activities.[43]

Should commercial properties and multi-storey residential apartments desire their fire suppression system to be designed without a sprinkler system, the demand on the reticulation system for Firefighting would be FW3.[44] Homestead Bay will achieve FW3 in those areas, which is equivalent to FW2 plus Sprinkler Demand which has been assessed in Table 2-2 should this option be preferred by property owners.

2.5 Existing Infrastructure

Homestead Bay is not currently serviced by any of QLDCs water supply schemes. The closest water supply scheme is the Jacks Point Water Supply Scheme, which is immediately to the north of Homestead Bay. This is a private scheme, intended to serve only the Jacks Point development.

The Queenstown Southern Corridor (QSC) is bordered by the Kawarau River to the north, Remarkables Mountain Range to the east, Peninsula Hill and Lake Wakatipu to the west, and Oraka (Drift Bay) to the south. Some developments between the Kawarau River and Jacks Point are supplied from the Kelvin

[37] Based on 90% of water becoming wastewater. Metcalf & Eddy Inc., George Tchobanoglous, Franklin Burton, and David H. Stensel. 2003. Wastewater Engineering - Treatment and Reuse. 4th. New York: McGraw Hill Professional.155.
[38] Queenstown Lakes District Council. 2020. *Land Development and Subdivision Code of Practice*. Queenstown: Queenstown Lakes District Council. 141
[39] Standards New Zealand, *New Zealand Fire Service Firefighting Water Supplies Code of Practice*, SNZ PAS 4509:2008. (Standards New Zealand, 2008). 33., https://www.fireandemergency.nz/assets/Documents/Business-and-Landlords/Building-and-designing-for-fire-safety/NZFS-firefighting-water-supplies-code-of-practice.pdf. 33.
[40] SNZ PAS 4509:2008, 19-20.
[41] Sprinkler system assessed to be Ordinary Hazard (OH) Type as defined in SNZ PAS4509:2008 Table C1.
[42] SNZ PAS 4509:2008, 33.
[43] SNZ PAS4509:2008. 18 & 33.
[44] SNZ PAS 4509:2008, 19-20.

Heights Water Supply Scheme and supplemented from the Shotover Country Bores. However, the transfer main terminates at Hanley's Farm, immediately north of the Jacks Point Water Supply Scheme.

2.6 New Water Supply

2.6.1 Introduction

The Jacks Point Water Supply Scheme, which is privately owned, is at full capacity so servicing from this scheme is not possible without significant upgrades and capital investment.

Similarly, QLDCs closest water supply scheme has limited capacity and requires significant upgrades for any new developments to connect. The remaining capacity is earmarked for consented developments within the existing scheme boundary. The main deficiency in the existing network is that the Shotover Country Bores operate constantly during periods of peak demand in conjunction with insufficient storage capacity in the scheme. Consequently, as a minimum QLDC have identified the need for a new storage reservoir in the QSC area to keep pace with the growth of development. QLDC have an undefined construction horizon so, a new water supply scheme was investigated.

In planning a new water supply for Homestead Bay, one needs to consider the potential sources of raw water. The main options are Lake Wakatipu and local groundwater, which have contrasting microbiological security characteristics. Lake water is available and abundant with excellent overall measurable water chemistry. However, the protections in water supplies based on surface raw water have been brought into question by water contamination of water supplies involving such water-borne pathogens as protozoa, bacteria and viruses.[45]

2.6.2 Assessment of Potential Water Sources

2.6.2.1 Introduction

Water supply in the QSC is currently obtained from three sources; lake water piped via the QLDC Water Scheme, lake water pumped directly from Homestead Bay, or local groundwater from wells F42/0150 and potentially F42/0103. See Table 2-3 for details and their location in Figure 2-2.[46] The QLDC scheme has limited capacity and therefore alternatives were considered. Lake water has problems with water quality and is not generally being adopted in the area (e.g., QLDCs approach, Jacks Point).

Table 2-3: Existing Bores

Well No.	Depth (m)	Dia. (mm)	DTW (m)	Date Drilled	Owner	Screen Top (m)	Screen Bottom (m)	Water Take Consent
F42/0150	35.76	300	0.89	8-Jul-17	Homestead Bay Trustees	29.77	35.76	_
F42/0103	50.45	150	32.45	20-May-99	Lakeside Estates	47.48	50.48	RM11.151.01.V1

Note: Dia. = diameter of bore; DTW = Depth to Water.

[45] Rekker, J.H., *RCL Homestead Bay Ltd: Groundwater Exploration & Effects of Taking Groundwater for Water Supply,* Prepared by Kōmanawa Solutions Ltd for Stantec NZ Ltd and RCL Group, KSL Report No. Z24004GSI, Christchurch, 2024, 9.
[46] Rekker, J.H., *RCL Homestead Bay Ltd: Groundwater Exploration & Effects of Taking Groundwater for Water Supply,* 2024, 8

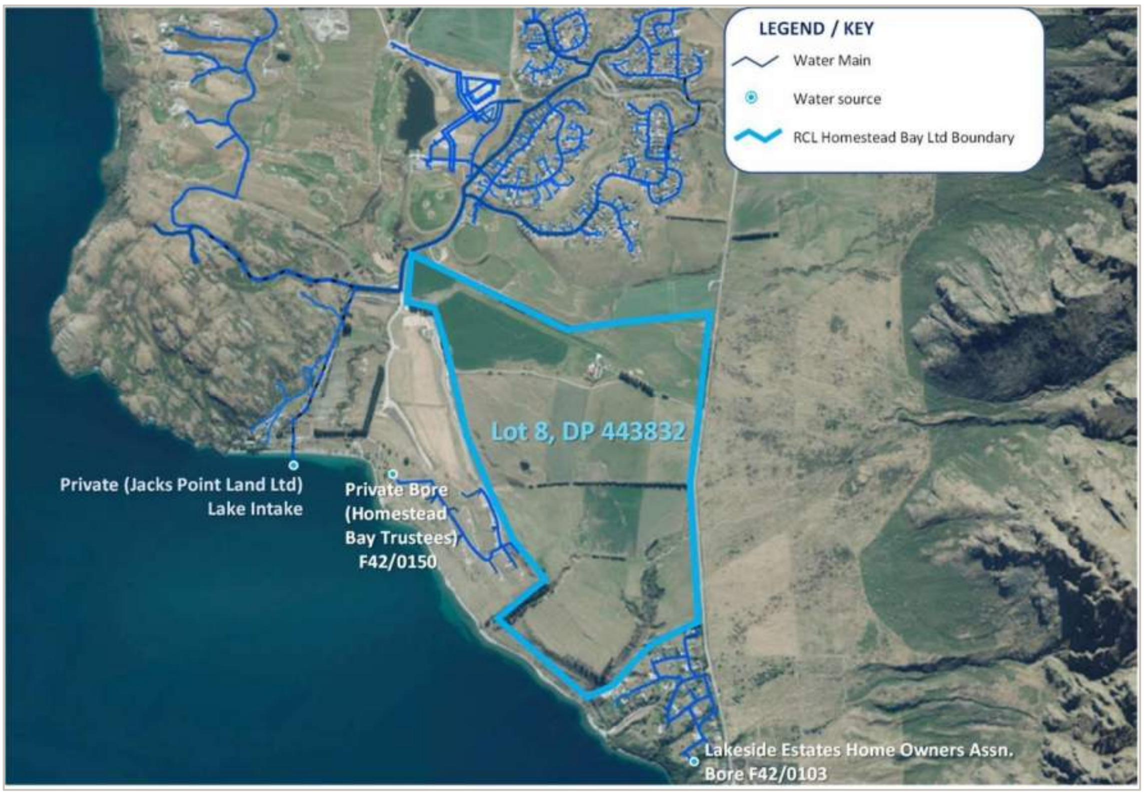

Figure 2-2: Location of Local Groundwater Bores F42/0150, F42/0103 and Jacks Point Lake Intake

Groundwater holds something of a premium for water supply development due to the usual stability in water quality (with respect to turbidity, microscopic pathogens, or solutes).

In view of a preference for a groundwater sourced water supply over lake water (due to the risk of contamination), and also in view of the potential groundwater resource indicated to occur at private water supplies either side of the Lot 8 DP 443832, RCL Group commissioned Kōmanawa Solutions Ltd to carry out drilling investigations to establish a test production bore and undertake groundwater testing for quantity and quality aspects of a future water supply. Relevant aspects from their investigations are discussed in Sections 2.6.2.2 to 2.6.2.4

Homestead Bay will therefore be serviced by a new water supply scheme abstracted from lakeside bores. The major infrastructure consists of the following key components:

- Lakeside borefield
- Water treatment facility
- Water booster pump station
- Transfer mains from the borefield to the treatment facility
- Storage reservoirs
- Transfer main to the distribution network.

The new Homestead Bay Water Supply Scheme (WSS) is discussed further in the following sections.

2.6.2.2 Bore Investigations

Background Hydrogeology

Kōmanawa Solutions Ltd investigated available groundwater sources within Homestead Bay and the effects of abstraction for water supply.[47] A copy of their report is in Appendix B. The following sections are a summary of key aspects in the report.[48]

It is clear from published geological and geomorphological mapping of the sedimentary basin that glacial deposits including alluvial fans containing silty, sandy gravel host an active groundwater flow system and local aquifers. The groundwater resource of this sedimentary basin is estimated as being at least 1.38 million m³/per annum.[49]

There is currently only one groundwater take consent in the Homestead Bay sedimentary basin with records showing only 0.09 million m³/per annum of groundwater is allocated and is therefore significantly under-allocated.[50]

Test Drilling

Two side-by-side bores were drilled in February 2024 near the shore of Homestead Bay, Lake Wakatipu, within the RCL property boundary.

Drilling at the primary site on exploration bore CC11/0151P with 150 mm diameter casing was logged as non-water-bearing silty sands and clays down to the top of water-bearing gravels at 75 metres below ground and the drilling extended further to a depth of 95 metres amongst water-bearing sandy gravels. To provide a bore for full-scale groundwater testing of capacity, a second bore (CC11/0151) with a 300 mm diameter was drilled and installed alongside the exploration bore. This test production bore was screened from 87.9 m to 94.9 m below ground and developed using air-lift over-pumping.

Indications are that the natural gravel pack is adequately graded for optimal well screen hydraulic efficiency. Subsequent multi-day pumping with a submersible pump caused variability in drawdown that suggested re-packing of the formation surrounding the well screen was still taking place. As a result, flow rates and the internal water level became unstable during the multi-day pumping test.

Test Results

Pumping test analysis point to moderate transmissivity (approximately 200 square metres per day) and leaky aquifer properties, with moderate storativity. The groundwater pressure measured within the bores immediately after their installation was 1 – 2 m above ground level. The groundwater pressure in the bores was also estimated to be approximately 12 m above lake level in Lake Wakatipu, of the bores lying about 85 m to the southwest. This combination of groundwater properties, significant depth available for pumping drawdown and high initial groundwater level contributes to a measured capacity for the test production bore of approximately 44 L/s.

2.6.2.3 Groundwater Security

Drinking water supplies need to consider the microbiological security and general water quality of the natural raw water source. Typically, both Lake Wakatipu and sedimentary basin groundwater have excellent overall water quality. However, Lake Wakatipu has a higher potential for microbes as protozoa, bacteria or viruses episodically entering a lake water intake require broad spectrum water treatment for microbe removal and protection against human pathogens.

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[47] Rekker, J.H., RCL Homestead Bay Ltd: Groundwater Exploration & Effects of Taking Groundwater for Water Supply, Prepared by Kōmanawa Solutions Ltd for Stantec NZ Ltd and RCL Group, KSL Report No. Z24004GSI, Christchurch, 2024, 19.

[48] Stantec New Zealand Ltd, *Groundwater Take from Homestead Bay, Queenstown, Assessment of Environmental Effects*, Technical Report, Draft for Client Review, (Queenstown: 1 August 2017).

[49] *Groundwater Take from Homestead Bay, Queenstown, Assessment of Environmental Effects*, 2017.

[50] Rekker, J.H., *RCL Homestead Bay Ltd: Groundwater Exploration & Effects of Taking Groundwater for Water Supply,* Prepared by Kōmanawa Solutions Ltd for Stantec NZ Ltd and RCL Group, KSL Report No. Z24004GSI, Christchurch, 2024, 19.

Kōmanawa Solutions considered prospects for water quality management to be good[51]. The source aquifer has solid indications of exhibiting a level of protection against protozoa, bacteria and viruses. Water chemistry monitoring for stability analysis and potentially isotope dating is recommended, and regular testing is in progress. The bore construction using a 6-metre-deep pre-collar and bentonite seal demonstrated that a sanitary seal could be installed into the top of the silty capping layer, which was found at 4.8 metres depth.

2.6.2.4 Basis for Allocating Groundwater

The allocation and regional water resource management of groundwater in the project area is currently governed by the Otago Regional Plan: Water.

Extractive groundwater use is not considered to be significant, as most water requirements for communal water supply or irrigation are obtained from piped water supply networks derived from surface water, while the two communal water supplies obtained from bores [Homestead Bay Trustees, Nathaneal Place and Lakeside Estates] are small in terms of the number of dwelling connections. ORC does not undertake any State of the Environment (SOE) groundwater monitoring in the area, such as recording groundwater level or taking regular groundwater samples for laboratory analysis. The Jacks Point – Homestead Bay – Remarkables sedimentary basin does not form a declared aquifer within the Regional Plan: Water for Otago, therefore the groundwater resources are subject to only general Policies and Rules within the Regional Plan: Water. Special groundwater management of the area is not proposed within the Otago Land & Water Regional Plan set down for notification in June 2024 and is still to be enacted.[52]

2.6.3 Additional Bore Supply Capacity

The investigations undertaken to date have proven that the sustainable yield from the test production bore referred to as Bore A is 44 L/s. This is adequate to meet the peak day water demand from 1,900 dwellings or dwelling equivalents in Homestead Bay Lot 8.

To meet the peak day water demand for the ultimate development of 2,547 dwellings or dwelling equivalents in Homestead Bay Lot 8, an additional water supply of 16.8 L/s is required. This additional supply will be obtained from a second future bore.

Investigations will be undertaken to confirm the availability and location of a second bore source. The preferred drilling site for the second bore is Bore B (shown in Figure 2-1) where there is a formed platform at the edge of the proposed development, close to the new production Bore A. This location was previously identified by KSL as an alternative should drilling of Bore A not have found a suitable supply. Other alternatives within the development area will be considered but are not favoured at this time because of the likely additional depth (based on the results found in Bore A) to a source aquifer.

Alternative bore sources within the lakeshore reserve are also possible as an additional water source. Well No. F42/0150 has shown that there is suitable groundwater in the vicinity of the lakeshore that this suitable for water supply. Jacks Point Ltd is currently drilling an investigatory bore near to this location.

Subject to the results of investigations for the second bore, a supply quantity in excess of that needed for Homestead Bay can be obtained with a second bore, which could also be used to supply other areas outside Homestead Bay.

2.6.4 Borefield

The borefield infrastructure will likely consist of the following assets:

[51] Rekker, J.H., *RCL Homestead Bay Ltd: Groundwater Exploration & Effects of Taking Groundwater for Water Supply,* Prepared by Kōmanawa Solutions Ltd for Stantec NZ Ltd and RCL Group, KSL Report No. Z24004GSI, Christchurch, 2024, 2.
[52] Rekker, J.H., *RCL Homestead Bay Ltd: Groundwater Exploration & Effects of Taking Groundwater for Water Supply, 2024, 2.*

- Only Bore A is needed for the first 1,900 DUEs, see Figure 2-3 and drawing 310104425-00-000-C0402.
- An additional bore, Bore B, is needed for the remainder of the expected ultimate development. Bore A and Bore B would be installed on a duty/assist arrangement—the assist bore (Bore B) will be on standby for much of the time and therefore also offer a level of redundancy should the other fail or be unavailable for maintenance. Bore A and Bore B are shown in Figure 2-3 and on drawing 310104425-00-000-C0402.
- Boxed spare pump to provide redundancy during peak periods.
- Vehicle access will be from the east as shown on drawing 310104425-00-000-C0402, with suitable turnaround. The concept allows for an 8m truck or 12.5m Hiab to lift pumps and undertake maintenance activities. Alternative access via an upgraded track along reserve land adjoining Lake Wakatipu from Homestead Bay Road will be investigated and may be adopted instead.
- Bore characteristics – bores would have approximately DN300 casings with 7m long screens.
- Wellhead - installation of an above ground sanitary wellhead on a concrete slab within secure building similar to that shown in Figure 2-4 — having a footprint of 5.5m x 3m, which is slightly bigger than the one shown.
- Control cabinet – an electrical control cabinet with pump starters, RTU will be located in the secure building or at a location at the mid-point between the two bore sites (to be confirmed following investigations for the second bore).
- Standby generator - A permanent standby generator will be provided to service both bores, and may be located at Bore B as shown in Figure 2-4, or at a location at the mid-point between the two bore sites (to be confirmed following investigations for the second bore).
- Power - HV power supply will be provided to a new transformer positioned next to the standby generator.

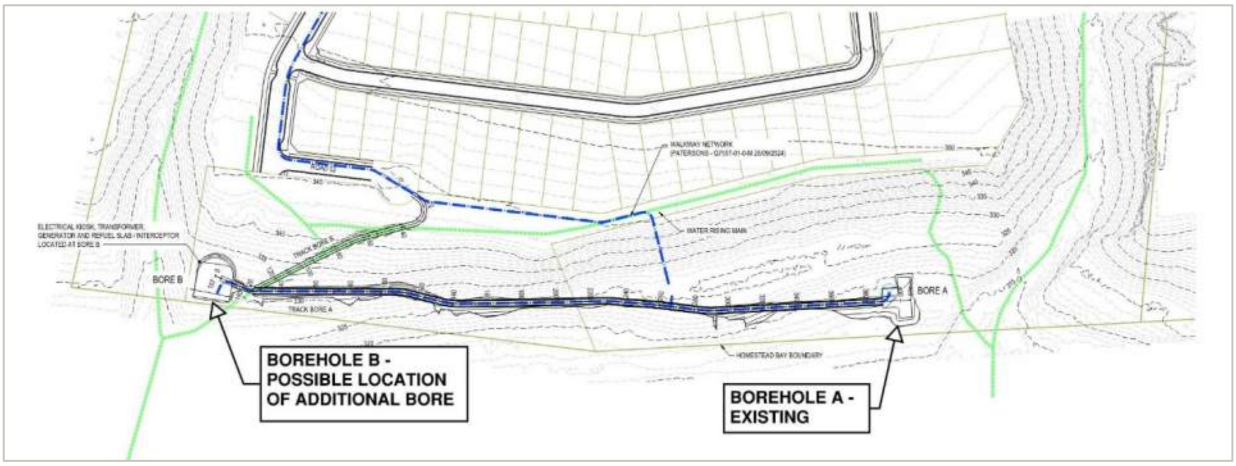

Figure 2-3: Bore A and Bore B Location & Access

Figure 2-4 shows a concept of a building that will house the borehead, manifold and switchboard.

Figure 2-4: Borehead and Manifold Building

2.7 Water Treatment

Raw water quality was assessed by Stantec New Zealand Ltd to inform the treatment process concept design to comply with the Drinking Water Standard for New Zealand (DWSNZ) 2022 and the Drinking Water Quality Assurance Rules (DWQAR) 2022, Revised 2024[53]. Details are contained in their technical note.

Table 2-4: Results of Raw Water Testing and Treatment Requirements

Parameter	Sampling Result	Treatment Requirements
Turbidity	Three out of four samples were measured to be greater than the maximum acceptable value of 1 NTU to enable bacterial compliance with the DWQAR using chlorination (i.e., sodium hypochlorite, or chlorine gas).	This turbidity must be removed to provide compliant potable water
pH	All samples were measured to be greater than 9.0, and above the maximum value of 8.5 for compliance	The recommended treated water pH is 7.0 to 7.6. Treated water within this pH range enables compliance with the bacterial rules and results in a treated water with better chemical stability
Iron	Three out of four samples were measured to be greater than the aesthetic value It is suspected that the elevated turbidity is associated with iron.	Water with these levels of iron does comply with the DWSNZ. However, not removing it below the aesthetic value is expected to result in increased maintenance for equipment (e.g., UV reactor), accumulation of iron in the distribution network, and customer complaints due to coloured water events

[53] Stantec New Zealand Ltd, *Homestead Bay WTP – Overview,* Technical Note, (Dunedin: 16 December 2024), 2.

Parameter	Sampling Result	Treatment Requirements
Manganese	Three out of four samples were measured below the aesthetic value, but greater than 50% of the aesthetic value It is suspected that the manganese is contributing towards the elevated turbidity.	Water with these levels of manganese does comply with the DWSNZ. However, similar to iron, not removing it would result in increased maintenance for equipment (e.g., UV reactor), accumulation of iron in the distribution network, and customer complaints due to coloured water events
Alkalinity and Hardness	The alkalinity of the raw water is above the recommended minimum, but is considered low. Total hardness values were similar in magnitude to the alkalinity, suggesting that the water is soft.	An acid will be required to lower the pH to comply with the DWQAR. Due to the low alkalinity, there is a risk of producing a treated water with high corrosivity when acid is added. Careful management of both pH and alkalinity will be required at the WTP

The objectives of the water treatment process are as follows:

- Produce drinking water that complies with the DWSNZ and the DWQAR
- Provide bacterial and protozoal treatment.
- Produce drinking water that is acceptable to consumers (i.e., appearance, taste, odour).
- Produce drinking water that is biologically stable, chemically stable, and minimally corrosive.
- Enable the water supplier to fulfil their requirements and responsibilities as outlined in the Water Services Act 2021.

To achieve the above treatment objectives, the following treatment process has been proposed:[54]

- Pre-oxidation using chlorine (e.g., sodium hypochlorite or chlorine gas) to oxidise both iron and manganese to enable their removal.
- Greensand filtration to remove turbidity, iron, and manganese.
- UV disinfection to provide a 4-log protozoal treatment barrier.
- Chlorine dosing (e.g., sodium hypochlorite or chlorine gas) to achieve bacterial compliance.
- pH adjustment and alkalinity control to enable bacterial compliance and produce a chemically stable drinking water.
  - Dosing of both an acid (e.g., hydrochloric acid or carbon dioxide) and base (e.g., soda ash) are required.
- Treated Water Reservoirs provide chlorine contact time (C.t) and enable bacterial compliance.
- Backwash Waste Tank with Supernatant Recycle.
  - The tank provides time for the backwash waste solids to settle. The clarified water, or supernatant, at the top of the tank can then be recycled to the start of the treatment process. This increases the production yield of the plant and reduces the volume of waste that is directed to the Wastewater Treatment Plant.

2.8 Service Reservoir

Service reservoirs are used to balance water inflow with demand outflows during peak periods, provide adequate storage for firefighting purposes and include a certain volume for emergency works. They are generally preferred over on-demand booster pump stations as they are unaffected by power failures and other events that can typically cripple them.

The proposed Homestead Bay reservoirs are located at the highest elevation on the northeastern part of the site. Their nominal location is shown in Figure 2-1.

[54] See a schematic of the treatment process on drawing 310104425-00-000-C0404.

They are selected and sized based on a number of criteria. Those relevant to Homestead Bay include:

- QLDC Land Development and Subdivision Code of Practice (LDSCoP):[55]
  - "*Where reservoirs or pumping stations are required, reference shall be made to the TA for its specific requirements. WSA 03 contains design criteria for pumping stations and reservoirs*."
  - "*The water reticulation system shall be designed to comply with SNZ PAS 4509*."
- Unpublished Draft QLDC LDSCoP Addendum – Water Supply Reservoirs:
  - The minimum gross storage usable reservoir volume across each network shall be the greater of:
    - "*24 hours of Average Day Demand*
    - *12 hours of Peak Day Demand*
    - *6 hours of Average Day Demand plus the greatest firefighting storage requirement for the network as defined by SNZ PSA 4509:2008*"
- FENZ Code of Practice - SNZ PAS 4509:2008 requires:[56]
- "*Fire storage to be in addition to normal operational requirements, i.e., over and above Operational and Reserve Storage*".
- According the WSA03 Water Supply Code of Australia:[57]
  - Operating Storage = "…*shall cater for demands exceeding the maximum available inflow rate*."
  - Reserve Storage = "*Reserve storage shall cater for system component failure. Reserve storage capacity shall be determined from a risk assessment study of the supply zone / system, which considers characteristics of the zone / system to determine the risk (consequences and frequency) to water supply continuity and pressure in the event of a system component failure. Typically, reserve storage capacity should be taken to be equal to ⅓ peak day demand*"
  - Operating + Reserve Storage "*should be equal to a minimum of 8-24 h consumption at peak day demand, depending on the needs of the specific system. No net depletion of the operating capacity over the system design period is permitted e.g., where the design period is one day, no net depletion of the operating capacity over maximum day 24 hour period is permitted*"

The Basis of Design for the Homestead Bay Reservoir nominally consists of the following criteria:

- Capacity to balance peak diurnal and seasonal flows in excess of the maximum yield from the borefield.
- Reserve storage capacity to continue to operate while emergency repairs in the water supply system are carried out.
- Firefighting storage capacity.
- Enable sufficient contact time for chlorine dosing.[58]
- 1.1m freeboard for wave sloshing in a seismic event.
- Designed with an Importance Level 4 and design life 50 years
- Sized for demands from Homestead Bay only.

The Homestead Bay Reservoir is nominally sized in accordance with QLDCs unpublished Draft LDSCoP Addendum – Water Supply Reservoirs aforementioned. In addition, sufficient storage to enable sufficient chlorine contact time and freeboard to mitigate wave action during seismic events is included. A preliminary concept of the storage reservoirs and associated infrastructure required are shown in 310104425-00-000-C0406.

The required workable reservoir storage volume calculation is shown in Table 2-5. This is needed to balance diurnal and seasonal peaks. To this an additional volume will be added to achieve the required chlorine contact time and allowance in the freeboard for wave sloshing in a seismic event.

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[55] QLDC, Land Development and Subdivision Code of Practice. 2020, 174 & 177.
[56] SNZ PAS4509:2008. 59.
[57] Water Services Aussociation Australia. 2022. *Water Supply Code of Australia.* WSA03-2011. Version 3.2. Water Services Association of Australia Ltd. 88-89.
[58] Taumata Arowai. *Drinking Water Quality Assurance Rules (Revised 2024)*. 2022.

Table 2-5: Homestead Bay Gross Volume Requirement

Reservoir Size is the Greater of:[59]	ADD (m^3)	PDD (m^3)	Firefighting Storage Required (m^3)[60]	Required Total Working Volume (m^3)**	Required Working Volume for One Storage Reservoir (m^3)*
24 hours of Average Day Demand (ADD)	2,626			**2,626**	**1,313**
12 hours of Peak Day Demand (PDD)		2,626		**2,626**	**1,313**
6 hours of Average Day Demand (ADD) plus the greatest firefighting storage requirement for the network as defined by SNZ PAS 4509:2008	657		180	**837**	**418**

*Based on a total of two (2) storage reservoirs for Homestead Bay
**This is the total working storage volume required for Homestead Bay

A steel modular storage system comprising 700 mm high incremental panels and 29.032 m in diameter was considered. Two storage tanks with space for a third on the reservoir site is proposed. The total height from the bottom to the top of the tank is 4.1m plus 1.27 m high for a 5-degree pitched roof. See drawing 310104425-00-000-C0406 for details.

Table 2-6 shows the working volume and chlorine contact volume required and the actual volumes achieved with the steel modular storage system.

Table 2-6: Homestead Bay Indicative Sizing of Storage System for Potable Water

Chlorine Contact Volume Required (m^3)**	Working Volume Required (m^3)	Actual Chlorine Contact Volume (m^3)	Actual Working Volume (m^3)*
377	1313	377	1,476

*Based on 4.1m high tank – 0.2m dead space – 1.1m (sloshing) – 0.57 m (chlorine contact)
**Based on 0.57 height for chlorine contact

The main components of the water supply storage infrastructure will consist of the following:

- Two reservoirs with total usable gross storage capacity of a minimum 2,860m³.[61] Space is reserved on the site for a 3rd reservoir should RCL Ltd decide to expand the water supply system to accept other nearby developments.
- Independent inlet and outlet pipework.
- Scour and stormwater pipework with a stormwater culvert draining the surface runoff to discharge to the existing northern creek to the south of the reservoir site.
- Sunken reservoir site platform with external raised earth bund to shield the above ground infrastructure from view.[62]

[59] Mott MacDonald for QLDC. 15 October 2021. “Coneburn and Hanley’s Farm Water Supply Infrastructure Assessment 2021 (v5)".
[60] Standards New Zealand. 2008. SNZ PAS4509:2008 New Zealand Fire Service Firefighting Water Supplies Code of Practice. Wellington: Standards New Zealand.
[61] See drawing 310104425-00-000-C0403.
[62] See drawing 310104425-00-000-C0403

- Access that enters the reservoir site. An overland flow path conveys excess water to the northern creek located immediately south of the reservoir site.[63]
- Gabion retaining wall is provided to steepen the slope within the sunken reservoir site to minimise bulk earthworks on the outside to maintain QLDC's requirements of minimum 1V:3H batter slope.[64] The relatively gentle batter slope will enable grass mowing and landscaping (including plantings).
- Vehicle access, power and communications link between the storage reservoirs and WTP will be provided.
- A pipe easement leading to the site. It will encompass the access track, rising and falling mains, power and telecommunications.

2.9 Water Transfer and Distribution System

2.9.1 Introduction

The elements of a typical water supply Transfer System are shown in Figure 2-5.

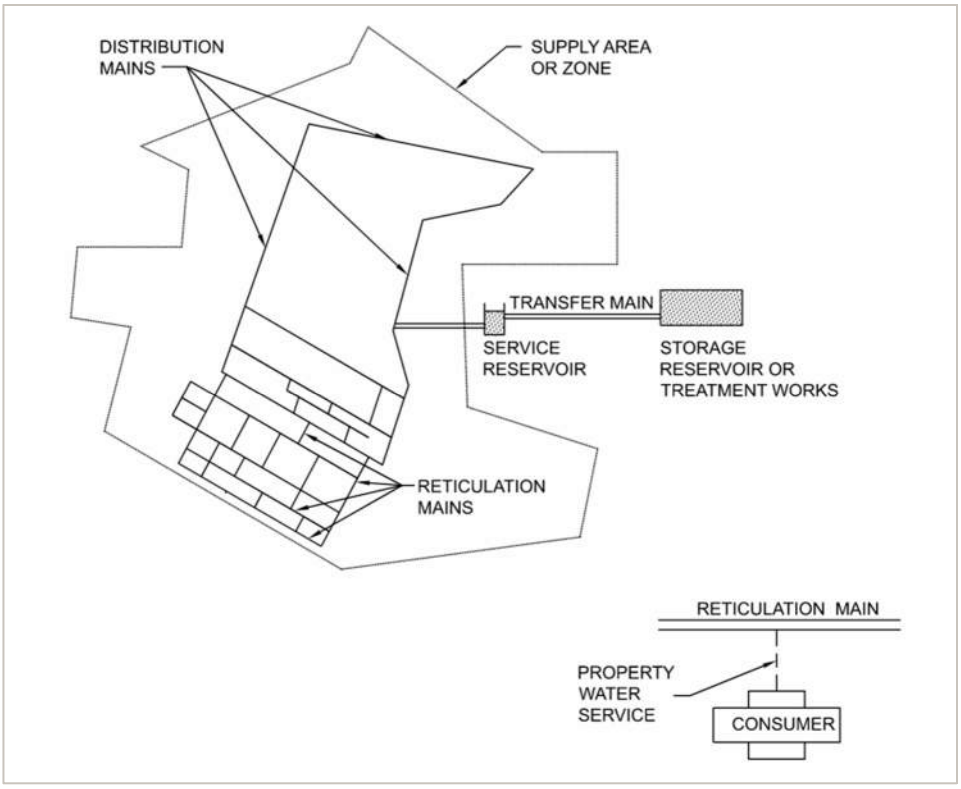

Figure 2-5: Components of a Typical Water Supply System (source: WSA03)

Borefield to WTP

The following infrastructure elements are likely needed to convey raw water from the borefield to the Water Treatment Plant (WTP):

- Bore pumps will be sized to convey the Peak Daily Flow to the WTP.
- Preliminary bore pump selection indicates it is feasible to pump from the borefield to the WTP. However, should future phases prove otherwise, an interstage booster pumpset located at a suitable elevation could be employed to boost water to the WTP.

[63] See drawing 310104425-00-000-C0403
[64] See drawing 310104425-00-000-C0403

- Rising main connecting the infrastructure.
- Surge anticipating devices will be considered based on a detailed surge analysis that will be undertaken during detailed design.
- Ancillaries, including air valve’s, scour points, isolation valves will be needed.

Homestead Bay Transfer Main

A dedicated 300mm ID transfer main from the reservoirs to the reticulated network is required.

A 150mm ID transfer main runs from gravity zone 1 to gravity zone 2 through the pumped zone.

2.10 Distribution and Reticulation System

The distribution and reticulation system were modelled to comply with QLDCs LDSCoP. They nominally include a system of principal and ridermains that are connected to appropriately sized transfer mains strategically located throughout the development to ensure compliance.

2.11 Pressure Management System

2.11.1 Introduction

Higher lot elevations will be served by a booster pump station where it's not feasible to service under gravity. The nominal location is shown in Figure 2-6. The concept consists of two gravity fed zones (northwestern and southwestern parts of the development) separated by a boosted zone.

The boundaries of the boosted zone were designed to minimise the length of dead ends while meeting pressure criteria for all lots.

There are three pressure zones, two are fed under gravity and the third is a pumped zone. However, due to the topography the high-level Pumped Zone bisects the lower-lying Gravity Zones. The lower-lying residential lots and commercial areas to the northwest are Gravity Zone 1, which are supplied direct from the reservoir. The lower-lying residential lots at the southwestern end of the development are Gravity Zone 2, and are supplied by Gravity Zone 1 through a gravity main which passes through the higher elevation Pumped Zone. See Figure 2-6 for a layout of the Boosted Zone, Gravity Zone 1, Gravity Zone 2 and key distribution infrastructure.

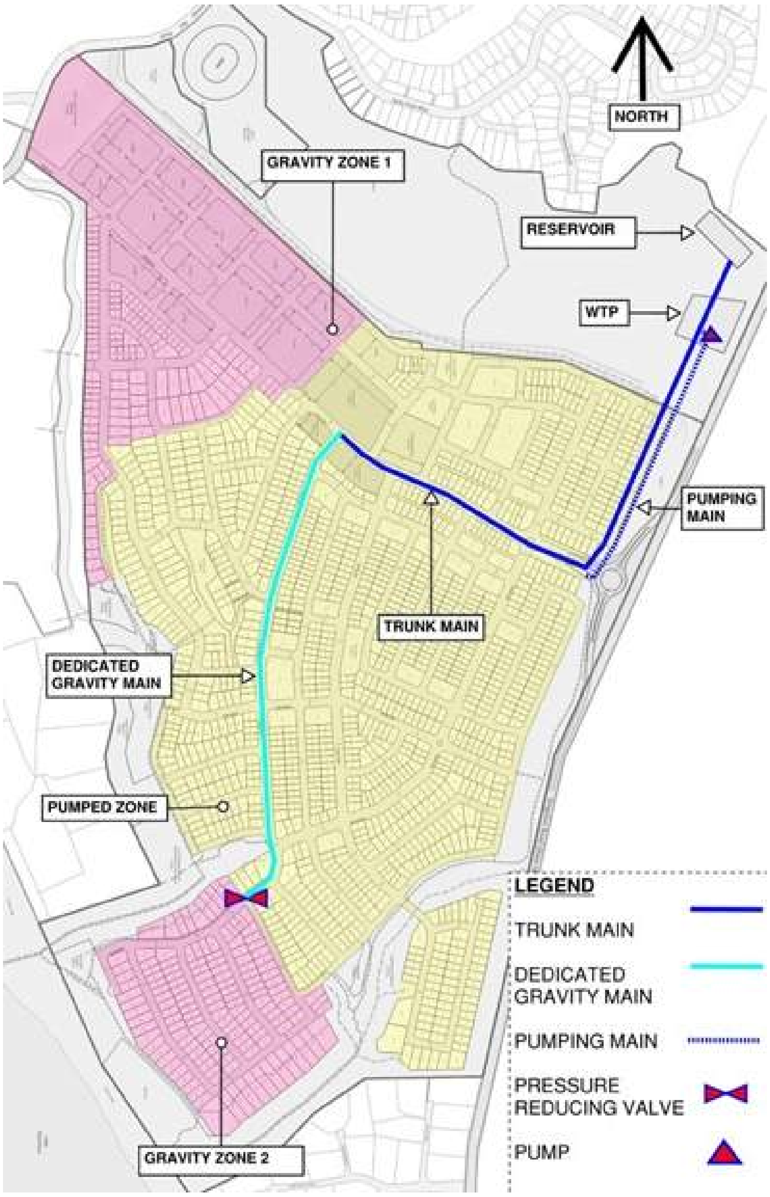

Figure 2-6: Layout of the Boosted Zone, Gravity Zone 1 and Gravity Zone 2

2.11.2 Staging

Developments of the scale of Homestead Bay are typically constructed in stages as the market dictates. Homestead Bay is no different.

Staging is likely to occur organically with lot's closest to the headworks, surrounding bulk conveyance infrastructure and requiring the least infrastructure, constructed first. It is anticipated that the higher elevated eastern lots bounding SH6 will form part of the early stages. They are part of the Pumped Zone, and the infrastructure required will initially consist of borefield A (including rising mains and potential inline booster station); WTP appropriately sized for the early stages, the first reservoir with associated distribution pipework; falling mains; booster pump station and relevant reticulated network.

The lower lying Gravity Zone 2 will follow and connect to the Pumped Zone in the short term, but be replaced with a connection to Gravity Zone 2 once it has been construction (discussion in Section 2.11.3)

Headworks and bulk conveyance infrastructure will be staged where practicable to suit lower demands during early phases keeping the network and associated equipment within acceptable performance envelopes. The capacity of bulk infrastructure will increase as demand increases with more stages connecting.

2.11.3 Pressure Regulation

Booster Pump Stations

One booster pump station is needed to service the higher elevated lots to QLDCs LoS. The nominal location of the booster pump station is shown in Figure 2-6.

The boosted zone is shown in the same figure and bisects the two gravity zones. This is a consequence of the topography.

Pressure Management Devices

A Pressure Reducing Valve (PRV) is needed during the earlier stages of Homestead Bay to serve Gravity Zone 2 as it will be supplied by the Pumped Zone before Gravity Zone 1 is built — Gravity Zone 2 is fed from Gravity Zone 1 in the long-term.

Figure 2-7 shows how the Pumped Zone feeds Gravity Zone 1 in the short-term.

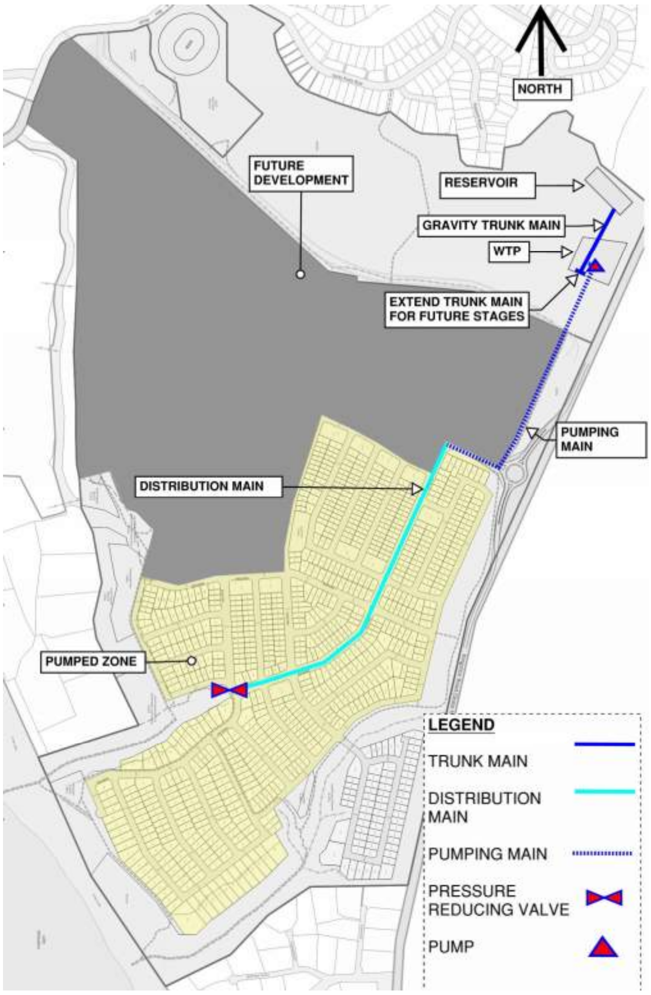

Figure 2-7: Pumped Zone Feeds Gravity Zone 1 in the Short-term

2.12 Water Modelling

Modelling has identified the requirements for three separate pressure zones in the water network — two are fed under gravity direct from the storage reservoirs as discussed in Section 2.11.1, so the static head from the top water level in the reservoir is common and available to both zones without any pressure management devices between them. The biggest zone is the Pumped Zone with relatively high elevations. Due to the likely staging, much of this zone is to be built first (compare Figure 2-6 and Figure 2-7). Gravity Zone 1 will feed Gravity Zone 2 in the long term so; it will initially need to be supplied by the Pumped Zone as it is likely part of the earlier stages. The preference is to install a PRV which should be set to maintain a minimum pressure at the downstream connection. When the development is complete, a dedicated 150 mm gravity main will connect Gravity Zone 1 to Gravity Zone 2 down Road 03. After the connection is complete the PRV can then be kept as backup in case the main gravity feed is compromised in the future. This gravity main will run through the Pumped Zone but will not be connected to this zone.

FW2 Fire Classification can be achieved throughout the network and FW3 (equivalent to FW2 plus 25 L/s Sprinkler Demand) is available to commercial land-uses and multi-level apartments.

2.13 Water Supply System Assessment

2.13.1 Summary of Infrastructure

Homestead Bay will be serviced by a new WSS. The key infrastructure that forms part of the proposed WSS is likely to include:

- A new borefield adjacent to Lake Wakatipu (direct lake take is not favoured because of previous problems with lake-snow). The borefield will include a minimum of two bores, Bore A and Bore B, which are expected to operate on a duty/ assist arrangement.
- A rising main and possible inline booster pump station to convey flows from the borefield to the WTP.
- A new water treatment plant to treat raw water to the New Zealand Drinking Water Standard (NZDWS). The proposed treatment process involves:
  - Pre-oxidation.
  - Greensand Plus® filtration.
  - UV disinfection.
  - Finished water chlorination.
  - pH adjustment and alkalinity control.
  - Backwash waste tank and supernatant recycle.
  - Chlorine contact is achieved in the proposed storage reservoirs.
- Two new treated water storage reservoirs with a total minimum usable gross storage volume capacity of 2,860 m^3. Space will be reserved for a third reservoir should RCL Ltd decide to expand the water supply system to accept other nearby developments.
- Booster pump station to service lots at higher elevations.
- A new transfer main to the distribution and reticulation system.
- New distribution and reticulation system

2.13.2 Conclusions

Homestead Bay is not currently serviced by any of QLDCs WSS. Nevertheless, it has limited remaining capacity that is reserved for previously consented developments. The closest WSS is at Jacks Point, however it too has limited capacity that is reserved exclusively for that development. Neither therefore will currently accept demands from Homestead Bay without significant upgrades and capital investment.

It has been shown that Homestead Bay can be served by a new WSS, via bores, treated on site and distributed via a system of storage reservoirs and booster pumps in compliance with local and national regulatory requirements with agreed amendments as appropriate. Subject to the results of further investigations, the WSS could also be used to supply other areas outside Homestead Bay.

3 Wastewater

3.1 Introduction

A key element of a healthy community is access to a safe wastewater collection and disposal system at an appropriate level of service for the activities being served. Consequently, potential sources that would meet these fundamental requirements for Homestead Bay were investigated and are reported in the subsequent sections.

[65]This wastewater section can be read in conjunction with Details of the Wastewater Treatment preliminary design for this consent application is discussed in detail in the in Lowe Environmental Impact (LEI) Consent Design Report in Appendix C[67]. That report has a particular focus on the levels of treatment and land disposal strategy. The following sections are primarily from this report with some minor updates to provide additional information for clarity. We note that the wastewater values in this report differ from those presented by LEI, for reasons explained below. The difference has no impact on the proposed servicing strategy.

The LEI report has been peer reviewed by Reeftide Environmental & Projects. This peer review is provided in Appendix D.

3.2 Regulatory Requirements

As discussed in Section 1.3, New Zealand's Fast-track Approvals Bill plans to make additional land development and housing available for thousands of new dwellings in New Zealand, and Homestead Bay is one of them and is therefore included in Schedule 2 of the Act.[66]

The Wastewater Scheme (WWS) serving Homestead Bay will comply with the following guidance and regulatory documents:

- Queenstown Lakes District Council *Land Development and Subdivision Code of Practice.* 2020.
- New Zealand Standard 4404:2010 Land Development and Subdivision Infrastructure.
- New Zealand Standard AS/NZS 1547:2012. On-site Domestic Wastewater Management.
- *New Zealand Guidelines for Utilisation of Sewage Effluent on Land*.[67]
- Regional Plan: Water for Otago.[68]

3.3 Servicing Strategy

The use, by Homestead Bay, of two nearby existing Wastewater Transfers Systems and Treatment Facilities were explored. The first is owned and operated by QLDC and the second by Jacks Point. However, a new Wastewater Servicing Scheme was preferred for the reasons discussed in the following sections.

There is sufficient grade to use primarily a gravity Wastewater Reticulation Network at Homestead Bay. A pressure sewer system, however, is proposed for the southeastern most part of the development as a gravity network will be excessively deep to breach the deep gully between it and the rest of the development. There may be other locations within the development where, through detailed design, it is concluded that low pressure sewer is preferred, and flexibility is sought in the conditions of consent.

Most of Homestead Bay will gravitate via the Reticulated Network to two large pump stations at low

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[66] Fast-track Approvals Bill. (2024), https://www.legislation.govt.nz/bill/government/2024/0031/39.0/LMS943195.html. Schedule 2.
[66] Fast-track Approvals Bill. (2024), https://www.legislation.govt.nz/bill/government/2024/0031/39.0/LMS943195.html. Schedule 2.
[67] New Zealand Land Treatment Collective (NZLTC). 2000. *New Zealand Guidelines for Utilisation of Sewage Effluent on Land*. New Zealand Land Treatment Collective and Forest Research. Rotorua, New Zealand.
[68] Regional Plan: Water for Otago - Otago Regional Council Updated to 3 September 2022 ISBN 978-0-908324-83-5

points on site. They will pump to the WWTP and on to the Low-Pressure Effluent Disposal System (LPED) located on multiple Land Treatment Areas (LTA) spread around the development. A smaller third pump station, located at the midpoint on the western edge of the development is required to lift wastewater from a few isolated low-lying lots to the nearest gravity network.

Other aspects considered are the implications of low flows during the early stages of the development. These include odour and septicity. They become an issue when wastewater is retained in the system for typically longer than eight hours. These will be solved by either supplementing low wastewater flows with fresh water or using odour control devices for the treatment of the gaseous phase.

The proposed servicing strategy for Homestead Bay is discussed in more detail in the following sections.

3.4 Wastewater Generation

Wastewater generation is based on QLDC's LDSCoP. The following design parameters for residential subdivisions have been used.[69]

- Commercial Usage Type is considered to be "low" as defined in table 5.1 of QLDCs LDSCoP and assumes operation is a maximum of 12 hours/day.[70]
- Wastewater generation of 250 litres per person per day for residential land uses.
- Three (3) persons per lot.
- Average Dry Weather Flow (ADWF) in litres per second is calculated by summing each land-use whose Unit Base Flowrate is multiplied by the number of Units and persons per lot (if applicable). This provides an instantaneous inflow averaged over 24 hours.
- Peak Dry Weather Flow (PDWF) in litres per second is calculated by summing each land-use whose ADWF is multiplied by a Diurnal Peaking Factor of 2.5.[71] This is an instantaneous flowrate that accounts for diurnal flow variations during an ordinary day.
- Peak Wet Weather Flow (PWWF) in litres per second is calculated by summing each land-use whose PDWF is multiplied by a Wet Weather Peaking Factor of 2.0.[72] This is the maximum instantaneous flow with dilution from inflow and infiltration occurring at the same time as PDWF. This is an instantaneous flowrate that accounts for diurnal flow variations during an ordinary day plus flows from rainfall dependant Inflow and Infiltration.
- Average Wet Weather Flow (AWWF) in litres per second is calculated by summing each land-use whose ADWF is multiplied by a Peaking Factor of 2 for rain dependant Inflow & Infiltration.[73] This provides an instantaneous inflow averaged over 24 hours.
- Peak Dry Weather Flow (PDWF) and Peak Wet Weather Flow (PWWF) for Commercial land-uses are the same as the ADWF as the base unit rate includes peaking factors, as per cl. 5.3.5.1(b) of QLDCs Code of Practice[74].

A primary school can also be accommodated if the Ministry of Education require.

The wastewater generated from Homestead Bay is shown in Table 3-1.

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[69] Queenstown Lakes District Council. *Land Development and Subdivision Code of Practice*. 2020. 140.
[70] Queenstown Lakes District Council. *Land Development and Subdivision Code of Practice*. 2020. 141.
[71] Queenstown Lakes District Council. *Land Development and Subdivision Code of Practice*. 2020. 140.
[72] Queenstown Lakes District Council. *Land Development and Subdivision Code of Practice*. 2020. 140.
[73] Queenstown Lakes District Council. *Land Development and Subdivision Code of Practice*. 2020. 140.
[74] Queenstown Lakes District Council. *Land Development and Subdivision Code of Practice*. 2020. https://www.qldc.govt.nz/media/s21al2gz/2020-qldc-land-development-and-subdivision-code-of-practice.pdf.

Table 3-1: Wastewater Generated from Homestead Bay

Activity	Number of Units	Unit Base Flowrate	Occupancy Per Unit	ADWF		AWWF		PDWF	PWWF
				L/s	Vol (m^3/d)	(L/s) PF= 2.0	Vol (m^3/d)	(L/s) PF= 2.5	(L/s) PF= 5.0
Standard Density Lots	1438 DUE	250 L/p/d	3 p/Lot	12.48	1078.50	24.97	2157.00	31.21	62.41
Medium Density Lots	203 DUE	250 L/p/d	3 p/Lot	1.76	152.25	3.52	304.50	4.41	8.81
High Density Super Lots	890 DUE	250 L/p/d	3 p/Lot	7.73	667.50	15.45	1335.00	19.31	38.63
Commercial	2.50 ha	0.40 L/s/ha	N/A	1.00	43.20	1.00	43.20	1.00	1.00
			Total	**22.97**	**1941.45**	**44.94**	**3839.70**	**55.93**	**110.85**

It is noted that the LEI Consent Design Report considered a different development scenario, based on advice at the time from the Applicant. This resulted in a total Average Daily Flow of 2005m^3 being estimated. The applicant has decided to continue to seek consent for this slightly higher flow to retain some flexibility to accept additional development (for example from neighbouring land or from community facilities).

3.5 Existing Infrastructure

The closest connection to QLDCs Wastewater Servicing Scheme is at the Hanley's Farm Wastewater Pump Station, which is shown in Figure 3-1.[75] Also shown is the main infrastructure servicing the Queenstown Southern Corridor (QSC), Kelvin Heights and Frankton.

[75] BECA, *Hanley's Farm Wastewater Pump Station Sizing and Network Planning for Interim Servicing of the Southern Corridor*, 27 August 2021.

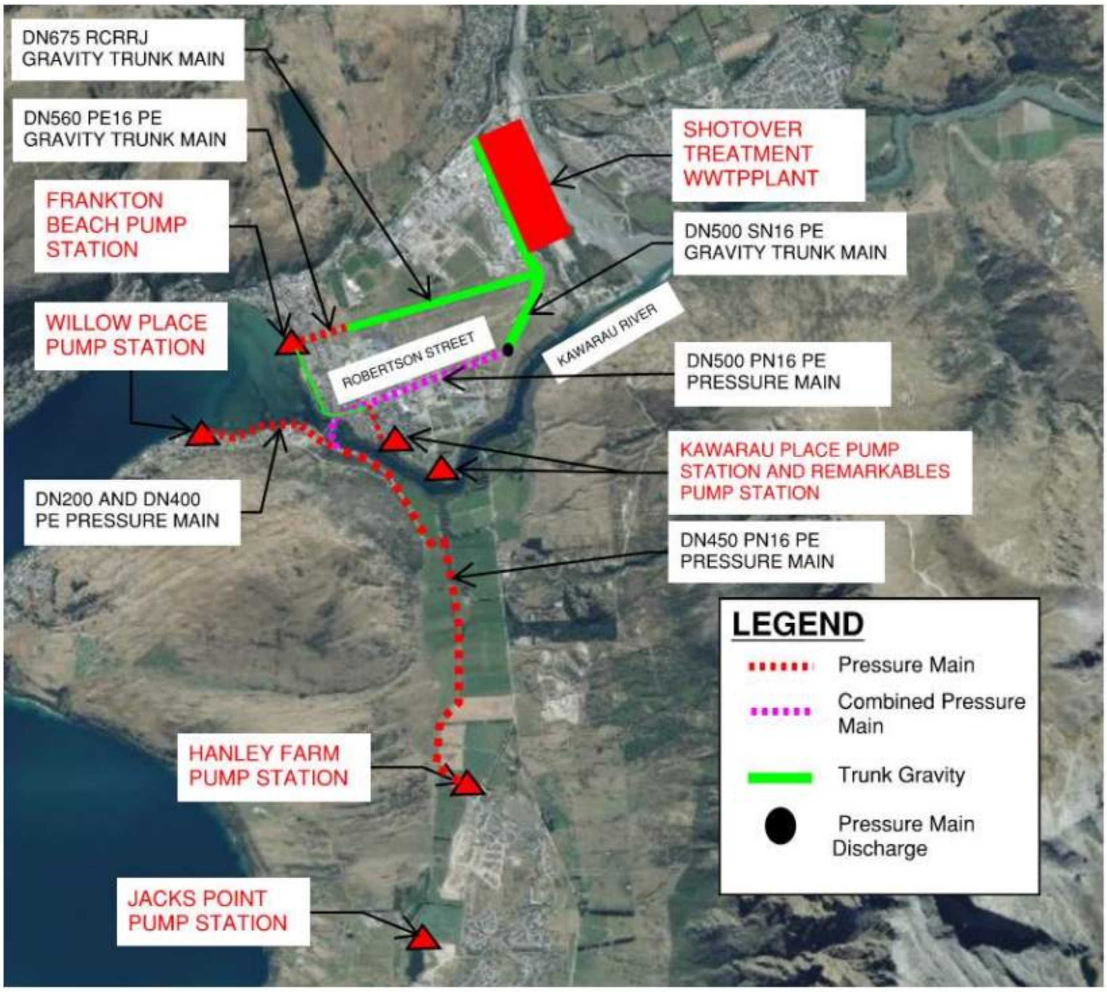

Figure 3-1: Existing QLDC Bulk Conveyance Wastewater Infrastructure in the QSC

The existing QLDC Wastewater Transfer System currently conveys wastewater from consented developments in the QSC, and there are several future developments that have shown an interest in benefiting from any remaining capacity . Notwithstanding any spare capacity, a significant increase in the capacity of the system would be required for Homestead Bay to connect. The following key elements of the Transfer System currently occurs:

- Jacks Point Village and the southern end of Hanley's Farm discharge into the Jacks Point Village Wastewater Pump Station. Wastewater is then pumped to a receiving manhole in Hanley's Farm and conveyed under gravity to the Hanley's Farm Wastewater Pump Station at the southern end of that development.
- The remaining areas of Hanley's Farm drain to this pump station.
- Hanley's Farm Wastewater Pump Station then pumps to a receiving manhole on the Frankton Flats and gravitates to the Shotover Wastewater Treatment Plan

At the request of the Applicant, from 2013 Stantec investigated options to increase the capacity of the network to accommodate estimated future development within the QSC. A long list of fifteen (15) options were evaluated to upgrade QLDCs Wastewater Transfer System to take flows from Homestead Bay and other future development areas. Five (5) of these were shortlisted as "potentially feasible" and progressed to a more intensive evaluation. However, each had their own inherent challenges including:

- Barriers to land purchase or approvals for use of existing support structures (such as supplementing the existing trunk mains on either the Historic Kawarau Falls Bridge or Kawarau River Bridge)
- Inherent patchwork nature of new infrastructure required to increase the capacity of the existing Wastewater Transfer System

- Overly complicated operational requirements — complex operations increases the risk of compromising the operational integrity and resilience of the Wastewater Transfer System.

Most significantly, QLDC expressed reservations around accepting more wastewater to the Shotover Treatment Plant given operational challenges there, which are not fully resolved at the time of writing. Council staff and elected member encouraged the Applicant to explore an independent system that would not contribute to that Plant.

Consequently, the proposed strategy to service Homestead Bay is to use a new Wastewater Servicing Scheme that includes new centralised on-site treatment and disposal.

3.6 Wastewater Treatment

Wastewater treatment that forms part of the Homestead Bay Wastewater Servicing Scheme is nominally presented in the schematic on drawing 310104425-00-000-C0350.

3.6.1 Basis of Design

3.6.2 Wastewater Influent Quality

Most wastewater generated will be from toilets, showers, laundry, and kitchen facilities and will, therefore, have the characteristics of conventional domestic sewage. Wastewater from some of the commercial areas, such as cafés or restaurants, will be stronger. The cafés and restaurants will require an additional grease trap.

3.6.3 Wastewater Effluent Quality

The effluent quality required from the WWTP will depend on the size of the selected area to be irrigated. It is proposed to be configured to produce an effluent with low total nitrogen and low total phosphorus to match the annual application rate of up to 220 kg N/ha/yr and a maximum average application depth of 8 mm/day. The WWTP treatment quality standards proposed for the final stage design are considered realistic to achieve on an annual average basis and are as follows:

- $cBOD_5$ – 20 mg/L
- TSS - 20 mg/L
- TN - 7.5 mg/L
- TP - 2.5 mg/L
- E.coli - 1000 MPN/100mL

3.6.4 Wastewater Treatment Plant

The WWTP will consist of the following components:

- Headworks including coarse and fine screens rated to PWWF.
- 1000m^3 bolted steel panel balance tank - to attenuate all wastewater flows above ADWF conditions.
- Activated sludge treatment system – see below for a discussion on this system.
- 488m^3 treated wastewater storage in the irrigation buffer tank for flow balancing to allow for batch discharge of wastewater to the LTA.
- Sludge dewatering centrifuge.
- Odour treatment system.
- The headworks and dewatering system shall be enclosed in a building to contain noise and odour.
- Some suitable activated sludge-type treatment systems have been considered, but others are available, including package plants or bespoke-designed plants.

The systems considered include the following and are summarised in Table 3-2;

1. Submerged Aerated Filter (SAF);
2. Sequence Batch Reactor (SBR);
3. Membrane Bioreactor (MBR); and
4. Membrane Aerated Biofilm Reactor (MABR).

3.6.4.1 Submerged Aerated Filter (SAF)

The SAF system is a form of the activated sludge process — a wastewater treatment process characterised by a suspended growth of biomass— usually with a floating media to enhance biofilm development and with the settlement of solids taking place within a clarifier.

Wastewater enters a recirculating (primarily anaerobic) chamber where oxidising bacteria break down suspended solids; the influent is also mixed with returned activated aerated sludge from the clarifying chamber. This mixing stimulates bacteria and enhances the digestion of solids. Following primary treatment, wastewater enters an aeration chamber that contains submerged media on “bioblocks” — bioblocks allow for an increased surface area. Treated wastewater passes from the aeration chamber to a clarifying chamber, where the remaining particles of suspended solids settle out of suspension. The suspended solids that sink to the bottom of the chamber are drawn back to the first primary chamber for further processing or removed for disposal off-site.

The SAF will be followed by further filtration (125 microns) and UV sterilisation to reduce pathogens.

3.6.4.2 Sequence Batch Reactor (SBR)

SBR is a form of activated sludge wastewater treatment. In a typical SBR process train, influent wastewater generally passes through screens and grit removal before the SBR. The wastewater then enters a partially filled reactor containing biomass, which is acclimated to the wastewater constituents during preceding cycles. Once the reactor is full, it behaves like a conventional activated sludge system but without a continuous influent or wastewater flow. The aeration and mixing are discontinued after the biological reactions are complete, the biomass settles, and the treated supernatant is removed. Excess biomass is wasted at any time during the cycle. Frequent biomass wasting holds the mass ratio of influent substrate to biomass nearly constant from cycle to cycle.

SBR technology generally requires a higher level of operator assistance to ensure the system is maintained and operating to a high standard; otherwise, it can be prone to failure and poor wastewater quality. SBRs are an aerated technology and, therefore, require a higher power input. They can reliably reduce nitrogen concentrations to low levels. As a result of the high level of aerobic microbial activity, a large volume of sludge is produced, requiring management and disposal.

The SBRs will be followed by further filtration (125 micron) and UV sterilisation to reduce pathogens.

3.6.4.3 Membrane Bioreactor (MBR)

An MBR system is a combination of the activated sludge process (a wastewater treatment process characterised by a suspended growth of biomass) with a micro or ultra-filtration system that rejects particles above 0.1 – 0.4micron in size (which is smaller than an individual bacteria). MBRs have two basic configurations: (1) an integrated configuration that uses membranes immersed in the bioreactor, and (2) a recirculating configuration where the mixed liquor circulates through a membrane module situated outside the bioreactor.

The key benefits of MBR technology for this application include:

- Reliably high level of treatment achieved
- Very compact process
- Good at handling seasonal loads
- Good at treating high strength wastewater
- Physical barrier prevents bacteria entering the treated water
- Treated water is suitable for municipal reuse such as garden watering

The WWTP does not require further filtration or UV sterilisation.

3.6.4.4 Membrane Aerated Biofilm Reactor (MABR)

MABR is a modified activated sludge process, where the conversion of ammonia in raw wastewater to nitrate, known as nitrification is carried out in a compact and energy efficient manner. An MABR is characterized by submerged gas transfer membranes which provide air directly to a biofilm attached to the surface of the membrane. The gas transfer membrane allows for efficient oxygen transfer, applied directly to the biofilm carrying out the nitrification reaction.

In an MABR, the aeration membrane is not used to filter the water. Instead, it is used to provide oxygen-enriched air to the process biology, replacing the conventional fine bubble diffuser. In doing so the oxygen is introduced in the molecular, or ‘bubbleless’ form. This leads to highly efficient oxygen transfer, since the oxygen is no longer limited by diffusion from the inside of the air or gas bubble to the gas bubble surface and then across the surface to the surrounding water.

Because the membrane is being fed with molecular oxygen and is immersed in the tank being fed with the influent wastewater containing biodegradable organic matter, a biofilm forms on the membrane surface. An MABR is therefore an example of a ‘fixed film’ process - like a trickling filter or a moving bed bioreactor - as opposed to a purely suspended growth process (i.e. one based on activated sludge), as is the case for the MBR.

The MABR still has a mixed liquor of suspended particles, as with other fixed film processes such the moving bed bioreactor (MBBR), but at lower concentration than the MBR. Biological treatment is thus achieved both by the biofilm and by the suspended flocs.

As an MABR process is usually implemented as a part of a modified activated sludge process, it can often be used to improve the performance of other treatment processes that are based on the activated sludge process, such as SBR and MBR treatment plants. The major benefits of MABR treatment processes include:

- They are easily scalable and can be designed to be modular
- Extremely robust when faced with fluctuating flows and loads
- Excellent performance at low temperatures.
- They typically produce much less waste biological sludge due to the high efficiency of the biofilm requiring less biology to achieve the same rate of nitrification compared to other conventional treatment systems.
- Lower sludge production means lower operational, and disposal costs associated with sludge handling.
- By virtue of their energy efficient design, they offer better environmental performance when measured against other conventional treatment options.

The MABR will be followed by further filtration (125 micron) and UV sterilisation to reduce pathogens.

3.6.4.5 Summary of Communal System Options

Table 3-2 provides a qualitative assessment of the treatment system parameters. However, weightings that are important to Homestead Bay will be assigned to each to select the preferred system in future phases of the development. The main focus for consenting purposes is to show that the concept is feasible, and to establish the key performance standards for a future plant which will be confirmed through consent conditions.

Activated sludge is an appropriate technology for Homestead Bay that generally requires a higher level of operator assistance to ensure the system is maintained and operating to a high standard. SBRs and MBRs are aerated technologies and, therefore, require a higher power input. As a result of the high level of aerobic microbial activity and the desire for Phosphorus removal, a large volume of sludge is produced, requiring management and disposal.

Table 3-2: Summary of Wastewater Treatment Options

Parameter	SAF	SBR	MBR	MABR
Capital expenditure	Moderate and high as capacity increases	Moderate	Moderate and high as capacity increases	Moderate and high as capacity increases
Requires separate Tertiary Filtration	Yes	Yes	No	Yes
Requires UV disinfection	Yes	Yes	No	Yes
Running costs	Moderate to High	Moderate to High	High	Moderate
Power requirement	Moderate	Moderate	Moderate	Low
Modularity/staging	Moderate	Moderate	Good	Good
Maintenance Requirement	Potentially High	Potentially High	Potentially High	Potentially High
Sludge production	Moderate to High	Moderate to High	Moderate	Moderate
Suitable for intermittent flow regimes	Moderate	Moderate	Low	Low
Remote servicing and trouble shooting	Yes	Unlikely	Unlikely	Unlikely
Operational Complexity	Moderate	Moderate	High	High
Reliability	Moderate	Moderate to High	Moderate to High	Moderate to High
Wastewater treatment stability	Moderate	Good	High	High

3.6.4.6 Conclusion

The preferred WWTP system and supplier are to be determined following confirmation of the consented treatment standard, the LTA area, the development rate of wastewater flows (i.e., how modular the WWTP needs to be) and further evaluation attributes in addition to those listed in Table 3-2.

A key requirement for this WWTP is that it shall have a low risk of odour; the treatment processes can be adjusted to meet possible future increases in treatment standards; the treatment plant can be readily staged and scaled to suit growth; proven treatment technology with activated sludge treatment systems being used to treat wastewater from approximately 70% of New Zealand reticulated sewer networks population.

It is not proposed that the resource consent specify the exact type of WWTP to be selected. Confirmation of the technology will require discussions with potential suppliers and operator, which will traverse capability and commercial matters. It is considered prudent to maintain flexibility in the consent conditions to allow the best system to be procured. It is, however, understood that as a basis for initial high-level design, at the time of writing, the preferred technology is a combined MABR/MBR activated sludge process.

The combined MABR/MBR system provides improved performance, a smaller plant footprint, and improved OPEX costs.

The WWTP treatment plant and LTA are considered an integrated wastewater treatment train. LEI have provided scenarios for staging the WWTP capacity and LTA to demonstrate that there is flexibility within the system implementation during the early stages of the development.

3.7 Disposal – Land Treatment Area and Low-Pressure Effluent Disposal System

3.7.1 Key Aspects

LEI investigated the feasibility of discharging WWTP effluent to Land Treatment Areas (LTA). The investigations focused on the following aspects:

- Soils investigation to determine the maximum Design Irrigation Rate (DIR) for each soil type.
- Groundwater investigation to determine the water levels and ground transmissivity.
- Surface water catchment assessment to determine the impact on the environment.

The investigations concluded that an LTA consisting of a Low-Pressure Effluent Disposal System (LPED), also known as sub-surface dripper irrigation is an appropriate system in conjunction with the WWTP.

Further details of the investigation can be found in the LEI report in Appendix C.

Multiple LTAs are proposed throughout Homestead Bay on suitable land as indicatively shown in Figure 3-2. The LTAs will be constructed at different stages of the development.

Figure 3-2: Indicative Land Treatment Areas Available in Homestead Bay

Within each LTA area, treated wastewater will be discharged at the required pressure into a number of the submains from the treated wastewater mainline. From the mainline, an automated valve will actuate before the water flows through the submain to each zone. The average zone size is 0.7 ha and contains approximately 7,000m of dripper line at 200 – 300mm depth. The zones will be grouped to allow dosing and rest periods to cycle around the LTA area.

Each of the zones considered will be based on a maximum DIR during dry weather conditions and during wet weather conditions. The daily volume will be partially stored in the irrigation buffer tank and based on the level of the treated water stored; irrigation will occur. The discharge rate will be automated based on the LTA zones next available for irrigation. At times of higher inflow, additional zones and pumps would be used to discharge the daily volume at a rate of up to 53 L/s.[76] The actual pump/s duty will vary based on the zones irrigating at any one time.

The soil types encountered and proposed DIRs confirm that the estimated ADWF for Homestead Bay can be discharged within the proposed 28.5 ha LTAs available at sufficiently low DIRs.[77]

3.7.2 Concept Design of LTA and LPED System

LEI undertook a preliminary design of the LTA and LPED system. The feasibility of the system was assessed by evaluating potential development scenarios whether dosing rates within each zone were reasonable, pump duties for the system and flushing of the mainline, submain and sub-surface drippers were appropriate.[78]

They show that irrigation of treated wastewater via an LPED system to LTAs by means of a sub-surface drip irrigation is feasible.

3.7.3 Conclusion

LEI has demonstrated in their investigation that there is a feasible treatment chain involving a WWTP and LTA discharge options that can be configured, flexibly implemented and managed to treat the development's wastewater to achieve a very small increase in current total losses of nutrients from the site at design flows, while being considerably (about 30%) lower than the total site loading calculated as being permitted under the Regional Plan: Water for Otago. Designing the wastewater treatment and discharge system to align with the expected nutrient losses from this landform enable social, economic, and cultural well-being of the QLDC community to be met while minimising adverse environmental effects.

Key conclusions include:

1. Sustainability: The system facilitates the beneficial reuse of water and nutrients. The staged development approach ensures flexibility as wastewater flows increase.
2. Nitrogen Management: The system manages total nitrogen (TN) leaching potential through a combination of optimised treatment at the WWTP and efficient land application.
3. Environmental Protection: Subsurface drip irrigation and conservative hydraulic loading rates minimise the risk of groundwater contamination and surface water impacts.
4. A detailed nutrient loss assessment and environmental effects study will be undertaken in future phases of the project.

Overall, the proposed WWTP and LTA design is intended to meet regulatory requirements while providing a robust and scalable wastewater treatment solution for Homestead Bay.

3.8 Wastewater Reticulation Network

It is anticipated that wastewater in Homestead Bay would be collected via a network of gravity pipes that convey flows to pump stations at low points in the development.

The topography in Homestead Bay is undulating but generally falls from SH6 towards Lake Wakatipu. This is conducive to the use of primarily a gravity Wastewater Reticulation Network. Where ground elevations dictate a pressure sewer system is proposed for the southeastern most part of the development, which is bisected from the rest of the development by a deep gully. There may be other

[76] LEI used a maximum AWWF 3973 m^3/d and maximum flowrate of 53 L/s. The AWWF is as defined in Section 3.4. The current design AWWF daily volume is 3839.70 m^3/d and flowrate of 44.4 L/s, which is lower than LEIs preliminary design flows.
[77] The LEI investigation used an ADWF of 2005 m^3/d. The current ADWF for Homestead Bay is estimated to be lower at 1941.45 m^3/d.
[78] LEI. 2025. *Homestead Bay Consent Level Design Wastewater Land Application*. 27-29

locations within the development where, through detailed design, it is concluded that low pressure sewer is preferred, and flexibility is sought in the conditions of consent.

A preliminary sizing of the network shows it is feasible to service Homestead Bay.

3.9 Wastewater Transfer System

3.9.1 Introduction

Homestead Bay collects wastewater from individual properties via their connection to the Wastewater Reticulated Network. The elements of Homestead Bay's Wastewater Transfer System consist of a combination of bulk conveyance infrastructure that conveys wastewater to the new wastewater treatment plant in the development.

3.9.2 Pump Stations

The gravity Reticulated Network falls primarily to two large pump stations at low points on site — known as Pump Station A and Pump Station B.[79,80] They in turn pump to the WWTP and on to the subsurface drip irrigation disposal system located in the Land Treatment Area (LTA). Each pump station will have nine hours emergency storage volume (at Dry Weather Flow conditions), standby pump facility and standby power generator to maintain system continuity during rising main failure, pump failure or power outages. The vertical alignment of the gravity wastewater network and associated emergency storage tanks will be refined during detailed design. A third, much smaller pump station located around midpoint on the western edge of the development is required to lift wastewater from a few isolated low-lying lots to the nearest gravity network. Servicing these lots via gravity resulted in very deep pipelines.[81] This is an example of an area where Low Pressure Sewer may be further considered in lieu of a pump station through detailed design.

It was decided to design two low points rather than one as the benefits include:

- Facilitates staging by having more than one WWPS as it mitigates odour and septicity issues as residence times are lower and self-cleansing velocities are more likely to be achieved.
- Results in two shallower WWPSs, rather than one very deep one, mitigating risks during construction and ongoing risk for maintenance and repair when the wet well is accessed.
- Avoids a very deep gravity reticulation network which would otherwise drain to a single low point.

3.10 Odour and Septicity

3.10.1 Wastewater Network

In the early stages of development, the number of property connections from Homestead Bay will be low. This may result in high retention times in the wastewater network with the potential for onset of septic conditions resulting in the generation of high levels of sulphides in the wastewater (liquid phase). The associated gas phase concentrations may be sufficient to potentially result in odour complaints, corrosion and pose a potential health and safety issue should anyone access the discharge points or be in close proximity to air vents when they relieve. It is considered that the onset of septic conditions is unlikely to occur when the development is complete and therefore any mitigation measures are likely to be of a temporary nature. In the early stages of the development, wastewater flows can be supplemented with water to reduce retention times. This could be achieved by making a temporary connection from the raw water rising main into the wastewater network. This will also have the added benefit of mobilising solids if self-cleansing velocities are not achieved. This will be considered further

[79] The location of pump station B is shown in drawing 310104425-00-000-C0353.
[80] The location of pump station A is shown in drawing 310104425-00-000-C0351.
[81] The location of pump station C is shown in drawing 310104425-00-000-C0355.

during detailed design.

3.10.2 Wastewater Treatment Plant

Elimination and minimisation of sources of offensive odour from the plant and discharge are of paramount importance due to its location within a residential development.

The WWTP treatment units and building have the potential to release offensive odours. The units / enclosed spaces that have the potential to generate offensive odours will be connected to a ventilation system. The ventilation system will be designed to achieve a prescribed number of air changes and maintain negative pressure to ensure that any odour is captured and transferred to an Odour Control Facility.

The Odour Control Facility will remove odour and other contaminants extracted from the Wastewater Treatment Plant. The odour sources produced by the Wastewater Treatment Plant and Odour Control Facility will be assessed following selection of the WWTP process during detailed design. If required, dispersion modelling can be undertaken in accordance with the 'Good Practice Guide for Assessing and Managing Odour' to demonstrate that the residual odour units at the boundary will not be objectionable.

For feasibility a high level of assessment was undertaken to confirm that there are available solutions to manage odour.

3.11 Network Assessment

Summary of Infrastructure

- Wastewater Reticulation Network – majority of the network will be via gravity with a pressure sewer system proposed for the southeastern most part of the development.
- Wastewater Transfer system – consisting of two medium sized pump stations and one small pump station with their associated pumping mains.
- 1000 m^3 buffer tank to control pass forward flow to the Wastewater Treatment Plant.
- New Wastewater Treatment Plant - At the time of consent submission the preferred treatment process is a combined MABR/MBR activated sludge process.
- Odour Control Facility.
- Discharge to Land Treatment Areas (LTA) by means of a Low-Pressure Effluent Disposal System (LPED)—proposed via a sub-surface dripper irrigation system.

3.12 Conclusions

The use, by Homestead Bay, of two nearby existing Wastewater Transfers Systems and Treatment Facilities were explored, but a new Wastewater Servicing Scheme was preferred

It has been shown that it is feasible to service Homestead Bay from this new scheme, which consists of a new Wastewater Reticulation Network, Wastewater Transfer System, Wastewater Treatment Plant, Odour Control Facility and disposal to LTAs via LPEDs.

4 Drainage and Flood Mitigation

4.1 Introduction

Natural risks, including flooding risk, present from both external and internal catchment areas relative to the proposed development area. Geosolve have identified and quantified external risks to the development and Stantec have assessed flooding risks inside the development area using desktop information such as rainfall and climate change information from NIWA. Stantec have then designed mitigation measures to manage those risks as described in this section of the report.

The Remarkables Mountain range is immediately upstream (and to the east) of the proposed Homestead Bay subdivision rising to above RL700m in the saddles and much higher on the tops. Three significant existing surface drainage channels issue from the Remarkables that approach the development and are shown in Figure 4-1. They are denoted as Northern Creek, Middle Creek and Southern Creek. A further channel, denoted as Southwestern Creek, originates from within the development area. The area immediately upstream of the development and SH6 is described geologically as a fan or combined fans and is described in more detail by Geosolve. Essentially, the fan indicates that the main creeks identified have the capacity to change their current alignment after large rainfall events and “sweep” across a wide front over the course of a long time period.

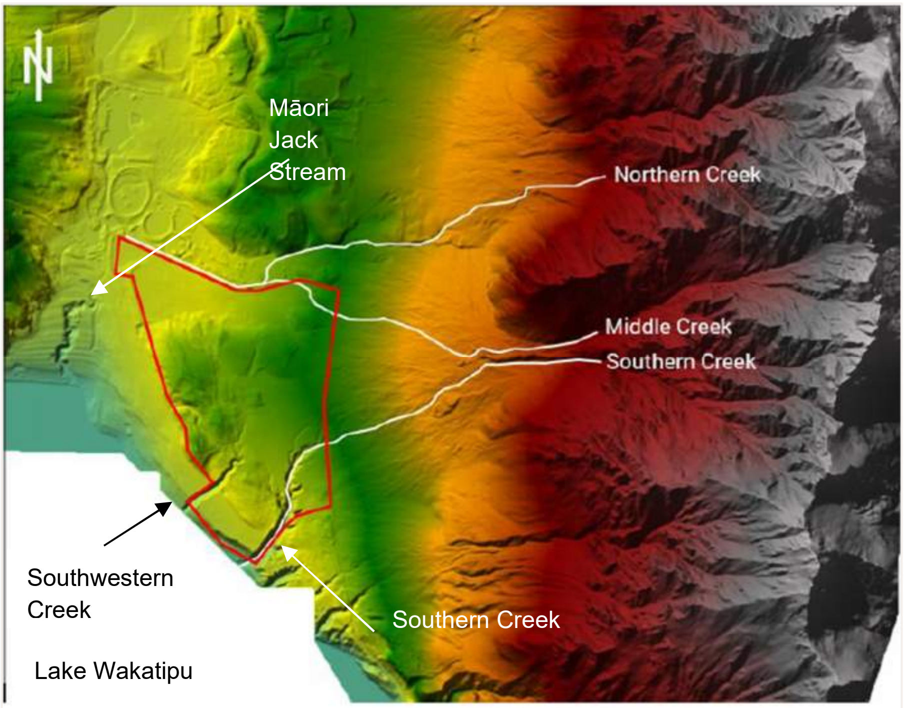

Figure 4-1: Pre-development Northern, Middle and Southern Creeks in Relation to the Proposed Urban Development Boundary

All three alpine creeks and the Middle Creek end up discharging into Lake Wakatipu outside the extents of the development area. The Southern Creek is the main creek that flows across the site and discharges directly to the lake. The Middle Creek flows cut across the top north-eastern part of the site and pass along the northern boundary of the development before joining Northern Creek flows and discharging to Māori Jack Stream beyond the Homestead Bay Road boundary that drains southwards to the lake. The Southwestern Creek drains a portion of the development area from the centre of the development and discharges directly to the lake. All these ultimately discharge into Lake Wakatipu.

The Homestead Bay subdivision area slopes from RL 400 m at the eastern boundary at SH6 to around RL 350 m at the lower levels of the developable areas. Two incised gullies at the lower southwestern corner of the subdivision (Southern Creek and Southwestern Creek) provide steep flowpaths to the

Lake level at approximately RL310m. Each gully is large in scale – over 20m deep by more than 40m top width, with the base level well below subdivision finished ground levels. These gullies provide good drainage outlets for new pipeline networks in the southern portion of the development.

The stormwater scheme plan for the development is shown in Figure 4-2 including the primary system (stormwater pipeline network) layout, contours and sub-catchment runoff areas draining to defined internal subdivision outlets. The main features of the drainage scheme plan are:

- Extensive primary stormwater network designed with pipeline hydraulics software.
- Extensive road network proposed as the secondary overland flowpath system.
- Two stormwater outlets to the north draining 64.5 hectares into the Northern Channel. The channel drains westward to the existing single 4m x 1m box culvert) under Homestead Bay Rd and then to Māori Jack Stream winding 2km to Lake Wakatipu. The Northern internal outlets are proposed with gross pollutant traps and attenuation storage basins.
- Five stormwater outlets to the south draining 89.4 hectares to Southern and Southwestern Creeks, leading to the Lake front. The outlets all have a similar configuration: gross pollutant trap, followed by steep pipeline to the gully floor, energy dissipation facility (such as impact basin), rock riprap apron transition to natural gully floor.
- One existing outlet draining to the west (identified as Catchment 7 Gully) will be maintained but will have a reduced post development catchment area due to subdivision earthworks and the primary drainage system.
- Several small pre-development subcatchments which drained westwards across the development boundary to Chief Reko Rd and to the Lakefront will be combined into the main post-development outlet locations. This will formalise the drainage from the post development land into the six controlled outlets and reduce flow demands at several smaller drainage lines downstream of the boundary.
- An alpine flood protection diversion channel/bund scheme is proposed along the eastern frontage of the development with State Highway 6. The diversion channel and bund formation are proposed over the full length of the eastern boundary of the development area parallel to SH6 to manage alpine flows from Northern, Middle and Southern Creeks which have the ability to alter alignment over time. The proposed roundabout entry to the development is at the high point of the SH6 frontage and splits the channel into northern and southern sections. The northern section of the diversion channel/bund drains to the proposed Northern Channel on the northern boundary to then drain west towards Homestead Bay Rd. The southern section of the diversion channel/bund will drain south to the existing Southern Creek and a residual section will drain along the southern boundary into the Southern Creek.

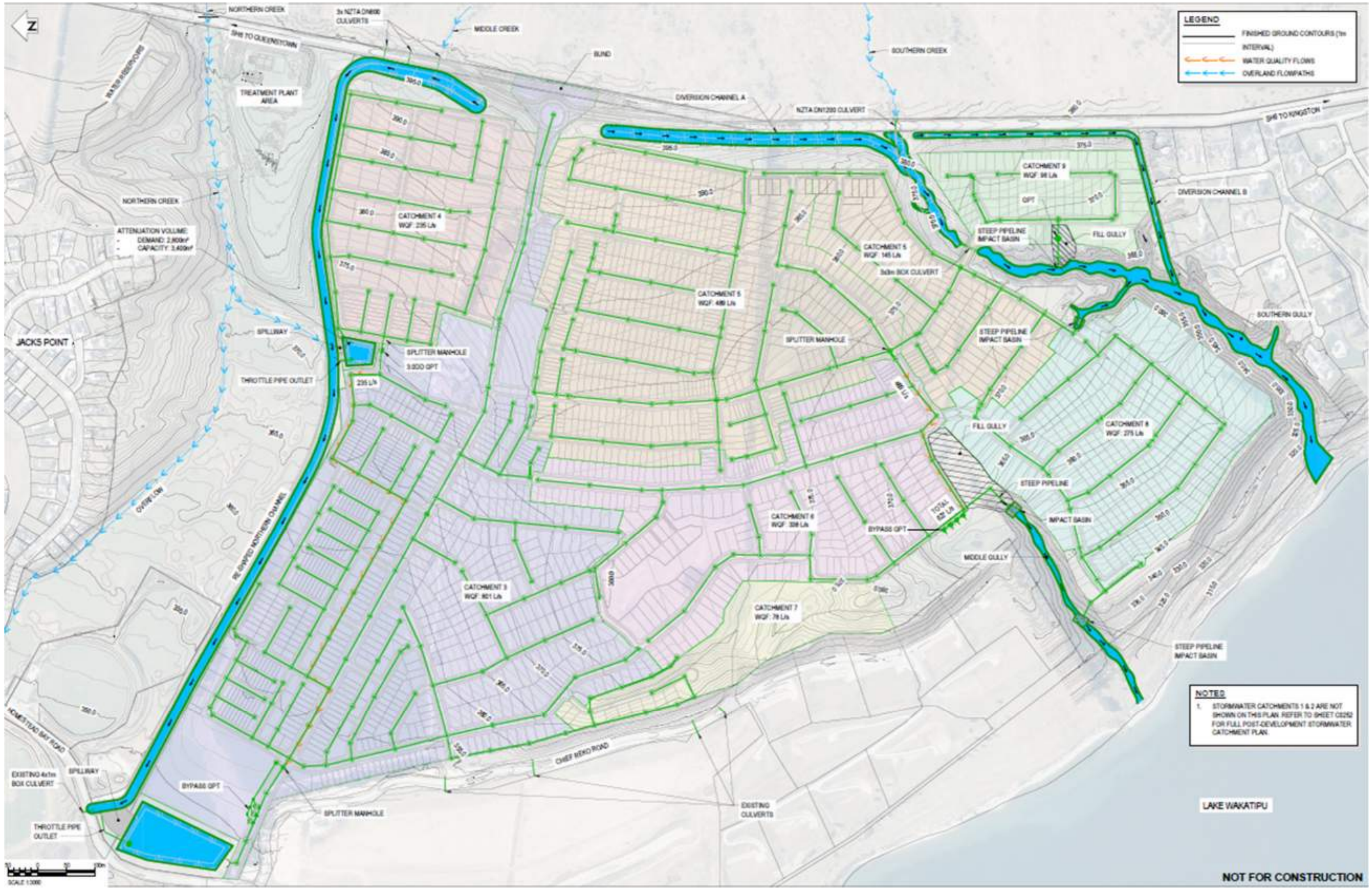

Figure 4-2: Stormwater Drainage Scheme Plan (reference to project drawing C0253)

4.2 Regulatory Requirements

The internal subdivision stormwater drainage will meet the requirements of QLDC COP section 4 Stormwater and this is described below in this report. General objectives from QLDC COP section 4.2.1. are listed below:

The stormwater system shall include provision for:

- A level of service to the TA's customers in accordance with the authority's policies
- Minimised adverse environmental and community impact
- Protection from potential adverse effects to aquatic ecosystems
- Compliance with environmental requirements
- Adequate system capacity to service the fully developed catchment
- Long service life with consideration of maintenance and life cycle cost
- Application of low impact design solutions
- Climate change

4.3 Servicing Strategy

Homestead Bay will deal with stormwater in two general strategies:

- *External origins*: For catchment runoff originating from outside the development, the proposal is to divert flows around the northern margins of the development in a diversion channel/bund and rock lined open channel (Middle Creek), and to divert flows to the south in a diversion channel/bund and rock lined open channel to the existing large capacity gully (Southern Creek). This hydrology and hydraulics risk is assessment has been carried out by Geosolve and coordinated with Stantec for mitigation.
- *Internal origins:* For catchment runoff originating from inside the development footprint, pipe and road provisions are proposed in line with QLDC COP practices as per normal subdivision standards. This includes the management of the effects of stormwater quantity and stormwater quality.

4.4 Catchment Runoff – Outside the Development

4.4.1 Flood Risk

Geosolve carried out flood risk modelling for Homestead Bay in 2024[82]. See Appendix E for details of their findings.

The Geosolve site-specific assessment consisted of undertaking a desktop study, review of historic aerial photography, interpretation of ground contours, site geomorphic mapping, numerical modelling for debris flow hazards using RAMMS and flooding hazards using HEC-RAS.

The contributing catchment presenting flood risk is a steep, west facing section of The Remarkables mountain range, with three distinct flow paths that could impact Homestead Bay originating from an approximately 2.4 km long (north-south) section of the hillside (Northern Creek, Middle Creek and Southern Creek shown in Figure 4-3.) The catchment extents for each of the three Creeks is presented in Figure 4-4 and Table 4-1.

[82] Geosolve, Natural Hazard Assessment Homestead Bay, March 2025.

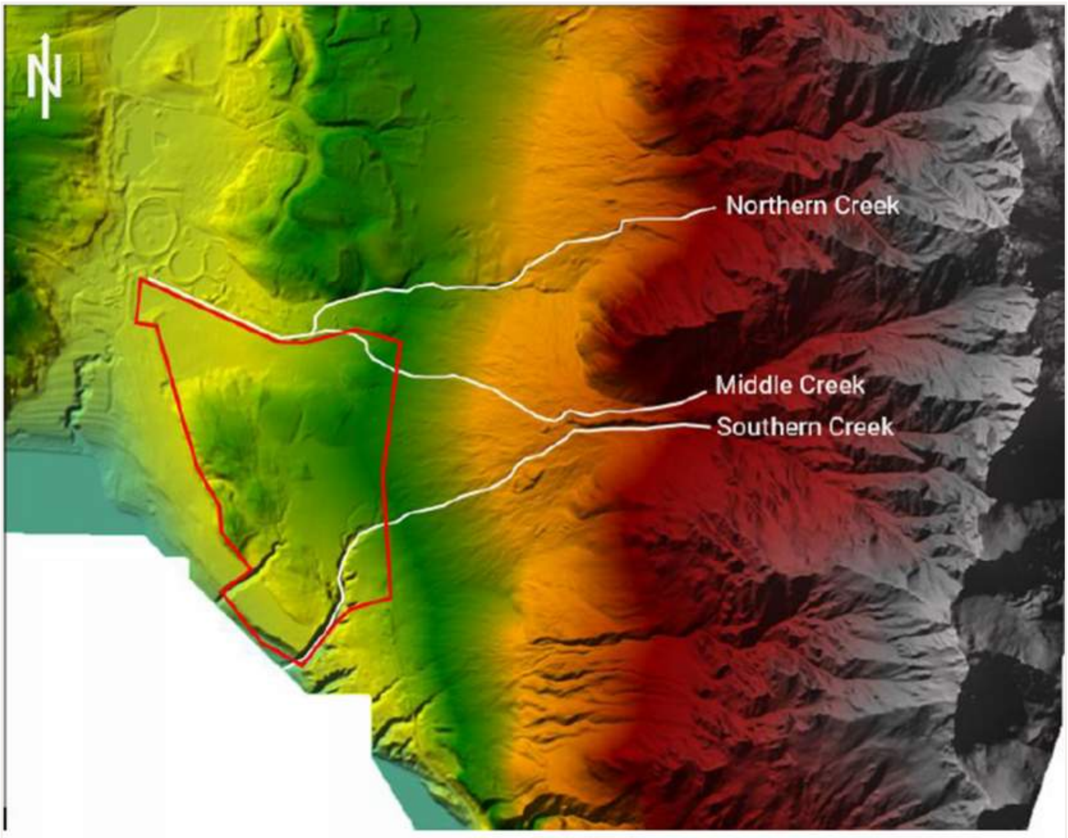

Figure 4-3: Northern, Middle and Southern Creeks in Relation to the Development Boundary

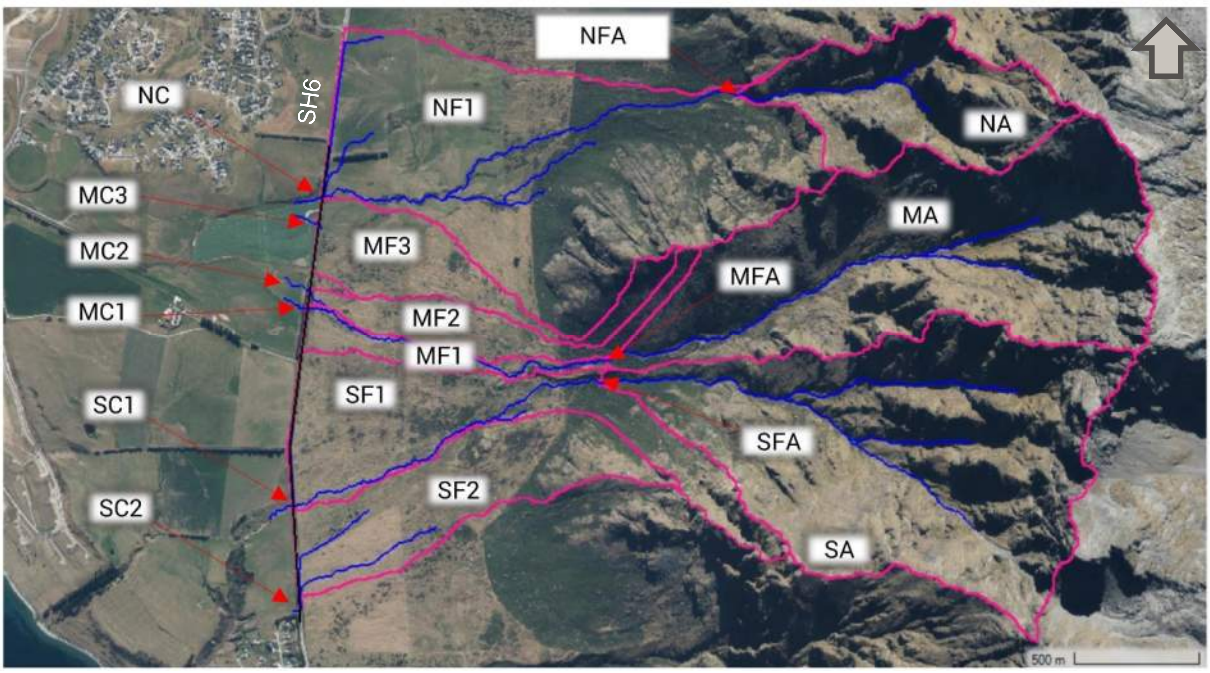

Figure 4-4: Alpine Creeks, Catchments and sub-catchments from The Remarkables draining to SH6 adjacent to the boundary of the Development, reference: Geosolve 2025

Table 4-1: Hydrological Feature Naming Convention

Label	Description
NA, MA, SA	Northern, Middle and Southern Alpine Catchments
NFA, MFA, SFA	Northern, Middle and Southern Fan Apex
NF, MF, SF	Northern, Middle and Southern Fan Catchments
NC, MC, SC	Northern, Middle and Southern Road Culverts (catchment outlets)

The process Geosolve followed to simulate a flood risk from a 1% AEP storm (QLDC requirements) to Homestead Bay is described in Section 4 of their report[83]. They used the Hydrologic Engineering Centre's River Analysis System computer program known as HEC-RAS.

The HEC-RAS model was developed by Geosolve in 2D mode from the outlets of the alpine gorges to the Lake. A hydrology runoff model was used to calculate upper catchment flow hydrographs. The HEC-RAS model calculates extents of flows (and peak flows) across the fan floodplains from all three creeks simultaneously as shown on Figure 4-5 below. The modelled area ranged from approximately 2 km upstream (east) of the site to the Lake shore on the western far side of the development and between neighbouring properties to the north and south. See Figure 4-6 and Figure 4-7 below for the HEC-RAS results before development, which have been overlain on aerial imagery for ease of reference.

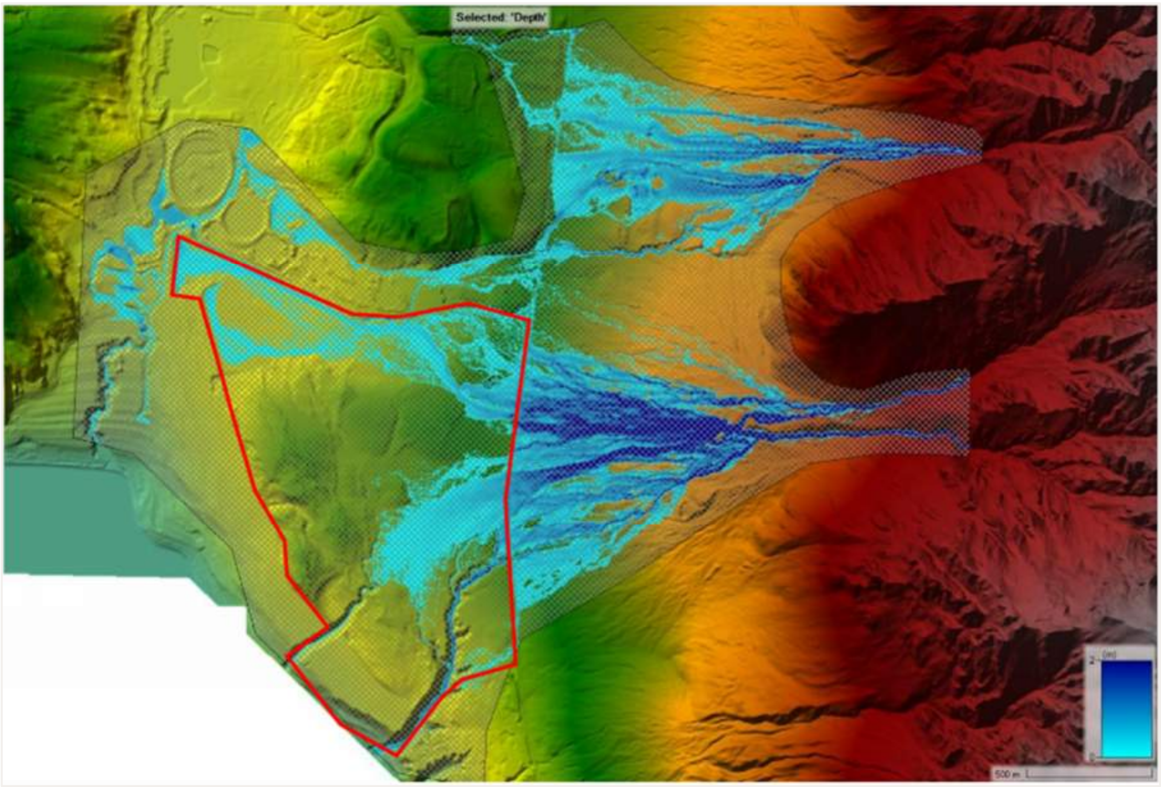

Figure 4-5: HEC-RAS pre-development clearwater flow-depth modelling results for 1%AEP runoff showing modelled area upon DEM terrain. The development site boundary is shown in red

83 Geosolve, Natural Hazard Assessment Homestead Bay, March 2025.

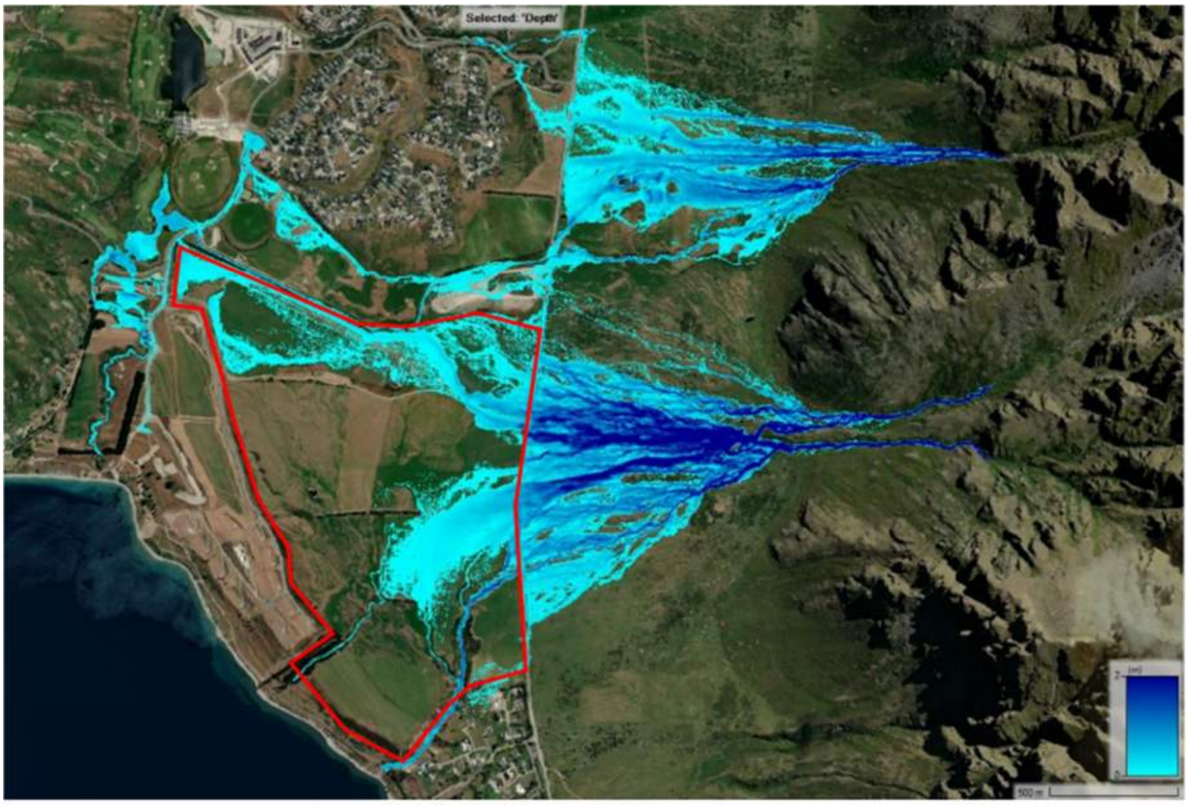

Figure 4-6: HEC-RAS pre-development clearwater flow modelling results for 1%AEP runoff showing depth in metres upon satellite imagery. The development site boundary is shown in red

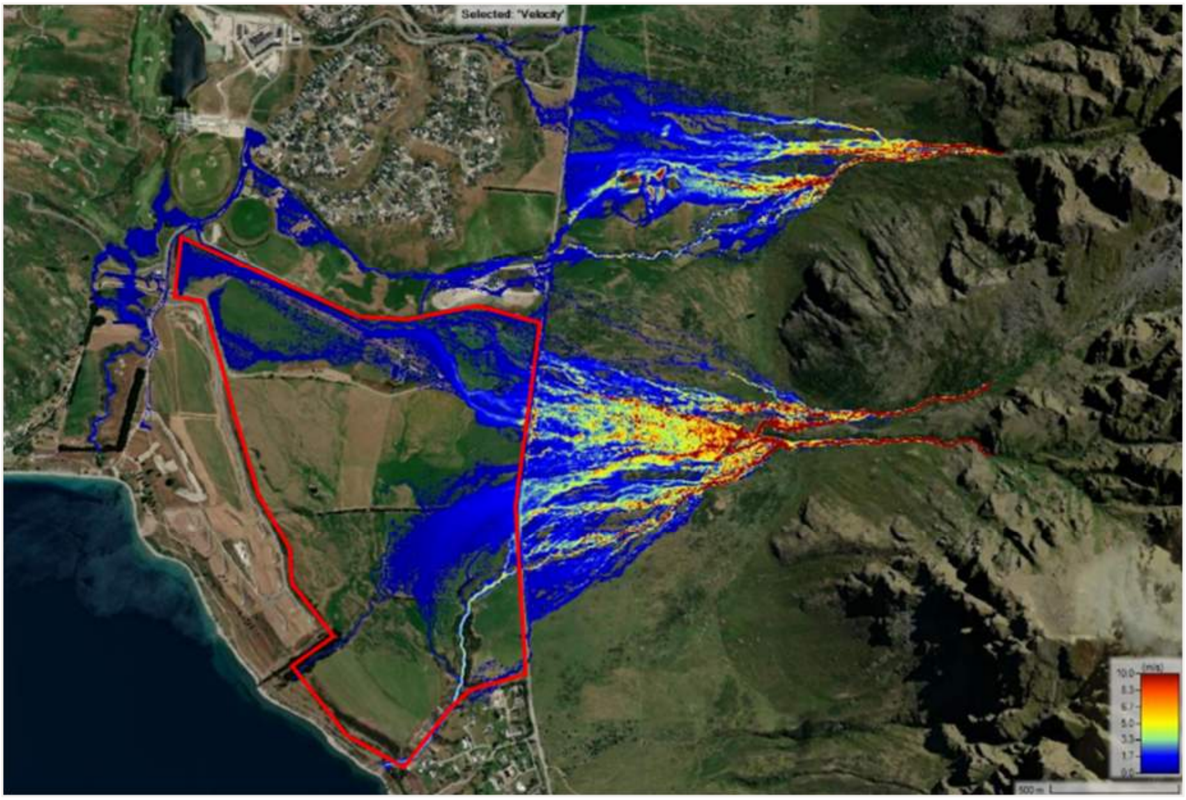

Figure 4-7: HEC-RAS pre-development clearwater flow modelling results for 1%AEP runoff showing velocity in metres/second upon satellite imagery. The development site boundary is shown in red

4.4.2 Rockslide Risk

Geosolve carried out debris mobilisation risk modelling for Homestead Bay in 2025[84].

Essentially, the Homestead Bay proposed development area is just outside the high-risk area for debris mobilisation during flooding, and residual risk is proposed to be addressed by the available freeboard within the diversion channel and bund around the perimeter of the development.

4.4.3 Flood Protection

The geotechnical risks assessment highlighted the potential risk of the three creeks (Northern, Middle, Southern) shifting alignment over alluvial fans. When the creeks shift over the fan(s) over time, the total combined flow carried down the three creeks will not increase but the proportional flow between them may, i.e. the Middle Creek flows could shift and combine with the Northern or Southern creeks over a new alignment.

To mitigate the risk of alpine catchment flows from the upper catchments affecting the proposed development within Homestead Bay, flood protection works are proposed in the form of a natural earthworks bund formed by constructing a wide channel along the entire eastern frontage of the proposed development. The majority of the footprint of the channel/bund is within the development property, immediately adjacent to SH6. The minority proportion of the flood protection earthworks is formed on SH6 around the requirements of the proposed main roundabout entrance– where an earthworks bund is proposed on the upstream boundary line of SH6 designation land offering protection to the proposed roundabout that will be built and placed “off-line” to the pre-development SH6 alignment, and slightly down-gradient of the bund.

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[84] Geosolve, Natural Hazard Assessment Homestead Bay, March 2025.

The proposed perimeter diversion bund and channel system is sized to pass flows from a worst-case combination of alpine creek alignments as indicated from Geosolve assessments. The diversion channel/bund is split into two parts and graded to pass flows to the north and south of the proposed SH6 roundabout footprint. The SH6 culverts are small (DN1200 and multiple DN225-600 culverts) and excluded from the large risk assessment modelling.

The two design cases for the diversion channel and bund are:

1. 1%AEP (including future climate change) runoff, existing stream alignments, conveyed with 500mm minimum freeboard to critical levels. This is the "design case" to meet normal standards for protection of buildings and infrastructure and for usability of facilities
2. 1%AEP (including future climate change) runoff, feasible worst-case combinations of stream alignments, conveyed with 0mm minimum freeboard to critical levels. This is the "over-design" case to provide additional protection in extreme events.

The design runoff cases are shown in Figure 4-8 and Figure 4-9.

Based on the higher level of the Southern Creek outlet from the mountains, assumptions are that the Southern Creek can merge with the Middle Creek outlet and "swing" across the full Development property boundary over time, and the Middle Creek can "swing" partially south of the proposed roundabout as well as north of it. The Northern Creek is already modelled at its extreme southern fan extent and is already in its worst-case alignment.

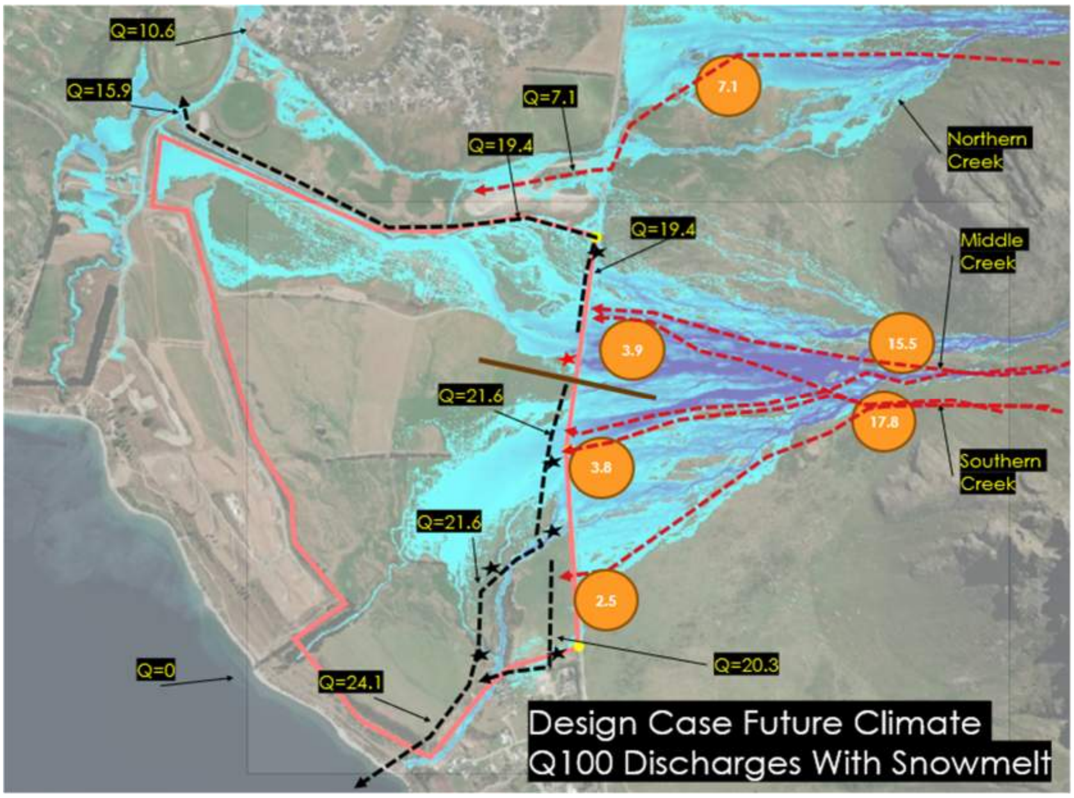


Figure 4-8: Diversion Channel and Bund: Design Runoff Case

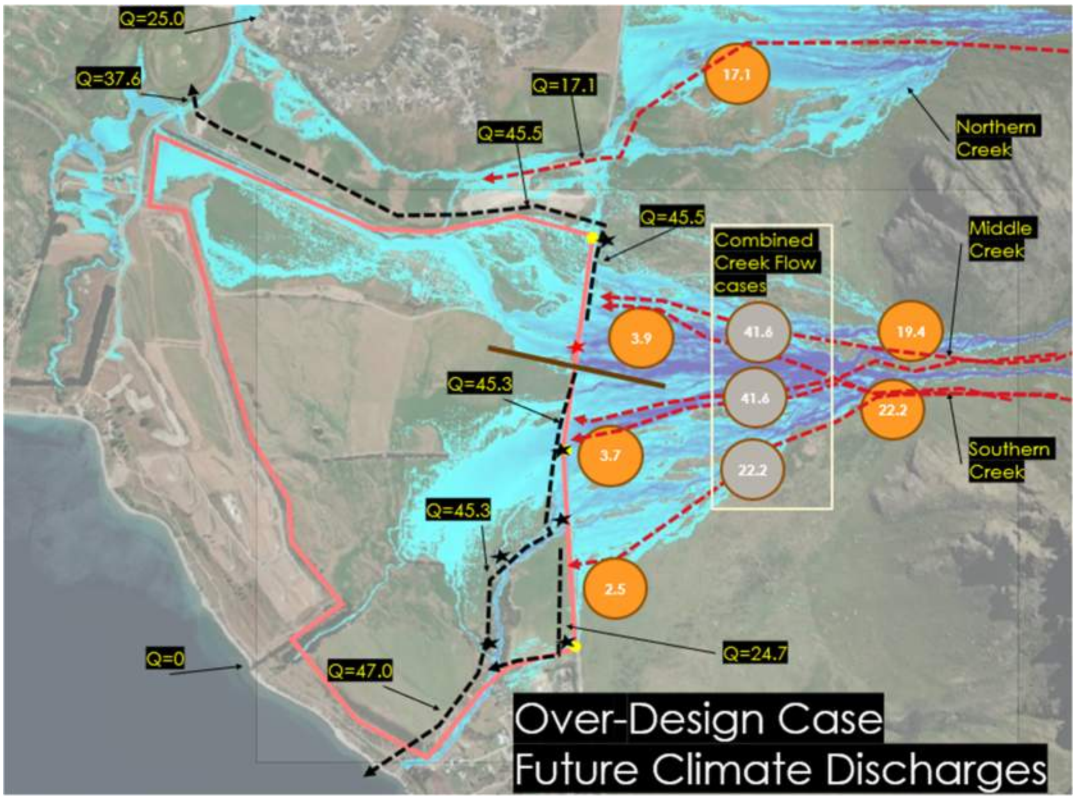


Figure 4-9: Diversion Channel and Bund: "Over-Design" Runoff Case

Drawings 310104425-00-000-C0273 to C0276 shows the concept design for the perimeter diversion bund and channel which has been sized with Geosolve design flood hydrology and hydraulic modelling input. This is described in the following sections and will be developed further in detailed design.

4.4.3.1 Proposed Diversion Channel/Bund Description

The proposed diversion channel/bund is a vegetation lined, with a nominal triangular cross section, shown in Drawings 310104425-00-000-C0274 to C0276. The channel and bund will also provide visual mitigation measures favoured for landscape / urban design. It can be shaped and planted to take on a natural appearance.

The size and effectiveness of the diversion channel/bund is demonstrated in Geosolve 2D hydraulic modelling scenarios that incorporate the proposed diversion channel/bund dimensions. The channel design has practicable dimensions that can be incorporated into the proposed development layout. These designs will be refined with more modelling and as subdivision details such as roundabout and landscape linings arise.

4.4.3.2 Proposed Roundabout Protection Bund

The proposed roundabout intersection on SH6 is sited at the highest part of SH6 along the development frontage and is currently at a dividing spur line between the Southern and Middle Creeks. The Geosolve alpine risk assessment identified that alpine creek movement during a large flood event or incrementally over time can (in the future) focus stream flows directly into the proposed roundabout intersection on SH6. A 2m high, 10m base width, bund is proposed to deflect flood flows north and south of the roundabout location.

4.4.3.3 Northern Boundary Channel

Alpine flows diverted to the north of the proposed development are to be carried westward through a new channel formation along the northern boundary of the development towards the existing 4m x 1m box culvert under Homestead Bay Rd. The proposed alignment would mostly cover the existing stream alignment but would also divert to follow the outside of the development boundary but within land owned by RCL. The new channel will have greater capacity than the existing channel. Drawings 310104425-00-000-C0270 to C0272 shows the concept design for this channel which has been sized with Geosolve design flood hydrology and hydraulic modelling input.

Beyond the box culvert under Homestead Bay Rd and the proposed development, flows drain through a winding rural stream called Māori Jack Stream to finally drain to the Lake shore.

4.4.3.4 Southern Boundary Channel

Alpine flows diverted to the south of the proposed development are emptied into the Southern Creek and carried directly to the Lake. The Southern Creek has a gradient of between 5% and 10% and a base width of over 30m, with enormous capacity. The gully is naturally lined with grass, vegetation and rocks, and currently manages the Southern Creek flows which pass under SH6 through a DN1200 culvert (but the Southern Creek will overtop SH6 in a larger runoff event and be caught by a residual length of the diversion channel/bund on the development land, south of the DN1200).

A residual length of the diversion channel/bund to the south is proposed at the southern corner of the proposed development adjacent to SH6, and sized to manage the Southern Creek flows in the event it naturally changes alignment across the fan and discharges further south. The diversion network is completed with a rock-lined, 4m wide, trapezoidal channel along the southern boundary of the development until it connects back into the Southern Creek again.

4.4.3.5 Proposed Culvert in Southern Channel

A 3m x 3m box culvert is proposed within the Southern Channel gully to allow a road crossing. The culvert size is designed to pass the1%AEP event with future climate change and can pass the design event with 50% blockage, and with 0.5m freeboard to the adjacent critical road level.

4.5 Catchment Runoff – Inside the Development

The proposed stormwater management within the development area will be designed to QLDC COPLD requirements to minimise stormwater risks to the development and to minimise adverse effects on existing neighbouring stakeholders and the environment. A network of piped reticulation will collect rainfall runoff and pipe it to discharge points at surface channels.

There are six groupings of sub-catchments that are described in detail from north to south:

1. Outlet 1 to Māori Jack Stream – green space (effectively no change in runoff characteristics due to proposed development
2. Northern Outlets to Māori Jack Stream – with attenuation volumes designed to maintain a neutral peak runoff in the proposed development
3. Western Outlets – these are small pre-development outlets that will have reduced catchment areas due to the proposed development
4. Southwestern Creek Outlet to Lake Wakatipu
5. Southern Creek Outlet to Lake Wakatipu
6. Minor Lakeside Runoff flowpaths

The potential adverse effects of stormwater from the new development are associated with possible increased runoff quantity and possible reduced runoff quality.

The development of roads and residential areas will have an increase in peak stormwater runoff. The increase in peak runoff rates to downstream channels of Southern and Southwestern Creeks resulting from the development will be minimal because the upstream catchment runoffs are far greater and are expected to peak at a different time. Further, no attenuation of peak flow is needed for discharge from the stormwater channels directly to Lake Wakatipu because of the minimal contribution of flows from the Homestead Bay development area to the much larger catchment of the lake.

However, where downstream property could be affected, control of discharges will ensure peak flow does not exceed predevelopment flow rates. The Middle and Northern Creeks along the northern boundary of the site discharge to an existing channel (Māori Jack Creek) on neighbouring land prior to reaching Lake Wakatipu. The northern portion of the site will therefore require attenuation of peak flows.

Potential water quantity effects are increased peak discharge and increased runoff volumes due to increases in impermeable areas and concentration of flows. The proposed subdivision mitigations will include:

- Primary stormwater pipeline network including direct stormwater drainage pipes from lots, kerblines and road catchpits, manholes and outlet structures, draining the 5%AEP rainfall runoff event to two outlets to the north of the development, and four outlets to the south of the development.
- Secondary stormwater network flows through the proposed road reserves and small accessways of the subdivision to the outlets draining the 1%AEP design event in conjunction with the primary pipe network with minimum 500mm freeboard to habitable dwellings.
- Two attenuation storage volumes within the northern part of the development for holding and throttling peak discharges up to the 1%AEP event,
- Four controlled pipe outlets with impact basins, rock aprons and protective rock riprap linings to reduce flow energy and minimise potential erosion.

Assessment and mitigation of potential increase in runoff quantities is discussed in Section 4.5. Summaries of the changes in peak runoff for the entire development are shown in subsections below. Pre and post development runoff catchment plans are presented.

Assessment and mitigation of potential impacts on stormwater quality is discussed in Section 4.6.

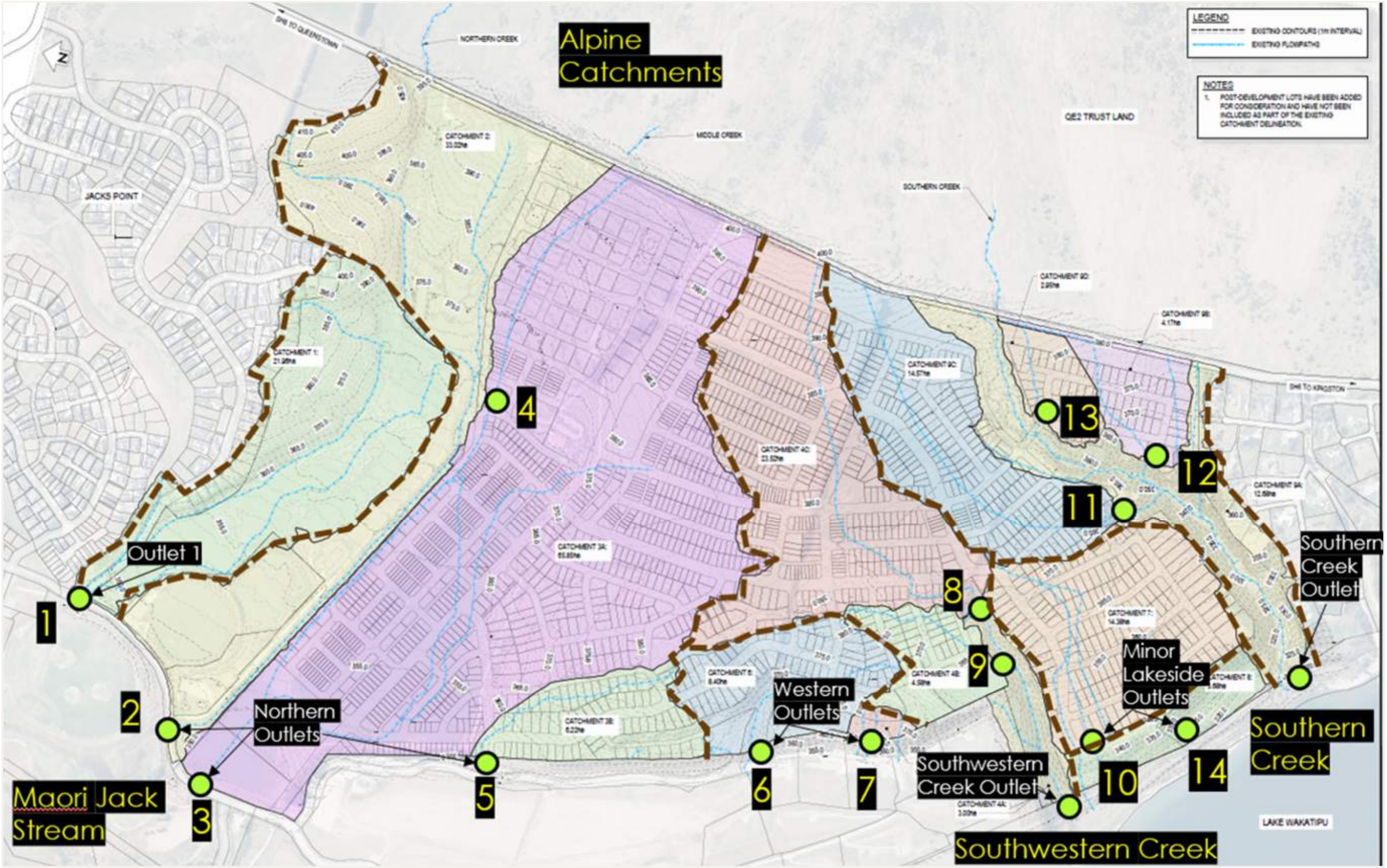


Figure 4-10: Homestead Bay - Pre-development Runoff Catchment Plan with numbered outlets with proposed development layout shown for context

Figure 4-11: Homestead Bay - Post development Runoff Catchment Plan with numbered and identified outlets

4.5.1 Outlet 1 – Unchanged Characteristics

Runoff from Catchment 1 will be unchanged by the development.

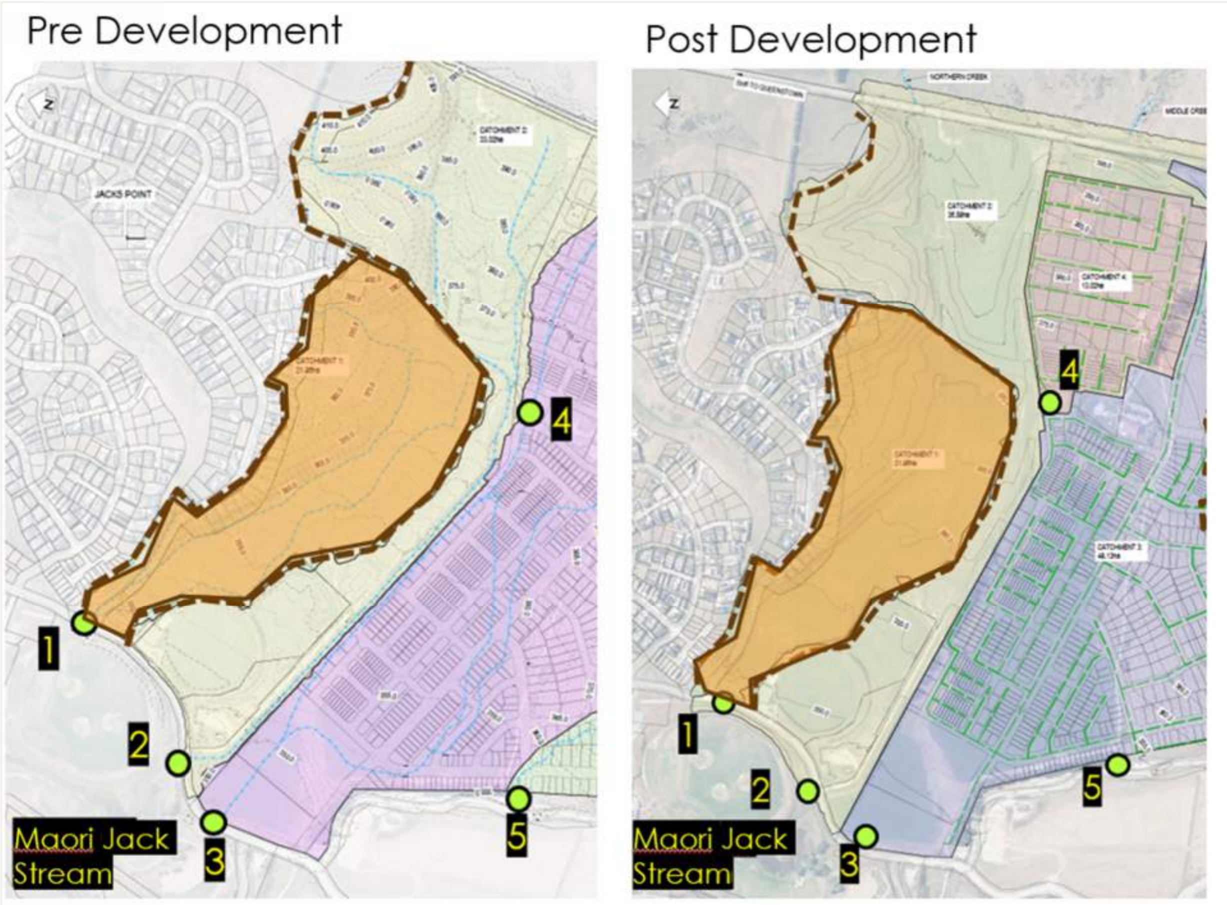


Figure 4-12: Outlet 1 – a catchment with runoff characteristics not affected by proposed development

Table 4-2: Outlet 1 - Catchment Runoff Peak Discharges

Outlet Node	Pre Development			
	Current Rainfall		RCP8.5/2100 Rainfall with Climate Change	
	5% AEP (m^3/s)	**1% AEP (m^3/s)**	**5% AEP (m^3/s)**	**1% AEP (m^3/s)**
1	0.771	1.601	1.261	2.555
Outlet Node	**Post Development**			
	Current Rainfall		RCP8.5/2100 Rainfall with Climate Change	
	5% AEP (m^3/s)	**1% AEP (m^3/s)**	**5% AEP (m^3/s)**	**1% AEP (m^3/s)**
1	0.771	1.601	1.261	2.555

4.5.2 Northern Outlets (2,3,4) – Attenuated Discharges

The Northern Outlets (2 and 3) currently service the northern portion of the proposed subdivision and drain to the existing Māori Jack Stream flowpath. Outlet 4 is a proposed new outlet to the Middle Creek. The proposed subdivision will combine the subdivision flows to pass through Outlet 2 (the existing 4m x 1m Box culvert under Homestead Bay Rd). Māori Jack Stream conveys flows for 2km and falls approximately 37m to its Lake outlet.

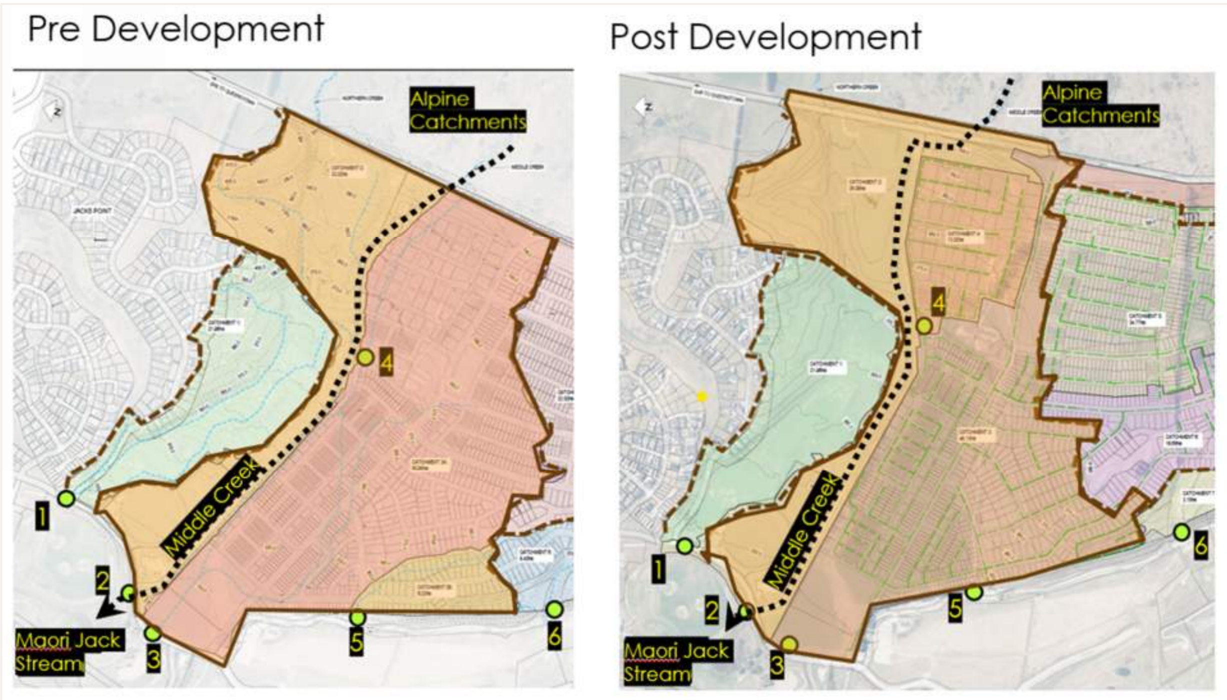


Figure 4-13: Northern Outlets to Middle Creek and Māori Jack Stream. Post development catchment runoff will pass through Outlet 2 (an existing 4mx1m box culvert under Homestead Bay Rd) draining to Māori Jack Stream. Flows will be attenuated through two Basins incorporated into the subdivision. Outlets 3 and 5 will pass minimal post development flows due to catchment diversions.

Tables below summarise the peak runoffs at the Homestead Bay Road boundary for four runoff scenarios. Increased peak catchment runoff is a potential negative impact on downstream Māori Jack Stream and adjacent properties. The proposed mitigation is to place attenuation basins in the subdivision to manage post-development peak discharge to pre-development discharges.

Table 4-3: Outlets 2,3,4 - Catchment Runoff Peak Discharges

Outlet Node	**Pre Development**			
	Current Rainfall		RCP8.5/2100 Rainfall with Climate Change	
	5% AEP (m^3/s)	**1% AEP (m^3/s)**	**5% AEP (m^3/s)**	**1% AEP (m^3/s)**
2	1.508	3.021	2.431	4.861
3	1.579	3.091	2.526	4.874
4	0.000	0.000	0.000	0.000
Sum	3.087	6.112	4.957	9.735
Outlet 2	3.087	6.112	4.957	9.735

Outlet Node	**Post Development (without attenuation)**			
	Current Rainfall		RCP8.5/2100 Rainfall with Climate Change	
	5% AEP (m^3/s)	**1% AEP (m^3/s)**	**5% AEP (m^3/s)**	**1% AEP (m^3/s)**
2	0.992	2.06	1.642	3.383
3	5.384	8.698	7.377	11.970
4	1.251	2.013	1.71	2.748
Sum	7.627	12.771	10.729	18.101
Outlet 2	7.627	12.771	10.729	18.101

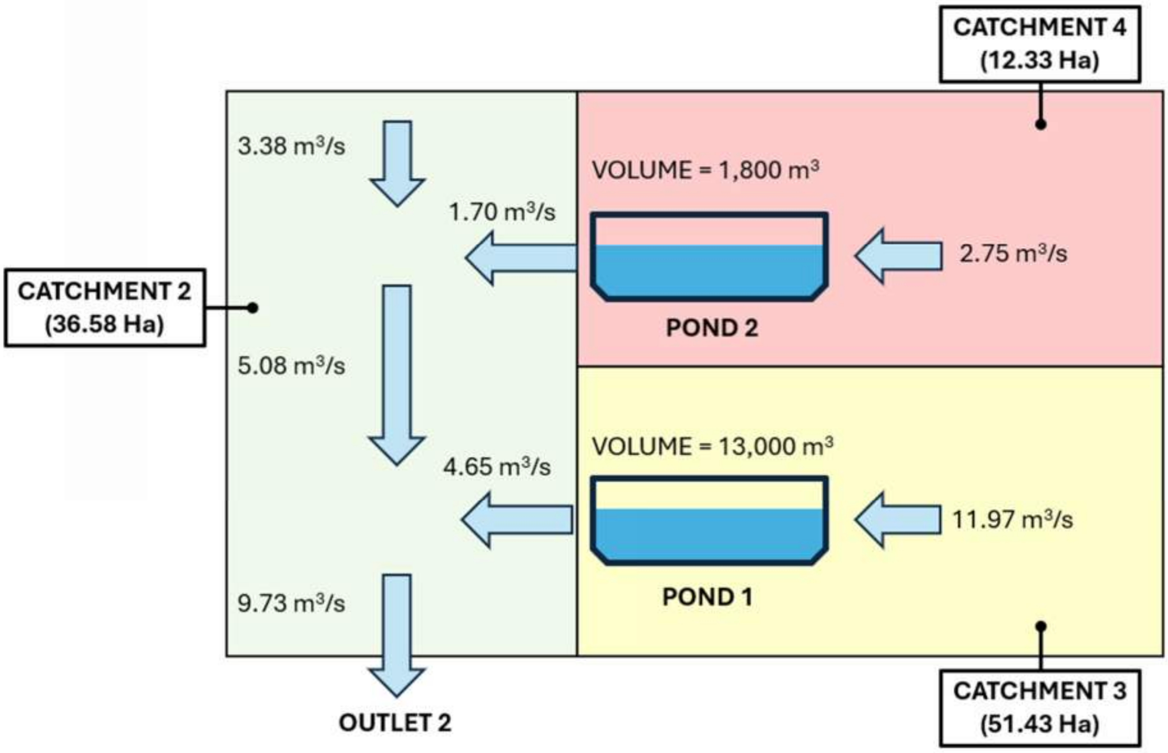
The proposed development will attenuate runoffs through two ponds – one at each of Outlet 3 and Outlet 4. Attenuation volumes are designed and provided for in the scheme to reduce peak post-development discharge to pre-development discharge for a range of magnitude events (5%AEP through to 1%AEP). These ponds are identified as Basins 1 and 2.

The critical flow condition for the detention pond size is the RCP8.5/2100 1%AEP.

Basin 1 in the northwest corner and lowest level at Outlet 3 in this part of the development will have a minimum capacity for 13,000m3 flood water storage. This has capacity to reduce the 1%AEP post development (including future climate increased rainfall) peak flow from 51.4 hectares of post-development catchment runoff from 11.97m3/s to 4.65m3/s.

Basin 2 serves 12.3 hectares of Catchment 4 in the northeastern portion of the development. It has a minimum capacity of 1,800m3 and reduces the 1%AEP post development (including future climate increased rainfall) demand at Outlet 4 from 2.75m3/s to 1.7m3/s.

The impact of the detention of flows is shown in the following schematic:



The effect of the mitigation is to match pre-development outflows and is summarised in the following table.

Outlet Node	Pre Development		Post Development with mitigation	
	RCP8.5/2100 Rainfall with Climate Change		RCP8.5/2100 Rainfall with Climate Change	
	5% AEP (m^3/s)	**1% AEP (m^3/s)**	**5% AEP (m^3/s)**	**1% AEP (m^3/s)**
2	2.431	4.861		3.38
3	2.526	4.874		4.65
4	0.000	0.000		1.70
Sum	4.957	9.735		9.73
Outlet 2	4.957	9.735		9.73

4.5.3 Western Outlets (5, 6, 7)– Discharge to Adjacent Property

Several minor pre-development catchments drain to existing culverts under Chief Reko Rd. These outfalls are labelled 5, 6 and 7.

For each of these minor catchments draining to Chief Reko Rd, the proposed post development catchment areas will be reduced compared to the pre-development catchment area by earthworks and new primary and secondary stormwater systems inside the subdivision development. These outlets will have decreased runoffs post-development.

Therefore, no additional mitigation measures are needed.

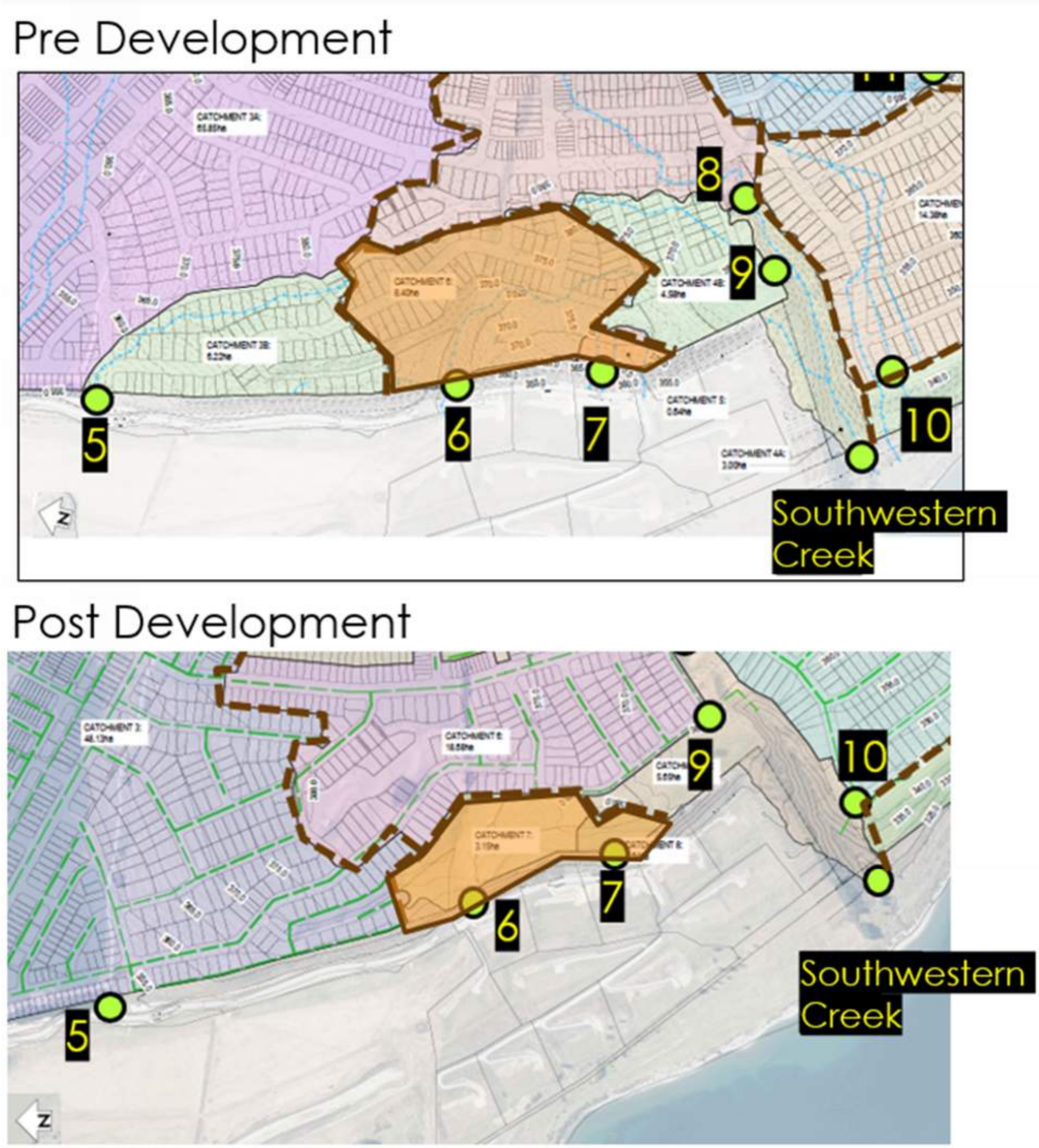


Figure 4-14: Outlets 5, 6 and 7 each drain to an existing culvert under Chief Reko Rd - Pre and Post Development Catchments showing reduction in catchment area

Table 4-4: Western Outlets 5,6,7 - Catchment Runoff Peak Discharges

Outlet Node	Pre Development			
	Current		RCP8.5/2100	
	5% AEP (m^3/s)	1% AEP (m^3/s)	5% AEP (m^3/s)	1% AEP (m^3/s)
5	0.542	1.024	0.834	1.535
6	1.061	1.446	1.293	1.845
7	0.087	0.161	0.131	0.236
Sum	2.140	3.466	2.940	4.850

Outlet Node	Post Development			
	Current		RCP8.5/2100	
	5% AEP (m^3/s)	1% AEP (m^3/s)	5% AEP (m^3/s)	1% AEP (m^3/s)
5	0.072	0.138	0.112	0.209
6	0.747	0.855	0.812	0.965
7	0.087	0.161	0.131	0.236
Sum	1.042	1.398	1.256	1.764

4.5.4 Southwestern Creek Outlets (8,9,10) - Discharge to Lake Wakatipu

The Southwestern Creek catchments of the proposed development (Outlets 8, 9, 10) do not need flows to be attenuated but will require erosion controls at the discharge points and within the base of the creek to manage and minimise erosion due to higher discharges.

Peak runoffs are summarised in tables below. The proposed subdivision will increase discharge into the Southwestern Creek compared to pre-development flows. The development of roads and residential areas will have an increase in peak stormwater runoff. The Southwestern Creek is a large channel and has ample capacity to accommodate the increase in flows. Discharge from the Southwestern Creek is then to Lake Wakatipu. No attenuation of peak flow is needed for discharge from the Southwestern Creek to Lake Wakatipu because of the minimal contribution of flows from the Homestead Bay development area to the much larger catchment of the lake.

Erosion mitigations will be achieved through the use of steep pipelines down the gully sides to the gully floor, impact basins at the ends of the steep pipelines to remove energy from the flow, rock aprons to transition turbulent flows to gully flows, and planting and other rock placements to form check dams and armouring for the gully floors (See Section 4.6.3.7).

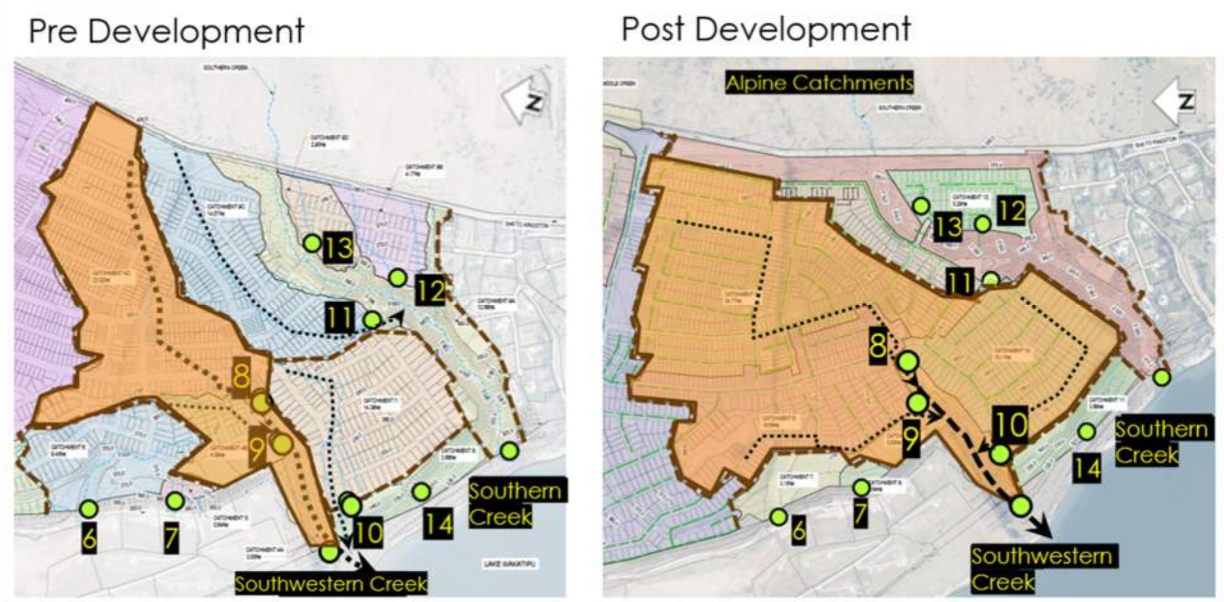


Figure 4-15: Southwestern Outlets 8, 9, 10 showing flood discharge Catchments Draining to Lake Wakatipu.

Table 4-5: Southwestern Creek Outlets (8,9,10) - Draining to Lake Wakatipu -

Outlet Node	Pre Development			
	Current		RCP8.5/2100	
	5% AEP (m^3/s)	1% AEP (m^3/s)	5% AEP (m^3/s)	1% AEP (m^3/s)
8, 9, 10 Gully Area	1.325	2.592	2.100	4.023
Gully Outlet	1.325	2.592	2.100	4.023

Outlet Node	Post Development			
	Current		RCP8.5/2100	
	5% AEP (m^3/s)	1% AEP (m^3/s)	5% AEP (m^3/s)	1% AEP (m^3/s)
8	4.11	6.772	5.741	8.992
9	0.201	0.331	0.279	0.459
10	1.491	2.436	2.061	3.357
Gully Area	0.076	0.175	0.134	0.306
Gully Outlet	5.878	9.714	8.215	13.114

4.5.5 Southern Creek Outlets (11,12,13) - Discharge to Lake Wakatipu

Similar to the Southwestern Creek, the Southern Creek catchments of the proposed development (Outlets 11, 12, 13) do not need attenuated flows but will require erosion controls at the discharge points and within the gullies to manage and minimise erosion due to higher discharges.

Peak runoffs are summarised in tables below. The proposed subdivision will reduce catchment areas but increase development runoff due to land use change. On the whole, peak post-development discharges into the Southern Gully will slightly reduce or be close to neutral compared to pre-development flows.

The increase in peak runoff rates within the Southern Creek resulting from the development will be minimal because the upstream catchment runoffs are far greater and are expected to peak at a different time. The Southern Creek is a large channel and has ample capacity to accommodate the increase in flows. Further, no attenuation of peak flow is needed for discharge from the Southern Creek to Lake Wakatipu because of the minimal contribution of flows from the Homestead Bay development area to the much larger catchment of the lake.

The Southern Creek will now convey larger alpine flows due to the collector bund and channel placed parallel to SH6 which will combine two alpine catchments into the one Southern Creek flowpath.

Erosion mitigations for post-development subdivision runoff will be achieved through the use of steep pipelines down the gully sides to the gully floor, impact basins at the ends of the steep pipelines to remove energy from the flow, rock aprons to transition turbulent flows to gully flows, and other rock placements to form check dams and armouring for the gully floors as needed.

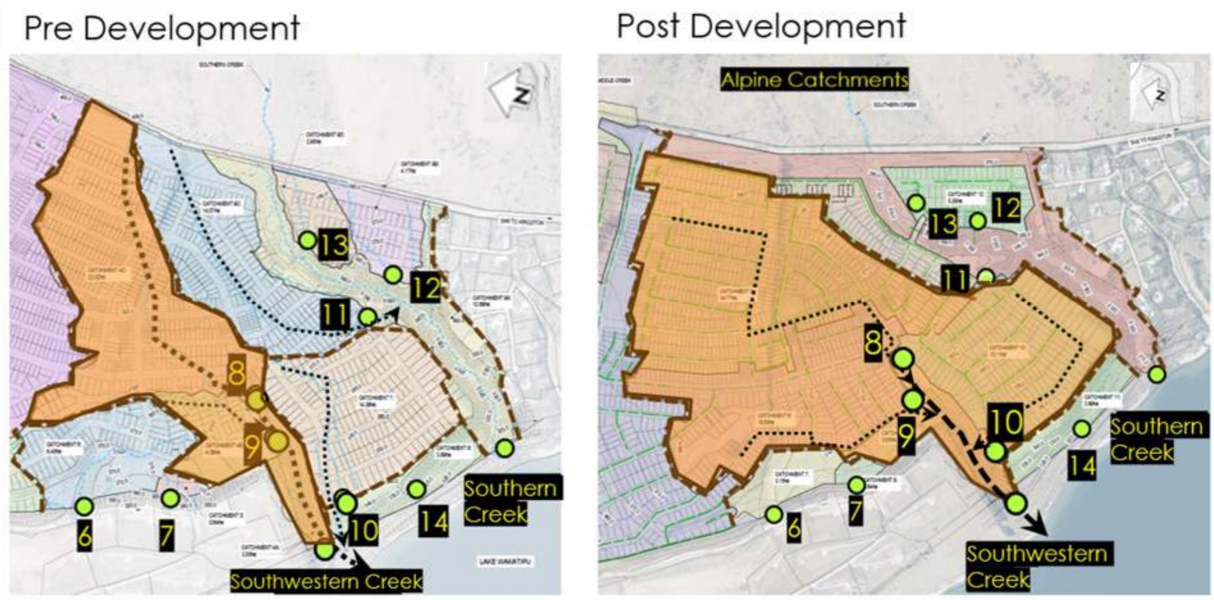


Figure 4-16: Southern Creek Outlets 11, 12, 13 showing reduced catchment areas and reduced reliance on Southern Creek floor for conveyance.

Table 4-6: Southern Gully, Catchment Runoff Peak Discharges

Outlet Node	Pre Development			
	Current		RCP8.5/2100	
	5% AEP (m^3/s)	1% AEP (m^3/s)	5% AEP (m^3/s)	1% AEP (m^3/s)
11	0.000	0.000	0.000	0.000
12,13	0.000	0.000	0.000	0.000
Gully Area	1.135	2.282	1.826	3.672
Gully Outlet	1.135	2.282	1.826	3.672

Outlet Node	Post Development			
	Current		RCP8.5/2100	
	5% AEP (m^3/s)	1% AEP (m^3/s)	5% AEP (m^3/s)	1% AEP (m^3/s)
11	0.279	0.463	0.390	0.644
12,13	0.488	0.801	0.676	1.109
Gully Area	0.406	0.884	0.688	1.472
Gully Outlet	1.173	2.148	1.754	3.225

4.5.6 Minor Lakeside Outlets (14) Discharge to Lake Wakatipu

Small lakeside runoffs will be controlled and reduced by proposed subdivision development. Peak runoffs are summarised in tables below. The proposed subdivision will reduce catchment areas. Flows are small and impacts will be reduced. No mitigations are required.

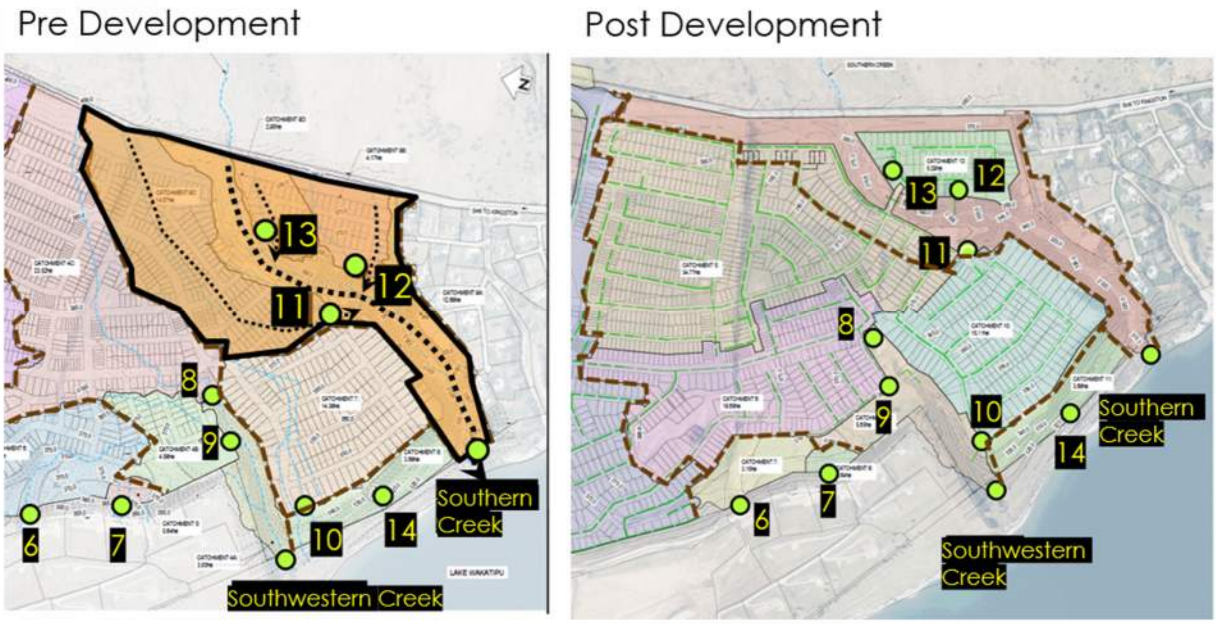


Figure 4-17: Minor Lakeside Runoff direct to Lake Wakatipu

Table 4-7: Minor Lakeside Runoff outlets (14) - Draining to Lake Wakatipu

Outlet Node	Pre Development			
	Current		RCP8.5/2100	
	5% AEP (m^3/s)	**1% AEP (m^3/s)**	**5% AEP (m^3/s)**	**1% AEP (m^3/s)**
10	0.136	0.276	0.220	0.439
14	0.393	0.773	0.840	1.201
SUM	0.529	1.049	1.060	1.640

Outlet Node	Post Development			
	Current		RCP8.5/2100	
	5% AEP (m^3/s)	**1% AEP (m^3/s)**	**5% AEP (m^3/s)**	**1% AEP (m^3/s)**
10	0	0	0.000	0.000
14	0.047	0.100	0.078	0.166
SUM	0.047	0.100	0.078	0.166

4.6 Stormwater Quality

4.6.1 Introduction

Potential downstream impacts from a land use change and development include water quality impacts including contamination levels and erosion or silting effects. Where these effects are more than minor, mitigations are required such as energy dissipation, attenuation storage volumes, treatment devices and strategies. This is indicated in QLDC COP section 4.2.7 and following sections.

An assessment has been made of likely stormwater quality (Section 4.6.2), and proposed stormwater treatment proposals have been formulated to be appropriate for this (Section 4.6.3).

4.6.2 Requirements for Stormwater Treatment

In determining the appropriate level of stormwater treatment for the Homestead Bay development, a review of the monitoring results from the nearby Hanley's Farm residential development was undertaken to:

- Identify key contaminants of potential concern that may be present in stormwater,
- Determine if concentrations of any contaminants will potentially be at levels that indicate potential risks to receiving environment quality, and
- Use the available information from Hanley's Farm to inform recommendations for appropriate stormwater management for the Homestead Bay development that would ensure stormwater discharging to the receiving environment had undergone appropriate management to mitigate anticipated risks of adverse effects of stormwater discharging to the receiving environments.

4.6.2.1 Stormwater monitoring at Hanley's Farm

Hanley's Farm is a subdivision that has been established in the same surface water catchment as the proposed Homestead Bay subdivision by RCL Homestead Bay Limited and is of a similar design. From the stormwater monitoring that has been undertaken at Hanley's Farm, the anticipated quality and management requirements of stormwater likely to be generated for the Homestead Bay subdivision can be gathered.

Appendix F sets out a summary of the Hanley's Farm routine monitoring under Consent RM22.263.01. A series of generalisations can be drawn about the quality of stormwater discharging from the various stormwater catchments within the Hanley's Farm subdivision.

Nutrients:

- There were generally low levels of Dissolved Inorganic Nitrogen (DIN) present in the various catchments (as assessed by the components of Ammoniacal-N (Amm-N), nitrate + nitrite nitrogen (NNN)).
- There were relatively low levels of Total Oxidised Nitrogen (TON) (NNN – i.e. nitrate + nitrite, taking into account the limited monitoring available, nitrate did not appear to be problematic in the stormwater discharging from the subdivision.
- It is apparent that the Total Nitrogen (TN) was contributed largely to by the organic component (i.e. as assessed by the Total Kjeldahl Nitrogen (TKN), and small component of TON). Given the low concentrations of Ammoniacal-N (Amm-N), the dominant TN was likely to be organic nitrogen.
- There was a strong correlation between Total Suspended Sediment (TSS) and TN, TKN and TP associated with stormwater discharging from newer lots within the subdivision (i.e. more recent development and less urbanised). This suggests that these nutrients were largely sediment associated, with concentrations relatively higher in the newer areas (and associated higher sediment concentrations), as compared with the older more developed lots. Given the apparent strong correlation between TN/TKN/TP and TSS, it is likely that managing and reducing TSS in the stormwater will concurrently reduce stormwater nutrient loads from the subdivision areas, in particular for newer less urbanised lots
- Elevated levels of Dissolved Reactive Phosphorus (DRP) were apparent in the farmland catchment that influences the discharge at DP9 but there was no apparent increase due to the subdivision itself.
- Overall there was no indication that the stormwater discharged from the subdivision contained significant amounts of urban derived nutrients (TN or TP)– this is apparent in the quality of the stormwater, and summary from LWP (2025[85]) that receiving environment levels of total nutrients are generally regarded as low.

Metals:

- Metals in Hanley's Farm stormwater samples were analysed for the total fraction only – not dissolved, meaning that direct comparison against available guidelines is limited.
- The relatively limited data set (n=4 for metals) indicated copper, lead, nickel and zinc were generally higher from the newest lots draining from the Hanley's Farm subdivision.
- Typical stormwater indicator contaminant concentrations for copper and zinc (as well as other metal/metalloids arsenic, chromium, lead and nickel) were generally associated with higher TSS loads, suggesting a higher component of the total fraction (i.e. sediment associated), rather than the dissolved fraction of metals were present in the discharge (noting this was not directly analysed).
- Metal concentrations (e.g. chromium and nickel) and arsenic (a metalloid) are likely to be associated with natural geology of sediments (derived from metamorphic schist rock formations) in the catchment.

Sediment:

- Elevated TSS were generally associated with the newer lots within the subdivision compared with older (more urbanised) lots, resulting in higher concentrations of TSS discharging in stormwater from newer sites
- This was reflected in the increasing turbidity, dissolved solids and conductivity at these more recent/less urbanised locations.
- Higher TSS in new lots was likely as a result of residual surficial sediment following development, that is anticipated to reduce to background levels as the development is finalised and exposed bare

[85] Land Water People (LWP), 2025. Memorandum: Assessment of sensitivity and water quality criteria for Lake Wakatipu and its tributary streams, and of risks to manage for the treatment and discharge of wastewater from Homestead Bay housing development. 20 March 2025

sediment areas are vegetated and maintained.

4.6.2.2 Implications for Homestead Bay

Drawing from the key findings above, the key contaminants of concern potentially contributing to stormwater quality discharging from the Homestead Bay subdivision include effects due to increased nutrients, metals, and sediments. These are summarised in turn.

For the proposed Homestead Bay subdivision, it is anticipated stormwater quality will be similar to that monitored for the Hanley's Farm subdivision. This is due to both the similar topography and proximity between the two subdivisions, and the similarity in design and construction for the subdivision (and associated materials and source control options) themselves.

Considering the low ecological sensitivities of the primary receiving environment (ephemeral channels), coupled with the very high likelihood of significant stormwater dilution in rainfall events that cause ephemeral flow in the channels, the overall risk of adverse effects of contaminants in stormwater discharges to adjacent streams/tributaries is considered low. For Lake Wakatipu, any residual stormwater that potentially discharges to the lake has already been appropriately managed, with any residual contamination likely to be negligible to low as compared to background water quality. Thus, overall risk to the Lake is also considered low.

The focus for stormwater management options is recommended to be on managing for sediment and bulk material (litter etc) and management of flow. Some reductions in nutrients and metals will be achieved for the fraction which is associated with the solids in the stormwater.

The findings from the Hanley's Farm monitoring indicate that by managing for coarse material (litter) and suspended sediment (e.g. by the use of gross pollutant traps and sumps), combined with the incorporation of hydraulic controls (hydraulic neutrality through the attenuation basins), the secondary treatment (as grassed attenuation basins) will further mitigate urban derived contaminants (including metals) by increasing the hydraulic retention time (HRT). This further treatment, and associated increase in HRT, have been demonstrated to reduce TSS loads up to 90% and reduce total metals (copper and zinc) up to 96% (Moores et al. 2010[86]).

Guidance on stormwater treatment devices for managing urban stormwater discharges (and improving water sensitive design) have been developed over the past 10-15 years (e.g. TP010 for Auckland, and Water Sensitive Design for Wellington Water). These documents discuss a range of benefits and limitations of a range of available devices. The guidance notes that different devices provide for different levels of treatment, depending on a range of factors (physical layout, chemical and biological processes used for the device, and the connection to the stormwater network itself) (Wellington Water 2019[87]).

Gross litter, suspended sediment, and to a lesser degree, urban derived metals are likely the main contaminants of concern to be managed by the selected stormwater management devices for the Homestead Bay subdivision. Applicable stormwater management devices for the management of these contaminants (catchpits, gross pollutant traps, and attenuation basins) are discussed in Section 4.6.3 below.

Catchpits and Gross Pollutant Traps (GPTs) are the primary method for sediment controls – along with landscaping to minimise erosion and scour across the catchment. Coupled with 'good housekeeping' (i.e. catchpit clearing, maintenance and cleaning of GPTs etc), significant reductions and management of sediment in the subdivision is likely to account for the main reduction of key contaminants of concern.

Control for metals (total and dissolved) will be focused on source control options, such as design controls on roofing and cladding material to reduce/eliminate galvanised and other zinc-based materials where possible. Whilst it is not possible to directly source control for vehicle emissions and associated contaminants due to tyre and brake pad wear, the nature of vehicle and driver behaviour in the subdivision will mean any discharges of vehicle related contaminants (metals, hydrocarbons and tyre/break wear related compounds) are kept to a minimum and managed via sediment associated

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[86] Moores, J., Pattinson, P., and Hyde, C. (2010). Enhancing the control of contaminants from New Zealand's roads: results of a road runoff sampling programme. New Zealand Transport Agency.
[87] WSD-for-Stormwater-Treatment-Device-Design-Guideline-December-2019.pdf

retention in catchpits and GPTs. Further treatment may in part be achieved using vegetated retention basis (see Section 4.6.3).

During construction phases, a detailed Erosion Sediment Control Plan (ESCP) is critical to the management of sediment (and sediment associated contaminants) discharging into surface water courses. Appropriate site-specific construction related sediment and stormwater controls are to be detailed in the ESCP.

4.6.3 Proposed Stormwater Treatment

4.6.3.1 Overview

The proposed stormwater treatment strategy is described below and in Drawing 310104425-00-000-C0253.

- *All Catchments*: Trap heavy grits, solid rubbish, natural leaf litter in roadside catchpits throughout all the road network.
- *All Catchments*: Install gross pollutant traps (GPTs) at downstream locations near all six catchment outlets to trap floating plastics, hydrocarbons and suspended sediments down to approximately 100-150 microns. These would likely be proprietary manhole devices buried under the surface of the road corridor with short bypass pipelines in parallel to the main primary system pipelines to direct water quality runoff (up to 10mm/hr rainfall) through the devices. Larger storm flows would pass through the primary pipeline without passing through these GPT devices.
- *Northern Catchments*: The two northern outlet locations of the development provide opportunity for flows to cross over attenuation basin footprints and provide additional “polishing” treatment by this opportunity. This would be sedimentation, limited soil infiltration to ground and soakage, slow overland flow through the vegetated base of the basins to provide biofiltration and vegetated uptake of contaminants.
- *Southern*: At the five southern outlet locations to the Southern Creek and Middle Creek install bypass GPTs to trap floating plastics, hydrocarbons and suspended sediments down to approximately 100-150 microns. Larger storm flows would flow through the primary pipeline without passing through these GPT devices. Incidentally, overland flows along the natural gully floor will provide natural biofiltration and vegetated uptake of contaminants. Water quality runoff rates from the development areas (10mm/hr rainfall intensity) onto the gully floors are small relative to the large gully cross sections and are slow-moving. Shallow-depth flows will pass through vegetation and natural roughness on the wide gully-floors. Again, as for the northern catchments draining to attenuation basin floors, these southern catchment flows draining to the natural gully floors will receive incidental “polishing” treatment by processes of vegetation uptake and soil infiltration before entering Lake Wakatipu.

Stormwater treatment strategies will be similar to Hanleys Farm, 5 Mile shopping centre, and other developments within Queenstown Lakes District.

As a note, fish passage is not identified as relevant to this development and is not discussed.

4.6.3.2 Catchpits as Primary System

Catchpits are considered the first line of contaminant capture and are a standard rainfall runoff feature associated with roads and part of QLDC COP requirements, section 4. More-detailed design will occur to define catchpit locations, but they will be sited throughout the development to support road drainage.

4.6.3.3 Gross Pollutant Traps and Inline Filtering

Water quality treatment is required to capture floating litter and gross pollutants such as oils before discharge into the receiving environment. The gross pollutant trap (GPT) is an efficient and effective asset for removing contaminants from stormwater that are visible. Many proprietary products are available on the market and are installed in the Queenstown area.

Figure 4-18: (Left) Typical proprietary GPT manhole as Stage 1 treatment. (Right) Proprietary filter.

For Homestead Bay, each of the seven stormwater outlets are proposed to have a GPT installed near to the outlet to capture as close to 100% of the large a catchment area as possible. A GPT is designed vertically for hydraulic head loss through the device where it is sited in the pipeline network and horizontally sited for ease of maintenance.

The expected performance of a GPT is described in proprietary product literature, but in general terms, a GPT is designed to capture ~75% of the total suspended solids (TSS) fraction down to about 100-150 microns size in a water quality storm event (that is: a 10mm/hr rainfall intensity, which is considered to be about the 90th percentile of all events in a year) I. The GPT performance assumes that other contaminants such as particulate metals, soluble metals, nutrients and hydrocarbons concentrations are partially attached to TSS and captured too. Lighter-than-water floating plastics and hydrocarbons are trapped behind baffles in the GPT design. All captured contaminants are held by the device even in heavy rainfall events and not re-mobilised. GPT maintenance frequency would be determined by practice but typically would be 1-2 times per year.

4.6.3.4 Attenuation Basin as Additional Treatment

The Homestead Bay attenuation basins in the northern catchments are primarily designed for peak discharge attenuation of storm flows. But for water quality flows downstream of a GPT, the attenuation basin floor will provide incidental opportunity for infiltration vertically through the topsoil, biofiltration and nutrient uptake horizontally through grass or vegetation linings and their root masses, and sedimentation from quiet flow velocities.

Figure 4-19: Example of a stormwater attenuation basin in Rangiora showing gradients and grass coverage.

4.6.3.5 Opportunities for Additional Treatment in Ephemeral Channels

Like the detention basins, the ephemeral Southern Creek and Southwestern Creek provide incidental opportunity for infiltration vertically through the topsoil, biofiltration and nutrient uptake horizontally through grass or vegetation linings and their root masses, and sedimentation from quiet flow velocities.

These channels also have potentially good infiltration and soakage areas in the gully floors according to observed test pits carried out for the road network design. Soakage testing has not been carried out to date, but high concentrations of gravels and sands in the test pits and boreholes indicate a potentially high long term design soakage rate is available. Environmental reporting identifies evidence of significant soakage (ref. Norton). Infiltration through a surface topsoil layer with vegetated lining would trap contaminants before entering the soakage layers for disposal.

Peak discharge of primary flows does not need attenuation in the two Creeks but does need energy dissipation to prevent erosion.

4.6.3.6 Decentralised versus Centralised Approach

A consideration of decentralised versus centralised water quality treatment approaches settled on a centralised strategy that could take advantage of strong land and pipeline gradients and the fact that the full subdivision area is serviced by six outlets. Reducing the numbers of water quality outlets was considered to give the best chance for long-term cost-effective maintenance for a Council contractor.

The decentralised approach would require devices to service smaller catchment areas by using dispersed units around the reticulation network (e.g.: smaller proprietary devices in manholes, rain garden areas, “filterra®” tree pits, localised natural infiltration and soakage – see Figure 4-20 for description of a test pit) before draining into the main stormwater network. Although pipelines do have strong gradients, each device would need hydraulic fall to function (in the order of 0.7m-1m per device) and this would force pipelines to be laid deeper throughout the network. Maintenance of many devices was considered to be a burden on a network operations contract.

Figure 4-20: Test pit to 1.8m indicating layers of gravels at 1.0m and 1.5m depth between other layers of sand (reference: bore hole HB13/5, Lowe Environmental ImpactSiols Site Investigation, 23 September 2024)

4.6.3.7 Erosion Controls

Erosion controls will be important to keep the discharge into Lake Wakatipu free of erosion sediments. The five southern catchments will discharge to the bases of the gullies through steep pipelines laid in large-diameter welded PE pipelines with an impact basin energy dissipator and rock lined spill apron to manage energy dissipation.

Figure 4-21: Example of a steep polyethylene pipeline down a low strength bank, to an energy dissipation impact basin, Culvert 880, SH58, Wellington

Figure 4-22: Example of an energy dissipation impact basin and riprap apron to minimise erosion

Rock lined flumes and chutes were not considered feasible given the steepness of the gully walls, and the risk of long-term erosion of the gully walls is too high, given the shape and composition of the existing walls being composed of gravels, sands and silts.

At the top of steep outlet pipelines, an increased number and capacity of catchpits will be required to take the 1%AEP overland flow into the pipeline. Further detailed assessment of earthworks levels and road layouts could indicate preferred overland flowpaths into the bases of the gullies but high-capacity catchpits are a feasible solution – similar to Hawthorn Drive at the airport runway safety embankment where high capacity catchpits were designed for the 1,000 year return event flow.

Potential long-term erosion of the bed of the Southern and Southwestern creeks will be controlled by reducing flow velocities by a combination of the following strategies along the gully floors. These will be adopted to suit the location and anticipated flow. These strategies will seek to spread flood flows across the gully floor width, to increase the gully roughness in shallow-depth flows to reduce velocity, and to line the gully floors with well rooted vegetation and rock placements.

- Develop enhanced rock lining around the centreline (low flow channel)
- Identify locations where inflows will join the gully floor and account for increasing flows in the direction of fall in the gully profile
- Retaining any well-developed trees and shrubs with good roots and landscape value, and existing grass vegetation in riparian parts of the cross sections
- Identify and retain any existing rock formations or grade changes in the gully profiles
- Focussed removal of vegetation not wanted or unsuitable (weeds and young conifers)
- Use turf-reinforcement (for example enkamat) as needed to establish new grass coverage or areas where low vegetation is removed and replanted
- Install improved vegetation including grasses, shrubs and riparian species as part of the gully floor cross section landscaping, choosing species that develop deep roots and can survive intermittent and irregular short term flooding episodes
- Install regular small sections of heavy rock placements across the low-flow channel flowpath to break up flow lines and form a meandering low-flow channel
- At key locations where flood flow velocity will increase with gradient and contributing inflows,

install rock riprap weirs across the gully floor width, to cause short lengths of backwater ponding and short lengths of controlled fall through the rock formations (steps)

- Large rock placement sporadically placed along the channel to blend with landscaping and generally increase the overall roughness of the gully floor and reduce velocity through turbulence.
- Shape the lower extent of the Creeks to make flows spread out (fan out) across a wider front with a part-buried rock formation lining the low flow channel.

5 Transportation

5.1 Introduction

Stantec undertook an Integrated Transport Assessment (ITA) for Homestead Bay[88] and investigated how it will be integrated with the existing and planned transport network.

Below is a summary of the ITA. The report should be consulted for further details.

5.2 Summary of ITA

Development of the Homestead Bay site proposes subdivision to enable more than 2,500 residential units, a local commercial centre, potential school, and recreational facilities. This will contribute further growth in an area that has been identified through Spatial Planning for urban growth in the Southern Corridor.

The Integrated Transport Assessment (ITA) is a requirement of the QLDC District Plan and investigates how the proposed development will be integrated with the existing and planned transport network. It also provides the relevant "design and access statement" as required by the QLDC Land Development and Subdivision Code for the proposed internal transport network.

SH6 is a primary roading corridor servicing the Southern Corridor for vehicle traffic. It is proposed that a direct and safe access point via a roundabout controlled intersection is provided as part of the development. The roundabout control will be consistent with a safe system intersection treatment and the proposed 1.3km spacing from Māori Jack Road is consistent with existing intersection separations along SH6.

SH6 within Frankton and at the northern part of the Southern Corridor is experiencing congestion, and this will worsen into the future. A range of transport related plans and strategies have identified that a shift from private vehicle travel to public transport and active modes will be necessary to support future travel demands from the Southern Corridor. It is expected that a focus will be development of bus and active modes infrastructure. A separate transportation analysis report by WSP has investigated these wide area matters and transportation effects of development beyond the site and potential solutions to support increased movement in the Southern Corridor.

To support increasing self-sufficiency of the Southern Corridor, the Site has made provision for a centrally located local commercial centre (supporting approximately 11,000m^2 of commercial building area), and a school if the Ministry of Education chooses to develop a school at this location. The transport integration of these facilities within the site have been considered as part of the overall planning for the development. These activities are well located to support local movement by a range of transport modes, offering transport choice.

It is anticipated that over time, bus services into the Southern Corridor will be more frequent, supporting reliability of use of buses. It is considered that Homestead Bay can form an efficient extension of the necessary and planned frequent public transport and active modes corridors that is most likely to travel along Homestead Bay Road and the central alignment of the communities to the north including Jacks Point, Hanley's Farm, and Park Ridge.

To support integration with the wider transport network the subdivision development will:

- Provide a primary spine road connection between SH6 and Homestead Bay Road, supported by a primary loop road within the development. These roads will maximise opportunities for an efficient bus connection,
- Provision of bus stops that support local accessibility and efficient expansion of the bus network into the site, as development proceeds.
- Provides a key central north south active modes corridor connecting to an east west corridor along the north boundary of the site supporting direct and safe movement to Homestead Bay Road.

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[88] Stantec. *Homestead Bay Fast Track Approvals Application – Integrated Transport Assessment*. April 2025.

These connections minimise conflict with the primary roads within the development.
- Enable a connected network of streets, shared paths, footpaths, and open space trails to support local cycle, micro-mobility, and walking that minimise walk distances to key destinations within and near the development including the adjacent Homestead Bay Village next to Lake Wakatipu (which is zoned as part of the Jacks Point Special Purpose Zone).

The multi-modal network has resulted in a range of bespoke road cross-sections that will provide a legible network for each transport mode. In terms of a traditional road hierarchy, the Spine Road and Loop Road have been assessed as Collector Roads, with a primary function of supporting access to residential development and the commercial centre. There is expected to be minimal through traffic unrelated to the trip generating activities of the site. In that regard, it is expected a slower speed road environment with a fine grain well connected road network will be achieved supportive of the expected level of traffic, cycle, pedestrian, and parking activity.

The Subdivision has been assessed against Council District Plan Rules and the QLDC Land Development and Subdivision Code. It is considered acceptable solutions are provided for a multi-modal transport network to support the local community and facilitate the desired wider transport network outcomes. Various non-compliances have been identified, and relevant conditions of consent recommended to ensure non-compliances are further considered in later engineering and building consent stages where necessary.

Broadly, it is considered that development of the land area supports transport objectives in the surrounding environment and enables a well-functioning urban environment from a transport perspective. The WSP report addresses wider area infrastructure considerations to support travel demand growth from the Southern Corridor.

Based on the above, it is concluded that the subdivision of the Site as proposed can be supported from a transport perspective.

5.3 Low Impact Design for Stormwater Runoff

Low Impact Design Principles have been included in the design by:

- Detention areas in the northern catchment to act as attenuation devices
- Using the ephemeral Southern Creek and Southwest Channel to slow peak flows
- All stormwater runoff from the roads will discharge via proprietary filtering devices with storm bypass arrangement prior to reaching surface channels
- Detention areas and ephemeral surface channels will provide additional treatment of stormwater from the development.

5.4 Street Lighting

Homestead Bay will be designed to AS/NZS 1158.3.1:2020 subcategory PR6 and/or PR5 for local residential roads and be consistent with the QLDC Southern Light Strategy. Luminaires will likely be LEDs complying with NZTA M30. The lux level specific requirements will be specified similarly as shown below:

- Subcategory PR5: 0.85 Lux (Avg), 0.14 Lux (Min) and Uniformity (Max/Avg) of 10 (Max)
- Subcategory PR6: 0.7 Lux (Avg), 0.07 Lux (Min) and Uniformity (Max/Avg) of 10 (Max).

Higher levels of lighting may be required in areas such as the local shopping centre and state highway intersection.

It is expected that the PR5 Subcategory will be used on the Collector Roads (refer to the ITA) 01, 02 and 03 and in the commercial area roads 103, 104, 105, 106 and 161. PR6 is expected to be suitable for all other roads. This will be determined in detailed design, taking account of light location and light spread.

6 Natural Hazard Assessment

6.1 Introduction

Geosolve Limited (GeoSolve) undertook a natural hazards assessment of Homestead Bay to assess the risk posed to the proposed development site from flooding, debris flow, lake seiche, rock fall and rock avalanche. Their report is attached in Appendix E.[89] Hazards internal to the site e.g., liquefaction and slope stability are assessed in their Geotechnical Assessment report.[90] Their report is attached in Appendix G. Drainage and flood hazard mitigation within the proposed subdivision itself (from rainfall falling directly onto the site) have been assessed in Section 4 of the current report, which was partially informed by GeoSolve's assessment of the upstream catchments as described in their Natural Hazards Assessment report.[91]

The following sections summarise key aspects from both of GeoSolve reports, which can be consulted for further details.

A review of the Queenstown Lakes District Council (QLDC) register hazard maps has been conducted for Homestead Bay. The mapped hazards are shown in Figure 1a, Appendix A of their register. Further, interpretation of the geomorphological mapping and aerial photography has concluded that the following natural hazards may affect Homestead Bay:

- Slope stability
- Liquefaction
- Alluvial fan, debris flow and flooding
- Rock fall
- Rock avalanche
- Strong ground motion associated with a seismic event.
- Lake seiche

Consequently, each potential hazard has been assessed and is discussed in detail in the sections below.

The Geosolve Limited (GeoSolve natural hazards assessment has been peer reviewed by Fluent Solutions Limited. Their peer review is attached in Appendix H.

6.2 Regulatory Requirements

GeoSolve's natural hazards assessment considered the acceptability of natural hazard risks in accordance with the Operative QLDC District Plan and proposed Otago Regional Council Regional Policy Statement (Objective 4.1 and Appendix 6 (APP6)).[92] Their risk assessment methodology and APP6 are appended in Appendix D of their report.[93]

6.3 Overview

Key elements from GeoSolve's investigations are reported in this section.

For the purpose of the assessment, the site has been divided into three generalized geomorphic zones, as shown in Figure 6-1.

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[89] GeoSolve. *Natural Hazard Assessment – Homestead Bay Queenstown*. March 2025.
[90] GeoSolve. *Geotechnical Assessment for Resource Consent – Homestead Bay Queenstown*. March 2025
[91] Geosolve. *Natural Hazard Assessment*. March 2025. 8.
[92] Otago Regional Council. 2021. *Proposed Otago Regional Policy Statement. Integrating the management of Otago's natural and physical resources*. Published June 2021
[93] Geosolve. *Natural Hazard Assessment*. March 2025. Appendix E.

- Zone A – Fan Alluvium
- Zone B – Lake Sediments and Beach Deposits
- Zone C – Glacial Till/Outwash Deposits

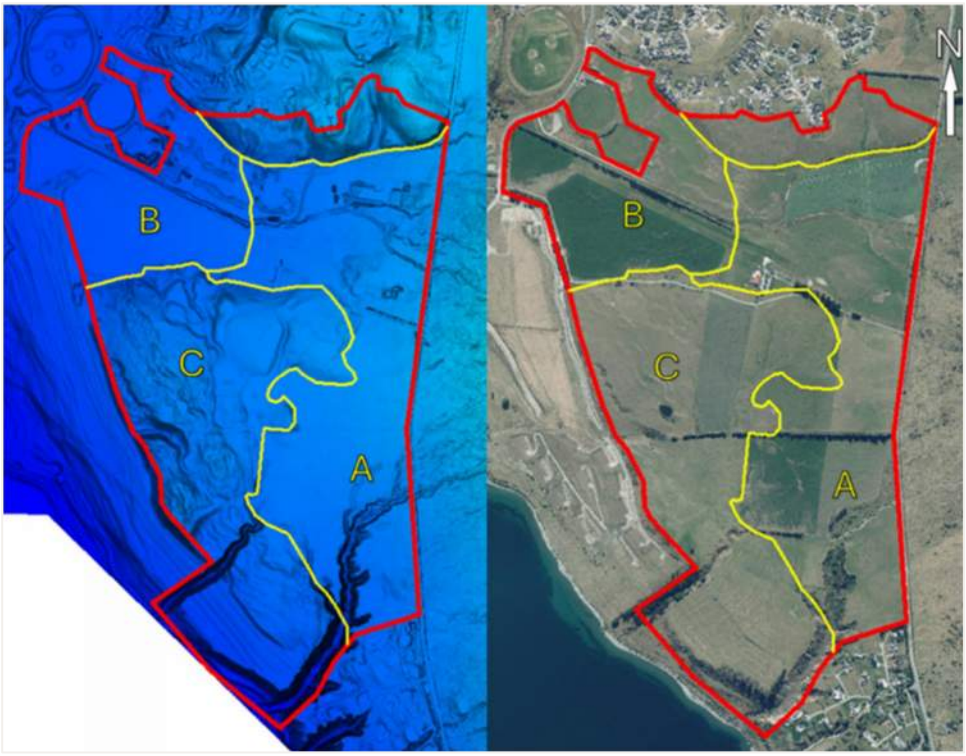

Figure 6-1: Generalized Geomorphic Zones

6.3.1 Slope Stability

GeoSolve found no signs of shallow or deep-seated slope instability within the site with the exception of the southern creek channels where localised shallow instability, stream bank erosion and slope crest regression are evident.[94]

Crest slope regression is likely to be ongoing for the southwestern terrace slope and the southern creek channels. Accordingly, Geosolve undertook a detailed slope stability assessment and provided preliminary building setbacks in Table 6.3 of their report.[95] The setbacks will have a minimum 3H:1V from the base of the slope in compliance with NZS 3604.

6.3.2 Liquefaction Hazard

The majority of the site, in particular Topographic Zones A and C are mapped as LIC 1 (P) / Classification A on the QLDC hazard maps, indicating a probably low risk of liquefaction.[96] The north-western extent of the site (Topographic Zone B) is mapped as LIC 2 (P) / Class B indicating a possibly moderate risk of liquefaction. The QLDC hazard mapping is shown in Figure 6-2.

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[94] GeoSolve. *Geotechnical Assessment*. March 2025. 17.
[95] GeoSolve. *Geotechnical Assessment*. March 2025. 18.
[96] GeoSolve. *Geotechnical Assessment*. March 2025. 13.

Figure 6-2: Liquefaction hazard mapping (Image retrieved from QLDC GIS Natural Hazards Map on 17 January 2024). The site boundary is shown in red)[97]

Figure 1b in Appendix A of GeoSolve's Geotechnical Assessment report identifies the approximate extent of the eastern perched water zone.[98] Therefore, most of the site, where there is no perched groundwater level, will have at least 10 m of dry non liquefiable crust. In these areas the liquefaction risk will be low for a standard shallow foundation structure (i.e. MBIE TC1 equivalent). The perched groundwater level is highly variable with seepages running through more permeable deposits in the alluvial fan.

The seismic scenarios considered are presented in Table 6-1.

Table 6-1: Seismic Scenarios Considered for Liquefaction Assessment[99]

Scenario	Performance Requirements	Annual Probability of Exceedance
Serviceability Limit State (SLS)	Avoid damage that would prevent the structure being used as originally intended without repair.	1/25
Intermediate Event - IL2	No requirements but assists in considering affects as per liquefaction planning guidelines.	1/100
Intermediate Event – IL3	No requirements but assists in considering affects as per liquefaction planning guidelines.	1/250
Ultimate Limit State (ULS) – IL2	Avoid collapse of the structural system.	1/500
Ultimate Limit State (ULS) – IL3	Avoid collapse of the structural system.	1/1000

[97] GeoSolve. *Geotechnical Assessment*. March 2025. 14.
[98] GeoSolve. *Geotechnical Assessment*. March 2025. Appendix A.
[99] Pertinent aspects extracted from GeoSolve's *Geotechnical Assessment* report. March 2025. 15.

A summary of the results of GeoSolve's liquefaction modelling is as follows:[100]

- Limited liquefaction occurs in the SLS and the IL2 intermediate design event in all groundwater cases. Therefore, for standard structures the risk for these events is low.
- If the groundwater/point of full saturation is at 2 m depth the IL3 intermediate design event does show that liquefaction is occurring but LSN and indexed settlement infer that moderate damage is expected. If the groundwater/point of saturation is one of the deeper cases limited/minor liquefaction risk is expected. An IL3 structure would require specific engineering design, so a development perspective in all cases there would be numerous design options to accommodate this risk.
- If the groundwater/point of full saturation is at 2 m depth liquefaction is calculated to occur in the ULS design event which could be moderate to major. However, if the groundwater was at lower depths minor to moderate liquefaction risk would be expected.
- Based on the variability of the groundwater levels and lack of free faces in close proximity to the site we consider that lateral spreading will not govern any liquefaction design. However, this should be re-assessed when the extent of earthworks, and presence of any proposed channels/basins, is understood.

A summary of GeoSolve's liquefaction analysis is as follows:[101]

- Given the relatively deep regional groundwater level, where there is no perched groundwater, the liquefaction risk has been assessed to be low to very low or MBIE TC1 equivalent for most of the site.
- In the elevated eastern area of the site, where perched groundwater is present, the liquefaction risk has been assessed as medium vulnerability. As there is no SLS liquefaction risk and no to minor intermediate risk, it is likely that common specifically designed foundation solutions could be developed to mitigate any liquefaction effects, such as robust foundation slabs similar to MBIE TC2 slabs.
- The above assessment is appropriate for Resource Consent and has shown that development is possible and the liquefaction risk over most the site is relatively low.
- There are some aspects of potential conservatism in the liquefaction risk given the soil types observed.

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[100] GeoSolve. *Geotechnical Assessment*. March 2025. 16.
[101] GeoSolve. *Geotechnical Assessment*. March 2025. 16-17.

6.3.3 Alluvial Fan Hazard

6.3.3.1 Introduction

Figure 6-3 shows the location existing creeks in relation to the boundary of Homestead Bay.

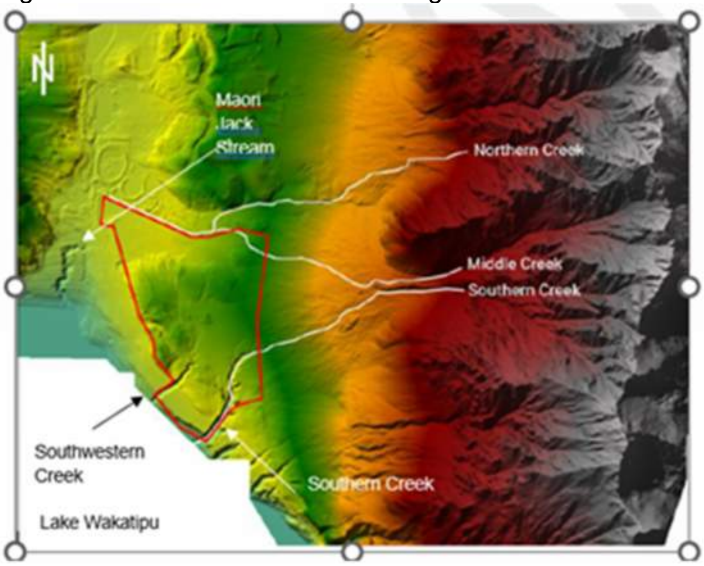

Figure 6-3: Existing Creeks in Relation to Homestead Bay

The catchment boundaries for each creek used in the assessment were developed therefrom and are shown in Figure 6-4 and defined in Table 6-2

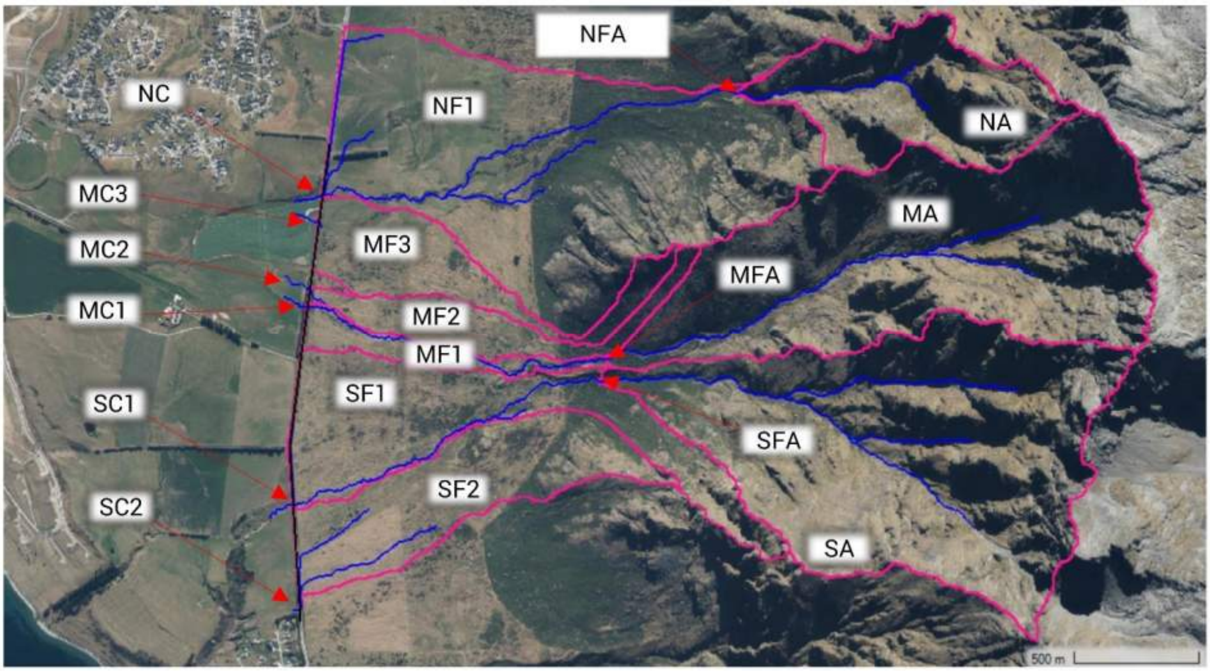

Figure 6-4: Catchment Boundary Extents

Table 6-2: Catchment Boundary Naming Convention

Label	Description
NA, MA, SA	Northern, Middle & Southern Alpine catchments
NFA, MFA, SFA	Northern, Middle & Southern Fan Apex
NF, MF, SF	Northern, Middle & Southern Fan catchments
NC, MC, SC	Northern, Middle & Southern Road Culverts (catchment outlets)

There are many other shallow ephemeral streams scattered across Homestead Bay. The Southern Creek flows across the site from east to west and terminates at Lake Wakatipu. The Middle and Northern Creeks also flow towards the west close to the northern boundary of the development and discharges into Lake Wakatipu outside the development extents.

The proposed development is located within an alluvial fan hazard area marked as ‘‘active, floodwater-dominated” based on regional scale mapping[102] as shown in Figure 6-5. Evidence of recent flooding was not observed during site visits. Observation of recent scour and erosion were limited to small areas on over-steepened banks adjacent to current flow paths

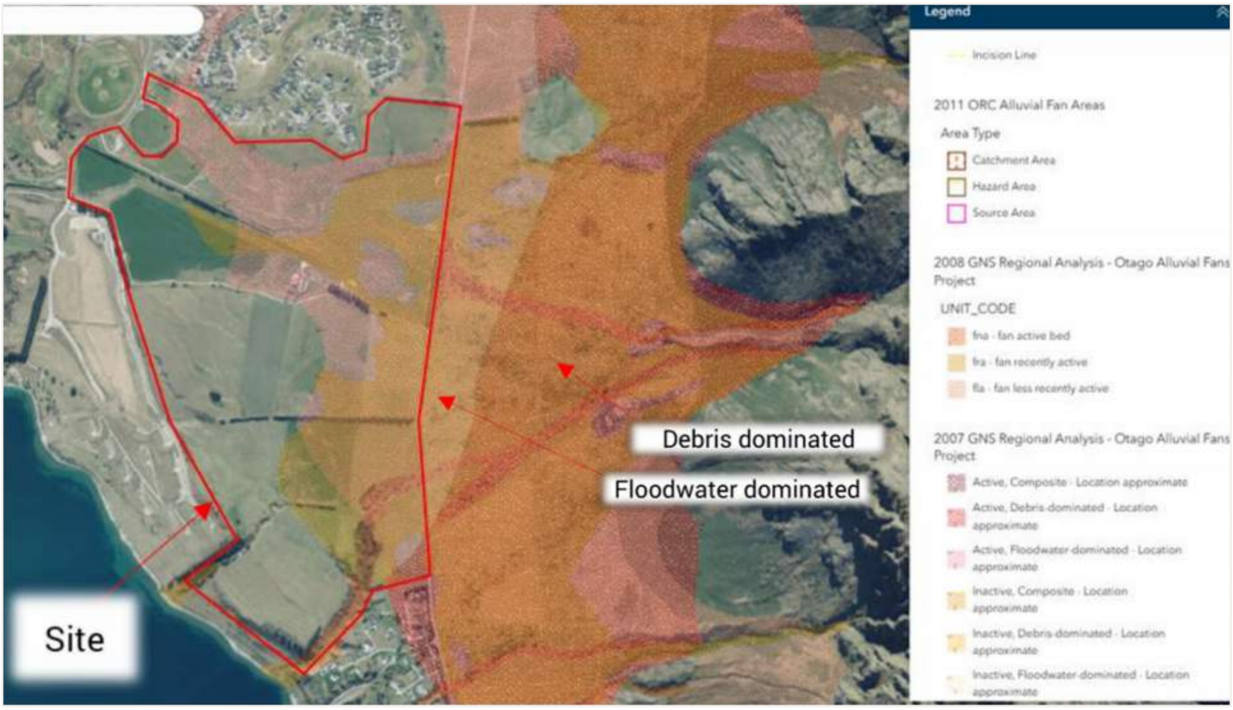

Figure 6-5: QLDC hazard mapping[103]

Pre-development and post-development alluvial flood hazard (especially in regard to the proposed mitigation measures) across the site has been assessed for three likelihood scenarios. Consequence and likelihood have been considered in order to qualitatively assess the risk.

The change in discharge at the downstream site boundary as a result of the proposed diversion channel has also been assessed; as well as the potential for flooding to reach roads and neighbouring properties.

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[102] OPUS (2009) Otago Alluvial Fans Project. Report #1205, Version 2.
[103] GeoSolve. *Natural Hazards Assessment*. March 2025. 6.

6.3.4 Alluvial Fan Flooding Risk Assessment

6.3.4.1 Pre-development Scenario

Table 6-3 presents the flood hazard level definitions used by GeoSolve for assessment purposes. They are based on depth and velocity of debris flow. Details can be found in their report.[104]

Table 6-3: Flood Hazard Level Definitions

Flood Hazard Level	Description
H1	Generally safe for people, vehicles and buildings
H2	Unsafe for small vehicles
H3	Unsafe for vehicles, children and the elderly
H4	Unsafe for people and vehicles.
H5	Unsafe for vehicles and people. All buildings vulnerable to structural damage. Some less robust building types vulnerable to failure.

Debris flood hazard for a 100-year ARI ('Likely') debris flood event in existing flow paths is shown below in Figure 6-6. Additional results for a 20-year ARI ('Almost Certain') and 250+ year ARI ('Possible') pre-development scenarios, including avulsion, are shown in Appendix F of GeoSolve's Natural Hazards report.[105]

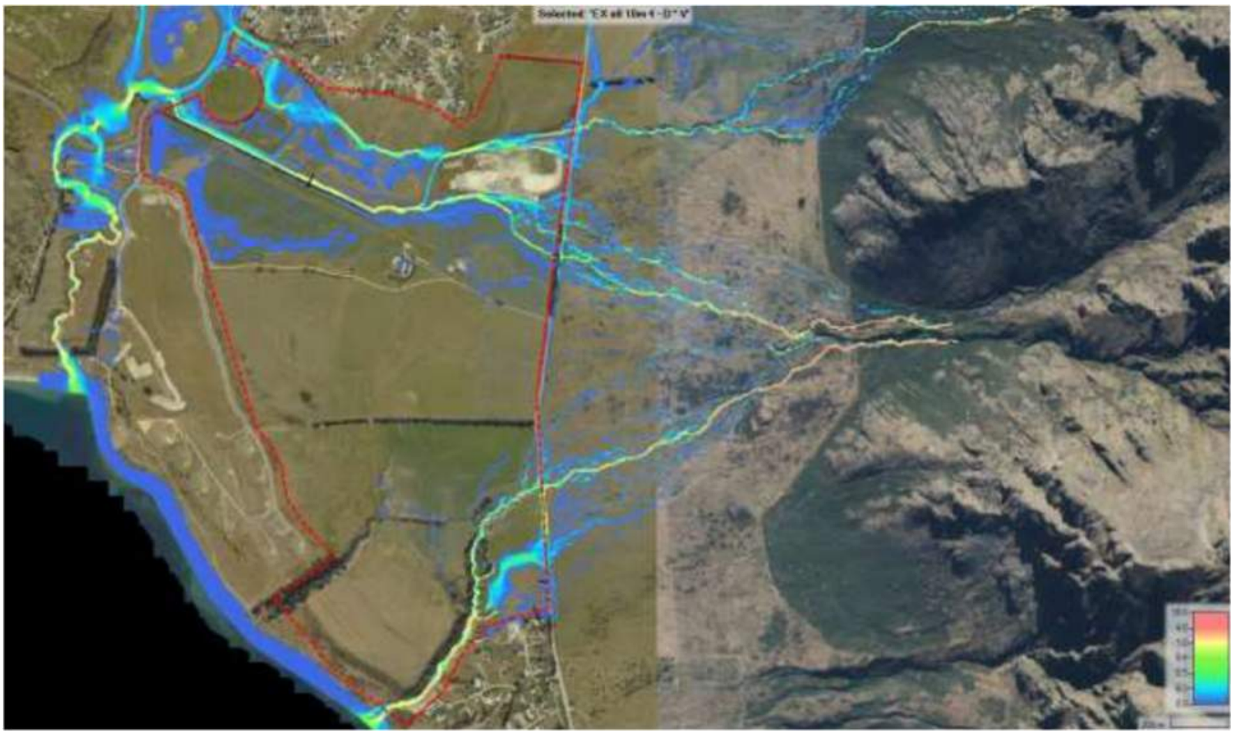

Figure 6-6: Debris flood hazard for a 'Likely' event, which is a 100-year ARI with conservative input parameters (site boundary is shown in red)[106]

For all scenarios a H5-H6 flood hazard is shown in the primary northern flow path along the northern boundary of Lot 8 and the incised southern flow path through the site. It is understood that development

[104] GeoSolve. *Natural Hazards Assessment*. March 2025. 25-26.
[105] GeoSolve. *Natural Hazards Assessment*. March 2025. Appendix F.
[106] GeoSolve. *Natural Hazards Assessment*. March 2025. 27.

is limited to public trails and reserve in these areas based on the provided development plans. Residential or mixed-use development in these areas is assessed to increase the consequence of a debris flood in these areas to moderate-major and is not recommended. Provided residential/mixed-use development is not undertaken in these areas, the consequence of flood flows in these channels paths is assessed as insignificant.

Without mitigation measures, the flood hazard is shown as H3-H5 for debris flooding in the northeastern corner of Lot 8. Mixed-use development is proposed in this area based on provided development plans and mitigation measures are therefore recommended.

Without mitigation measures, the flood hazard is shown as H3-H4 for debris flooding in the southeastern corner of Lot 8. Residential development is proposed in this area based on provided development plans, and mitigation measures are therefore recommended.

Development in these areas is assessed to have a consequence of minor for a likely debris flood event up to moderate for a possible event if not mitigated. Mitigation in the form of diversion channels along the northern eastern and southern site boundaries is recommended and details of the proposed diversion is given in Section 6.3.4.2.

Shallow sheet flow with the lowest possible hazard classification of H1 is shown across other areas of the development site. It is considered that this would be prevented from reaching those areas of the site by the proposed diversion channels.

6.3.4.2 Post-development Scenario

To mitigate flood hazard to the site from the upper catchments, diversion channels are proposed to direct flows from upstream catchments around the development areas. Diversions proposed are shown as blue arrows below in Figure 6-7.

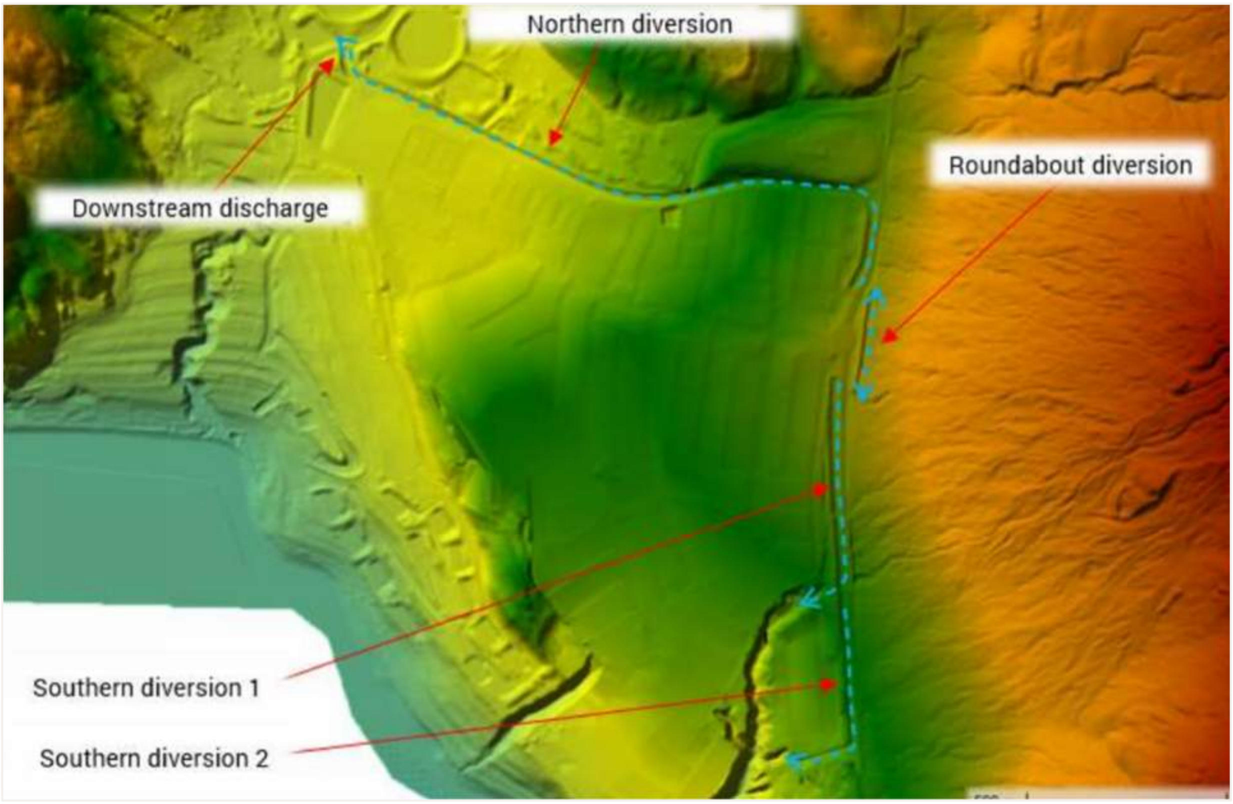

Figure 6-7: Proposed diversion channels as a DEM

Post-development model output is shown in Figure 6-8.

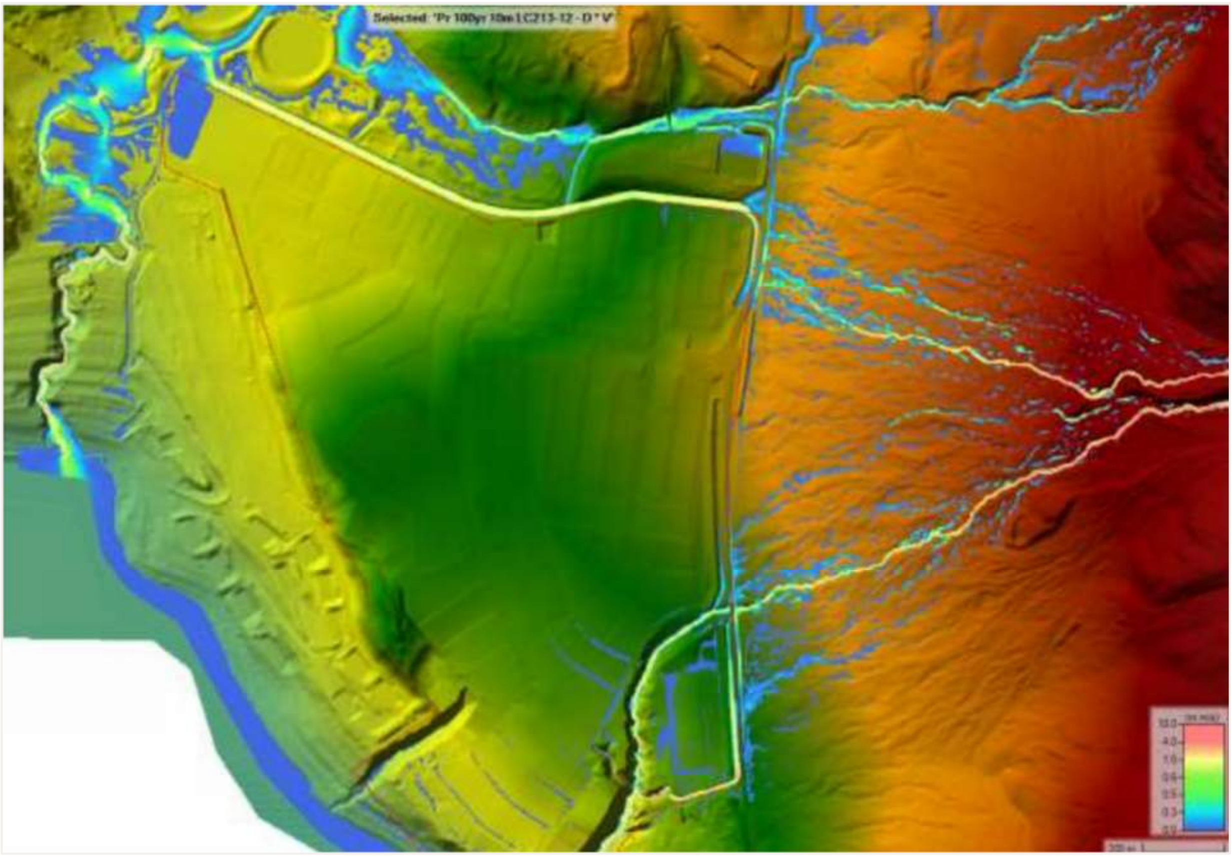

Figure 6-8: Post development debris flood hazard for a 100-year ARI design event with conservative input parameters (Lot 8 boundary shown in red). Note that the <0.1 m depth flow adjacent to channels within the development is considered to be due to minor "leakage" between computational cells, not anticipated to occur.[107]

This hazard assessment addresses the flooding hazards from the upslope eastern catchments and the efficacy of the proposed diversions with regard to mitigation of alluvial fan flooding hazard. The southern diversion channels will not receive any runoff originating from within the site or neighbouring land to the south. As such, the assessment of capacity in this report is considered to represent all flow contributions for these channels.

Assessment of all proposed diversion channels and potential flooding impacts along the development boundaries are discussed in Section 4.

It's clear from Figure 6-8 that the proposed diversion channels are appropriate to mitigate the risk of Alluvial Fan flooding.

6.3.5 Debris Flow Hazard

6.3.5.1 Introduction

Debris flows are a mass movement process that consists of a fully saturated liquefied mass of water and sediment. Once moving, the mixture of sediment and water behaves as a very dense liquid due to high pore pressure. Therefore, a debris flow is far more mobile than a non-saturated debris avalanche and can travel much greater distances.

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[107] GeoSolve. *Natural Hazards Assessment*. March 2025. 31.

Three likelihood scenarios have been identified for assessment of debris flow risk:

1. Likely (100 years)
2. Possible (100-1000 years)
3. Unlikely (1000-2500+ years)

The use of the relatively high return period events for all debris flow is considered appropriate. This is due to the relative magnitude of debris flow required to present a risk to the site and lack of recent debris flow activity. Shorter return period events were found to be even less likely to reach the site.

No recent debris flow deposits are present of the fan. Observed debris flow deposits may date back as far as the immediate post-glacial period. The fan has undergone post-depositional modification, including erosion, reworking, or burial by later flows.

6.3.5.2 Results from RAMMS Modelling

RAMMS debris flow software has been used to model potential debris flow paths and run-out distances. RAMMS is a numerical simulation model developed to calculate the motion of geophysical mass movements (such as debris flows) from initiation to runout in three-dimensional terrain.[108]

Results for the unlikely events are shown below in Figure 6-9 and Figure 6-10, with the site boundary in red. As shown, even the conservative Unlikely debris flow events for the Northern catchment do not reach the road. This is consistent with field geomorphic mapping, empirical assessment of run-out distance and existing hazard mapping (Figure 6-5 and Table 6-4)

[108] RAMMS (2022) Debris Flow User manual

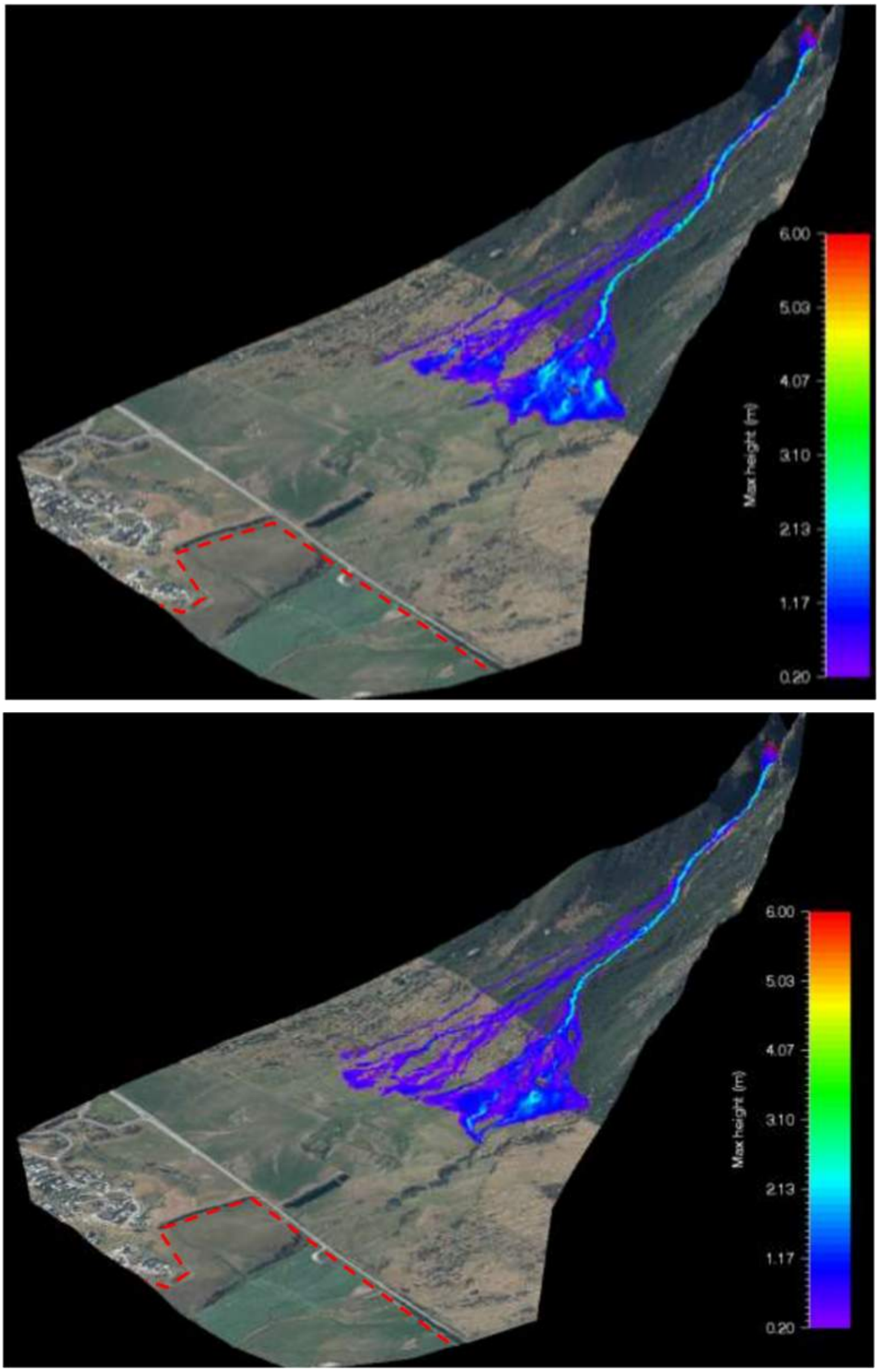

Figure 6-9: RAMMS result showing maximum depth of an unlikely debris flow event released from the Northern fan apex

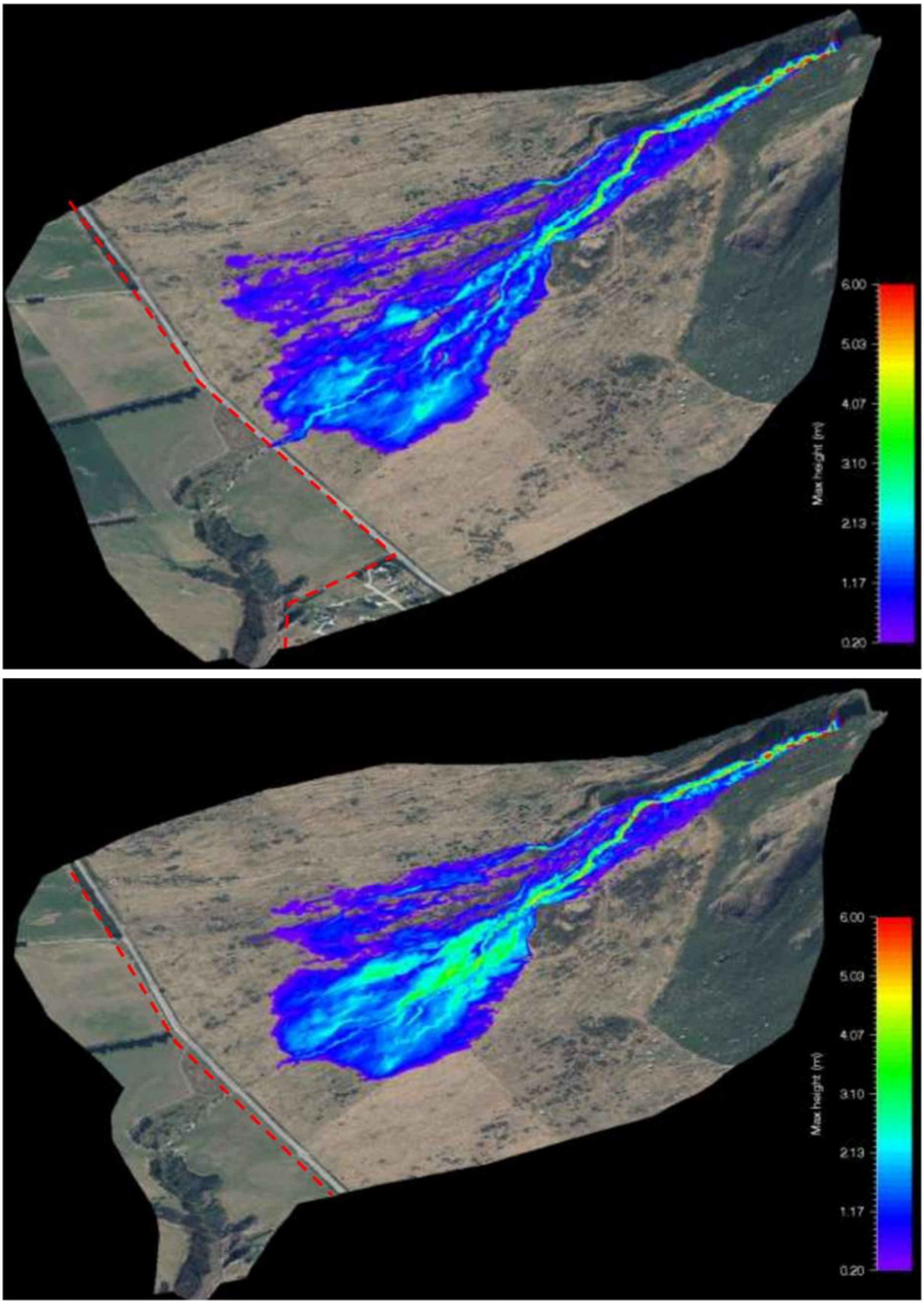

Figure 6-10: RAMMS result showing maximum depth of a unlikely debris flow event released from the southern fan apex

Table 6-4: Debris Flow run-out distance Summary

Release point	Maximum runout distance for an unlikely event (distance to site)			
	Direct observation	Hazard mapping	Empirical	RAMMS, u=0.15
Northern fan apex	~800 m	~1200 m	531 m	850 m (600 m)
Middle fan apex	~500 m	~650 m	616 m	650 m (250 m)
Southern fan apex	~700 m	~1000 m	754 m	1150 m (130 m)

For the Middle and Southern catchments, a debris flow event reaching the road is shown to be feasible for the conservative Unlikely event, but only when a very high mobility is assumed in modelling. In addition, significant flow depths are predicted to be confined to the primary channel at the road. See Figure 6-11. Flow is also shown to not run onto site even without the flow diversions/flood mitigation measures proposed

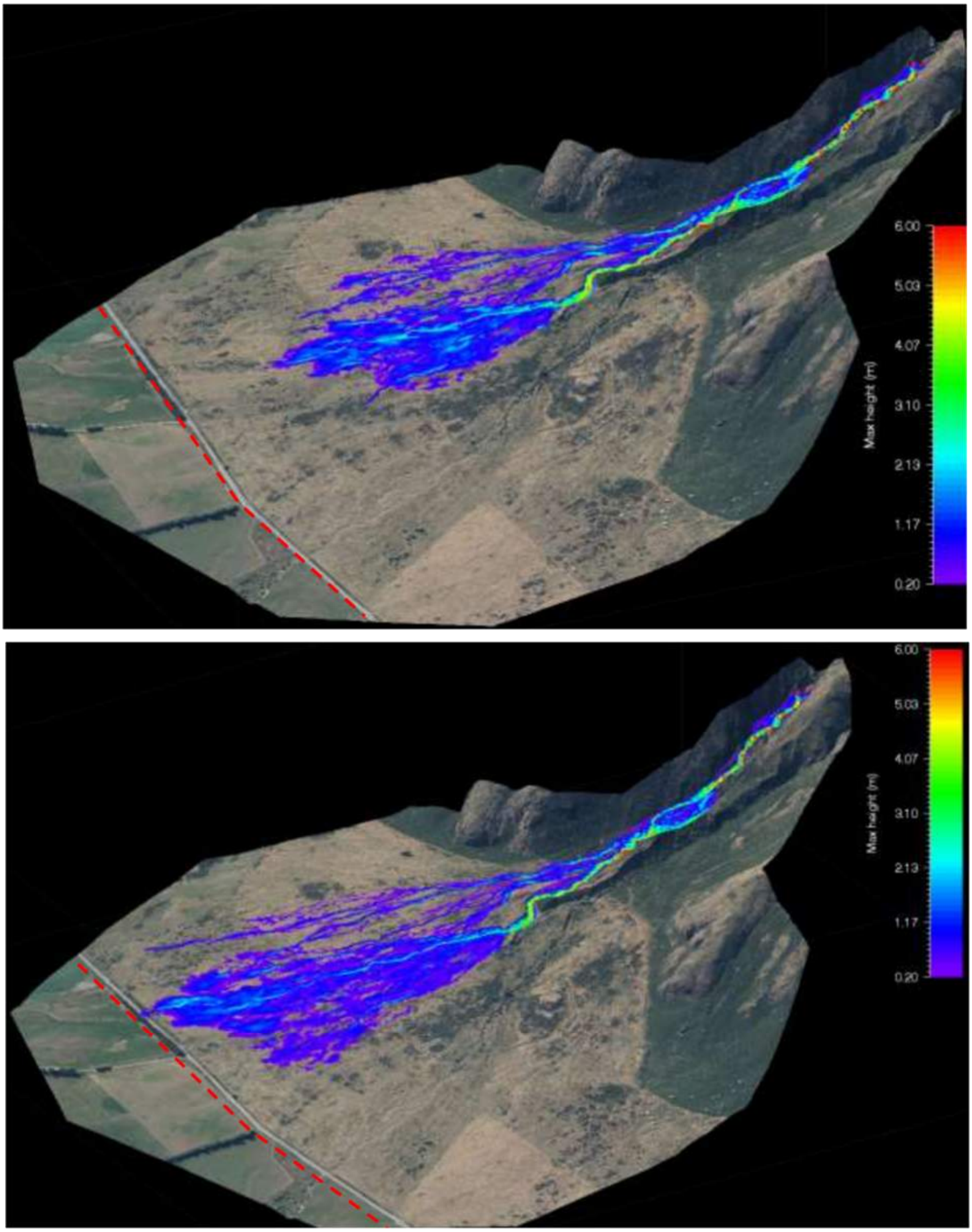

Figure 6-11: RAMMS result showing maximum depth of an unlikely debris flow event released from the Middle fan apex

6.3.5.3 Debris Flow Hazard Risk Summary

Three likelihood scenarios have been identified for assessment of a debris flow hazard. The results from all 3 scenarios are considered in the qualitative risk assessment discussed below.

As the Likely and Possible events are not shown to reach the road for any flow path, the hazard resulting from the **Unlikely** event is used to assess the consequence.

For debris flows originating from the northern fan apex, the consequence is assessed as **insignificant** as the debris flow is not shown to reach the site. As a result, the risk from this flow path is assessed as **Acceptable**.

An **Unlikely** debris flow originating from the Middle and Southern fan apex is shown to stop at or above the site boundary. In addition, significant depth is present only in the immediate vicinity of the channel immediately upstream of the road. The consequence is assessed as **insignificant**.

The road is shown to dissipate residual flow momentum, and the flood diversion channel and bunds proposed in Section 4 will provide additional protection to the development.

However, the risk from debris flow is considered Acceptable with or without proposed mitigation measures.

As the qualitative risk level is assessed as Acceptable a quantitative risk assessment is not considered warranted for debris flow hazard at this site.

6.3.6 Rockfall Hazard

No evidence of rock fall debris was observed inside the Homestead Bay development site, see Figure 6-12. The site has been farmed since publicly available aerial photos began (1959) and it is possible that boulders may have been removed to facilitate farm work.

GeoSolve identified potential for rock fall from the Remarkables Mountain east of the site. A rock fall hazard assessment has therefore been completed. Three assessment methods have been undertaken including, rock fall debris runout mapping, a shadow angle empirical assessment and 3D Rock Fall Analysis using RAMMS (RApid Mass Movement Simulation) modelling software.

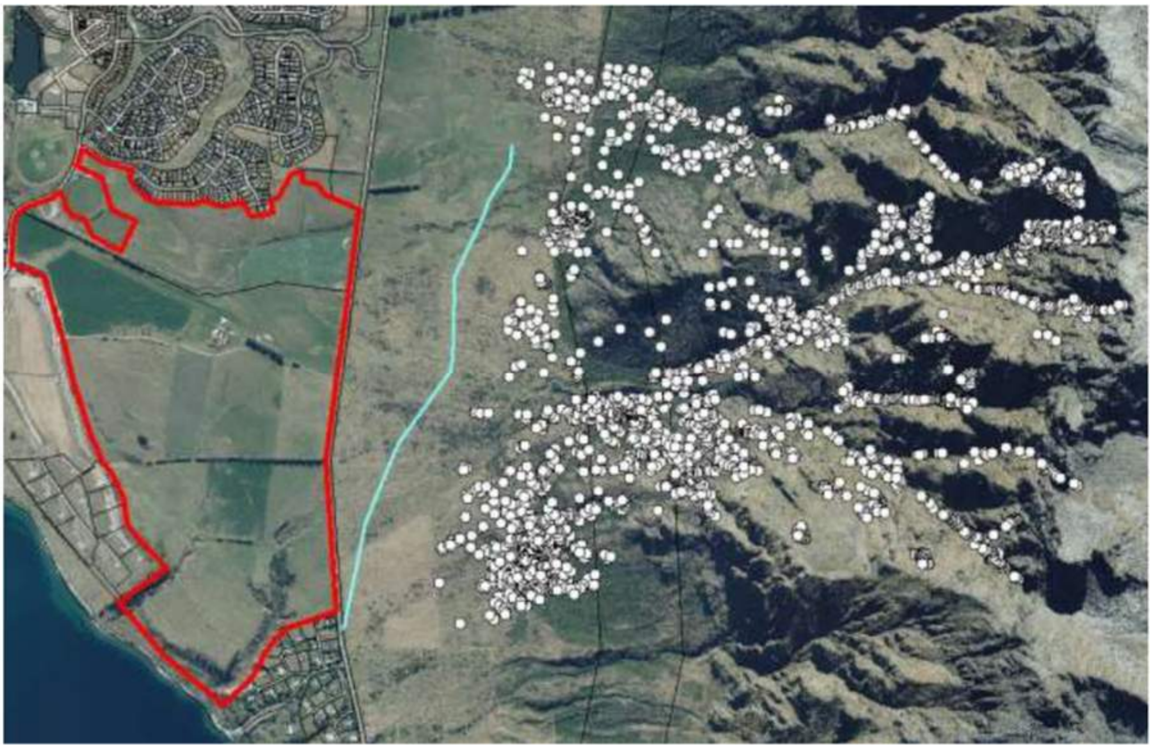

Figure 6-12: Plan showing the location of the observed rock fall debris boulders in white. The site boundary is shown in red. The 21-degree shadow angle line is shown in blue.[109]

The majority of the rock fall debris in the upper catchment was observed to have collected in the numerous channel and gully features (topographic funnelling). These channels have significant influence on the run-out direction of rock fall debris.

Rock outcrops were mapped by GeoSolve and potential fall source areas identified.[110] They then rock fall debris runout mapping.[111] The modelled rock fall trajectories for the analysis are shown in Figure 6-13, Figure 6-14, Figure 6-15, Figure 6-16 and Figure 6-17.

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[109] GeoSolve. *Natural Hazards Assessment*. March 2025. 56.
[110] GeoSolve. *Natural Hazards Assessment*. March 2025. 52-55.
[111] GeoSolve. *Natural Hazards Assessment*. March 2025. 55-55.

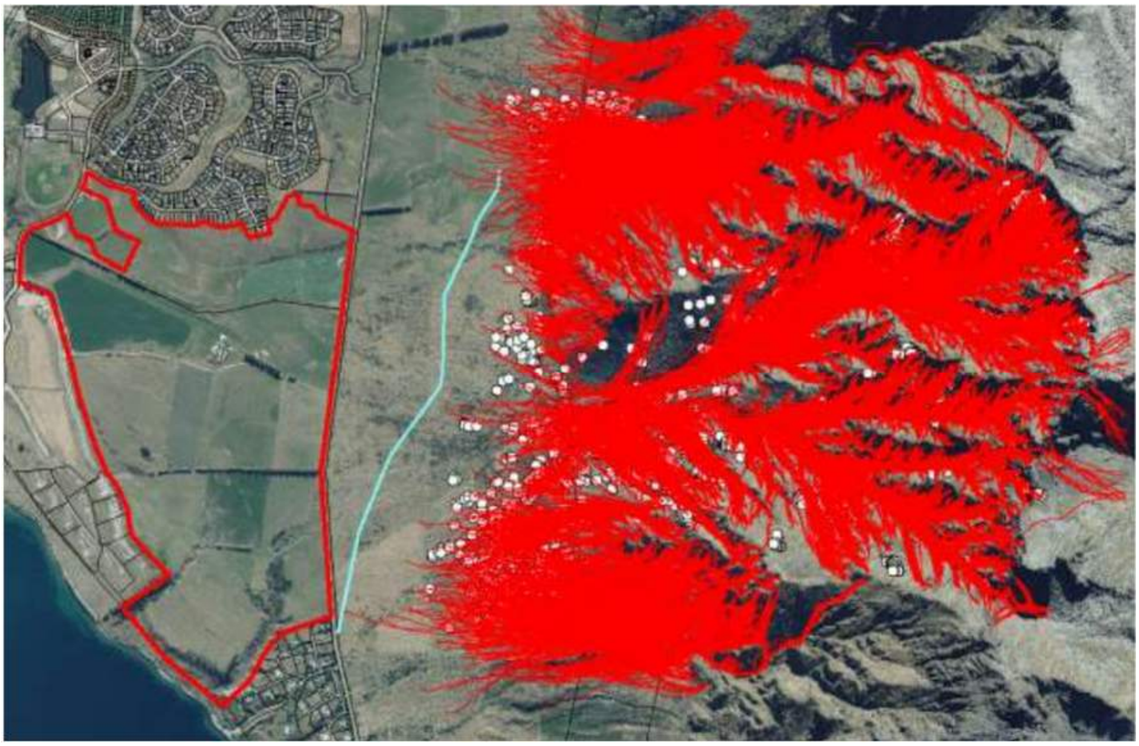

Figure 6-13: Modelled rock fall trajectory results (Analysis 1) including the proposed development area (red), the 21degree shadow angle (blue) and the mapped rock fall debris boulders (white dots)

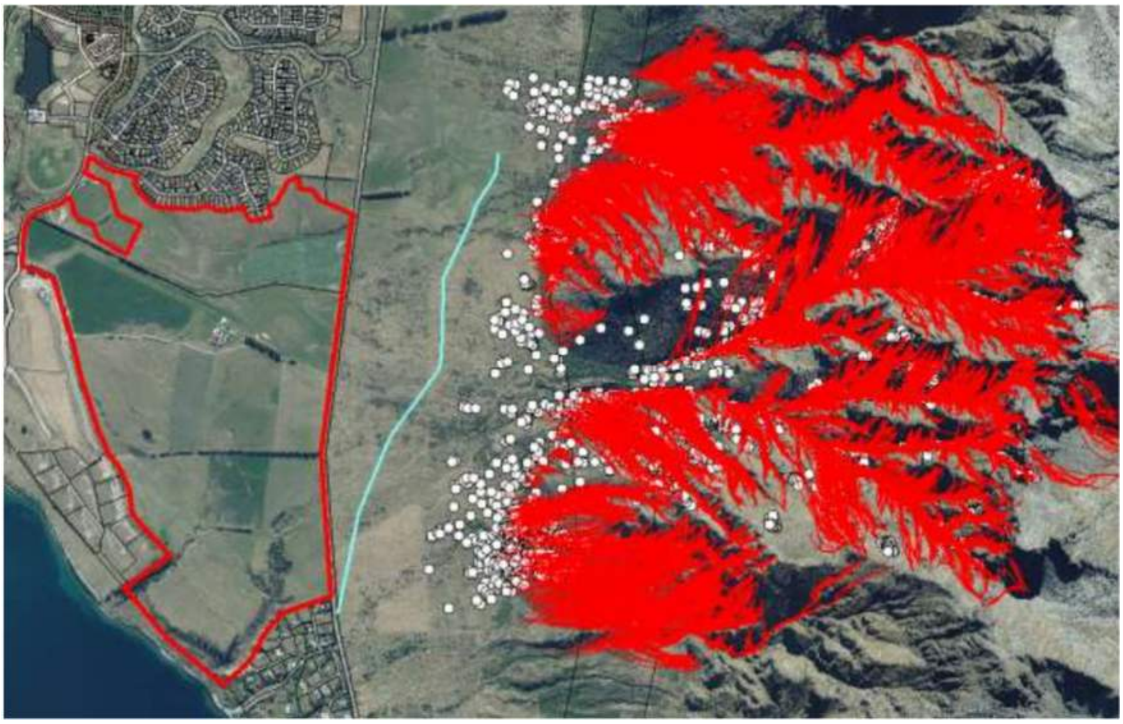

Figure 6-14: Modelled rock fall trajectories results (Analysis 2) including the proposed development area (red), the 21degree shadow angle (blue) and the mapped rock fall debris boulders (white dots)

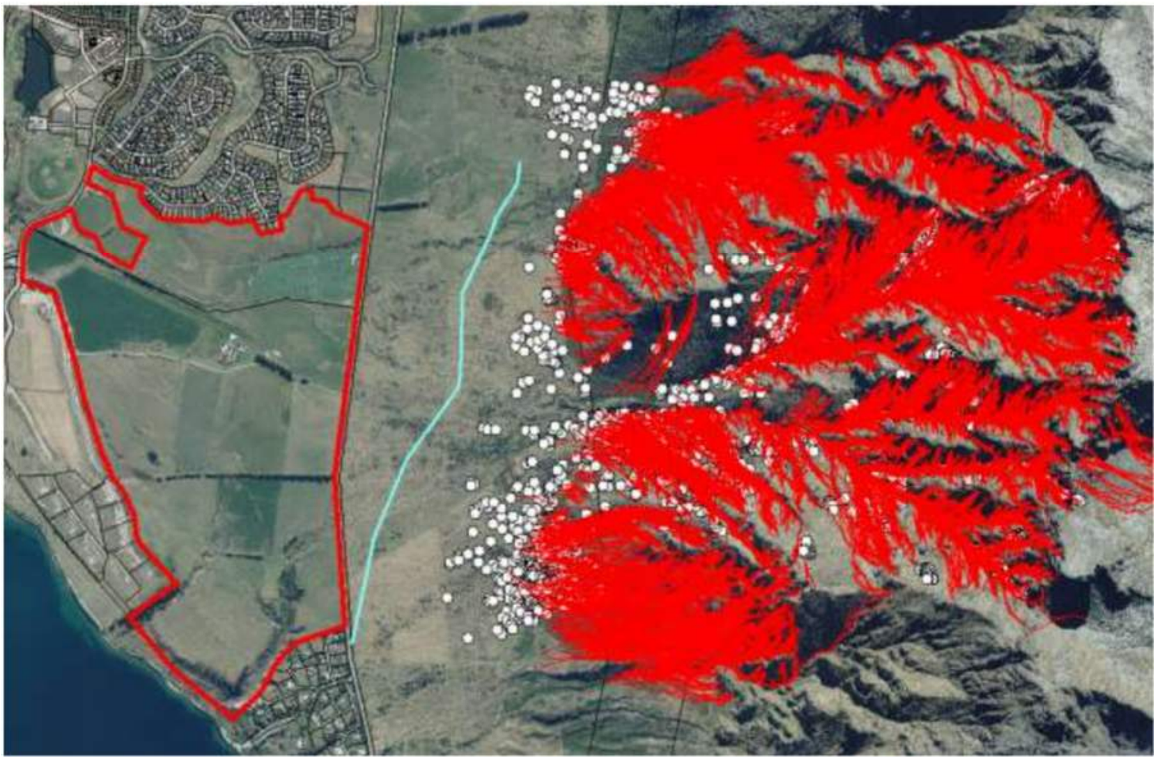

Figure 6-15: Modelled rock fall trajectories results (Analysis 3 & 5) including the proposed development area (red), the 21degree shadow angle (blue) and the mapped rock fall debris boulders (white dots)

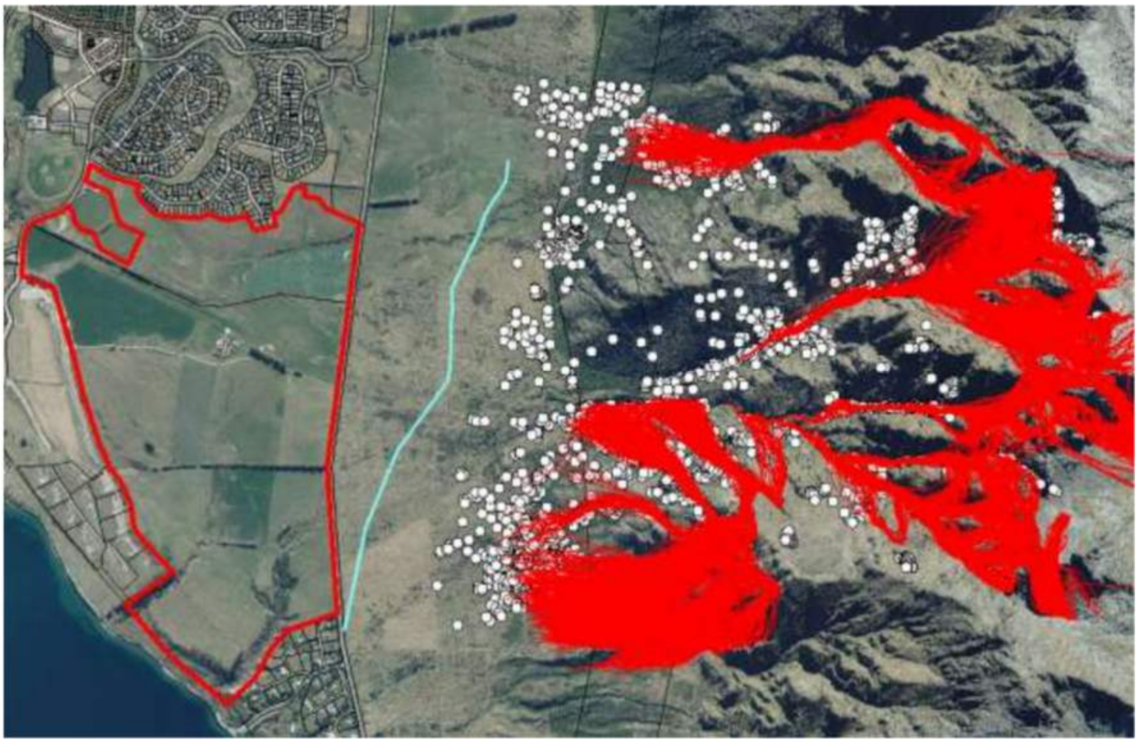

Figure 6-16: Modelled rock fall trajectories results (Analysis 4) including the proposed development area (red), the 21degree shadow angle (blue) and the mapped rock fall debris boulders (white dots)

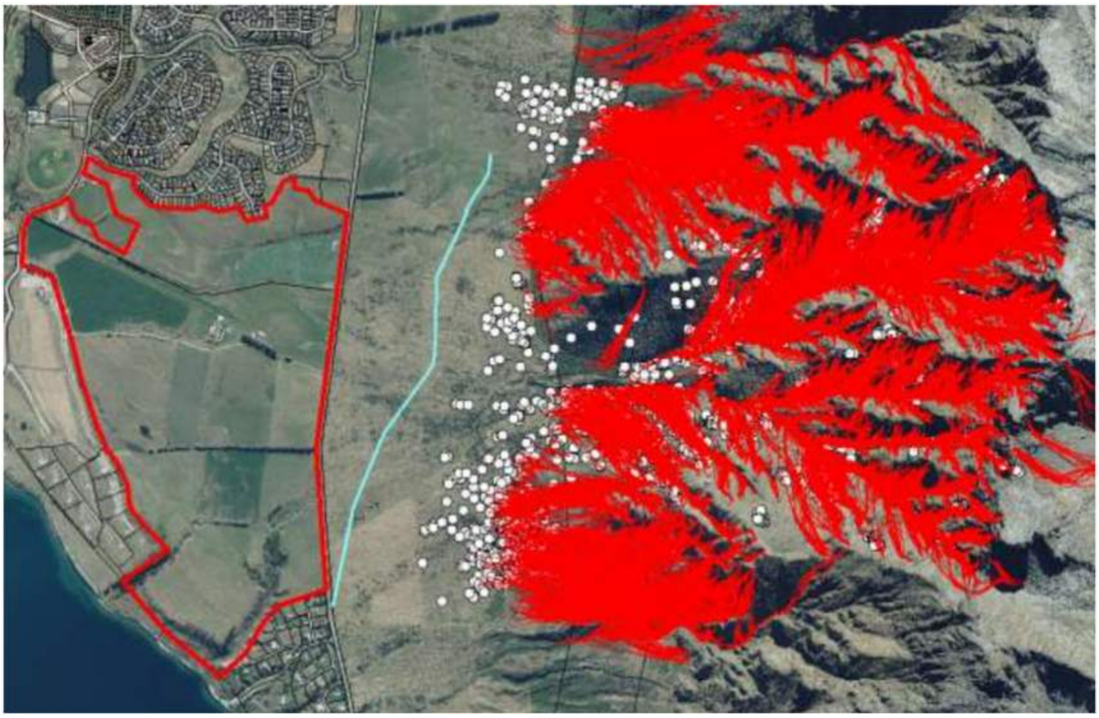

Figure 6-17: Modelled rock fall trajectories results (Analysis 5) including the proposed development area (red), the 21degree shadow angle (blue) and the mapped rock fall debris boulders (white dots)

The 3D RAMMS modelling shows that no rock fall trajectories will enter the eastern site boundary. The modelling is in agreement with the field geomorphic mapping and the empirical shadow angle assessment.

Based on the rock fall assessments described above and the qualitive risk assessment the rock fall risk at the site is considered Acceptable in accordance with APP6. No further assessment of this hazard is considered necessary for the Homestead Bay.[112]

6.3.7 Debris Avalanche Hazard

Cliff collapse is defined as a type of slope instability involving multiple boulders which inundate the ground beneath simultaneously in an event called a debris avalanche. Historic and active slope instability in the form of cliff collapse has been identified in 2 locations at the western end of ridgelines that extend west from the Remarkables. The two areas are identified as the Northern Cliffs and Southern Cliffs.

The Northern Cliffs are located at the end of the ridgeline which separates the northern and middle catchments, as shown in Figure 6-18.

[112] GeoSolve. *Natural Hazards Assessment*. March 2025. 63.

Figure 6-18: Photo showing the location of the Northern Cliffs in red taken from the corner of Kingston Road and North Zone.

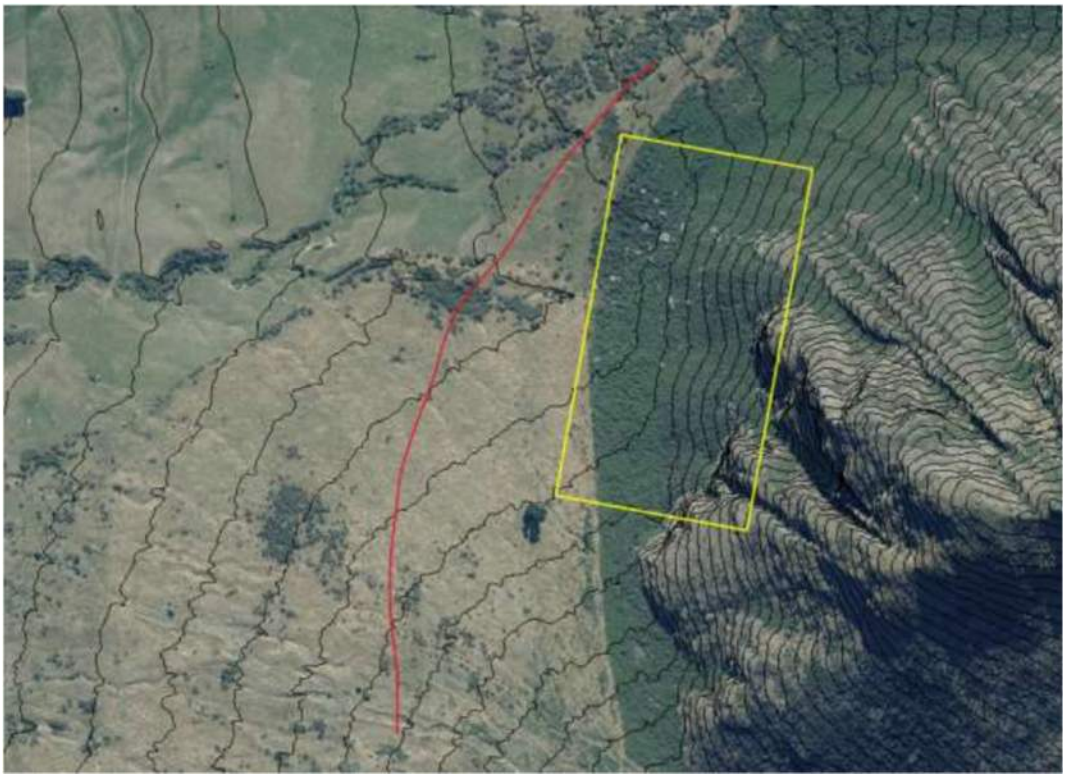

Figure 6-19: Annotated aerial photo showing the location of the inferred maximum extent of cliff collapse debris in red in relation to the Northern Cliffs. The rock fall debris area below some of the rock outcrops is highlighted in the yellow box.

The Southern cliffs are located on the western flanks of a generally southeast to northwest trending ridgeline on the true left side of the Southern Catchment.

Figure 6-20: Photo showing the location of the Southern Cliffs in red taken from Kingston Road

Figure 6-21: Annotated aerial photo showing the location of the inferred maximum extent of cliff collapse debris in red in relation to the Southern Cliffs. The rock fall debris area below some of the rock outcrops is highlighted in the yellow box.

GeoSolve undertook a qualitative risk assessment in accordance with the APP6.[113,114] . Three seismic events have been considered as triggers for a cliff collapse:

1. Almost certain (50 year ARI)
2. Possible (500 year ARI)
3. Unlikely (2500 year ARI)

Site observations and empirical assessment agree and show a 1/2500 year or smaller seismic event triggered rock avalanche originating from the Northern and Southern cliffs will not reach the site. The consequence from a debris avalanche to the site is therefore not considered significant for the development proposal.

The risk level is assessed as Acceptable for all three likelihoods considered. As the qualitative risk level is assessed as Acceptable a quantitative risk assessment is not considered warranted.

6.3.8 Lake Seiche Hazard Assessment

In the event of severe shaking, or sudden displacement of a large volume of lake water, the Lake may ‘slosh’ within its basin, potentially generating waves that may affect the site. This effect is known as ‘seiche’. Based on mountain topography and seismic environment it is inferred that such events may potentially have occurred around Lake Wakatipu in the geologic past.

There is no known record of destructive lake seiche events within historic human occupation, and on this basis, we consider the occurrence of such an event within the development’s lifetime to be of low probability. Alos, the site boundary is ~20 metres above the lake level and the lowest proposed development is ~30 m above lake level. A lake seiche in excess of this magnitude/height is likely to have a very high return period.

As the return period for lake seiche significant enough to reach the site is considered to be high it is deemed a **rare** event. For a **rare** lake seiche the consequence is conservatively assessed as moderate. However, due to the low probability, the risk from lake seiche is assessed as **Acceptable**.

As the qualitative risk level is assessed as **Acceptable** a quantitative risk assessment is not considered warranted for lake seiche hazard at this site.

6.3.9 Active Fault

No active fault traces are known to exist in the immediate vicinity of the site. However, numerous active faults have been identified within the region (Motutapu, NW Cardrona, Nevis, Pisa, Moonlight Faults). The nearest known active fault, the Nevis Fault, is located 19 km east of the site. The recurrence intervals for active faults in the region are assessed to be in the order of 5,500 to 120,000 years[115].

Significant seismic risk exists in the Lakes District Region from a rupture of the Alpine Fault which is located approximately 85 km NW of the site.[116] Strong ground shaking in the Lakes District region is expected during a rupture of the Alpine Fault. Recent research suggests there is a 75% probability of an Alpine Fault earthquake occurring within the next 50 years and an 82% probability that the next earthquake on the Alpine Fault will be of Mw8 or greater.[117]

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[113] Otago Regional Council. 2021. *Proposed Otago Regional Policy Statement. Integrating the management of Otago’s natural and physical resources*. Published June 2021.
[114] GeoSolve. *Natural Hazards Assessment*. March 2025. 68-71.
[115] Barrell, D.J., (2019). General distribution and characteristics of active faults and folds in the Queenstown Lakes and Central Otago districts, Otago. GNS Science Consultancy Report 2018/207. Published: March 2019
[116] Orchiston, C., Davies, T., et al. (2016) Alpine Fault Magnitude 8 Hazard Scenario. https://af8.org.nz/
[117] Howarth, J.D., et al. (2021). Spatiotemporal clustering of great earthquakes on a transform fault controlled by geometry. Nature Geoscience; doi: 10.1038/s41561-021-00721-4

7 Foundation Assessment

7.1 Introduction

The assessment of the foundation conditions at Homestead Bay was undertaken by GeoSolve in March 2025.[118] A copy of their report is in APPENDIX G. Key elements of their investigation are reported here.

The development proposal is mostly contained within Lot 8 which comprises pastural farmland, farm tracks and associated farming infrastructure, see Figure 7-1. The NZone skydiving company occupy an area in the northern portion of the site where a grassed airstrip and associated buildings are present.

Lot 12 remains largely unmodified under the proposal, however a reservoir structure is proposed in the north eastern area, see Figure 7-1. Associated pipework will head south to service the proposed development in Lot. 8.

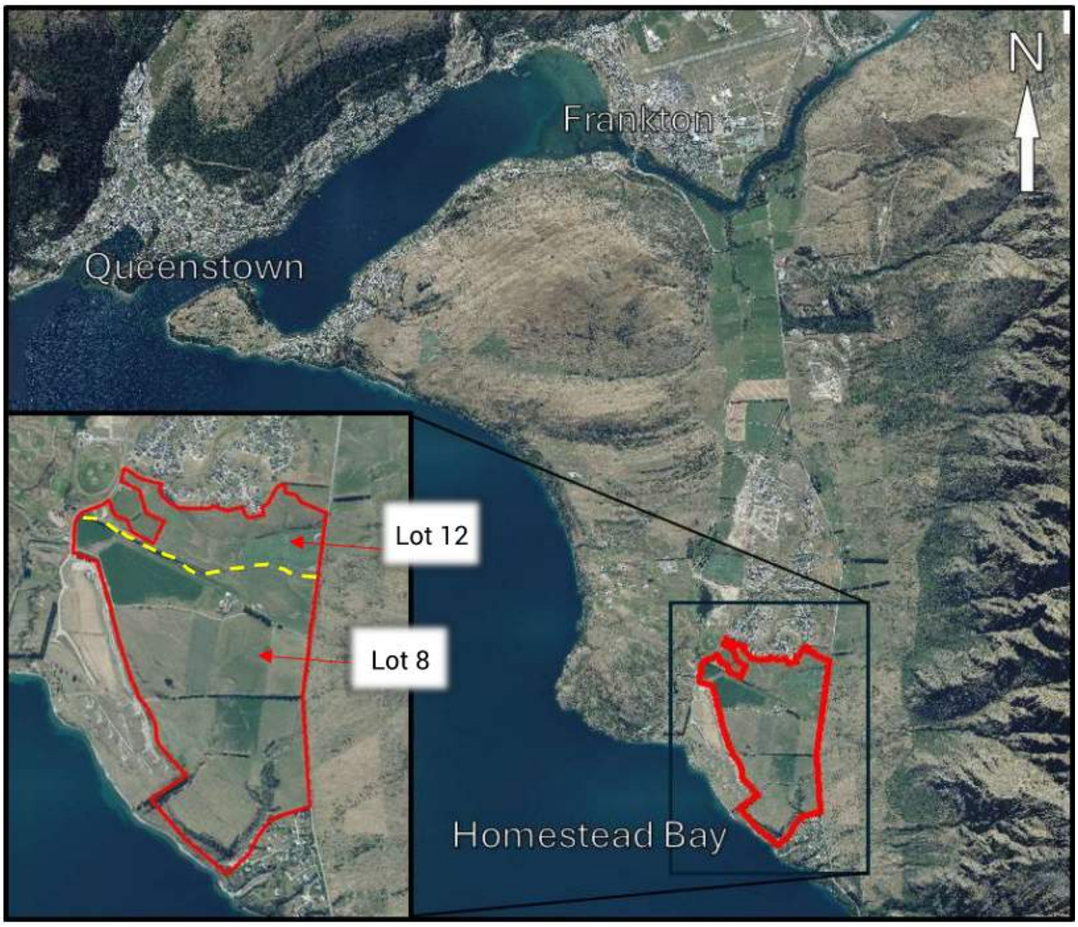

Figure 7-1: Site location plan showing Homestead Bay outlined in red. Aerial Image retrieved from QLDC GIS.[119]

[118] GeoSolve. *Geotechnical Assessment for Resource Consent*. January 2025.
[119] GeoSolve. *Natural Hazards Assessment Homestead Bay*. March 2025. 4.

7.2 Soil Stratigraphy

Site stratigraphy is discussed in Section 5.2 of GeoSolve's Geotechnical Assessment report.[120]

7.3 Groundwater

The regional groundwater table was measured in standpipe piezometers constructed in the boreholes (BH1 - BH3 and BH1-22 – BH2-22). Borehole locations are shown on the Investigation Site Plan, Appendix A Figure 1a pf GeoSolve's report.[121] Table 7-1 below summarizes the measured ground water levels.

Table 7-1: Site groundwater levels as observed in borehole piezometers[122]

Piezometer	Date Measured	Groundwater Depth m BGL	Groundwater m RL (NZVD2016)
BH1-22	2nd September 2022	14.8	339.2
	16th December 2024	13.4	340.6
	17th January 2024	15.4	338.6
BH2-22	2nd September 2022	5.1	386.9
	17th January 2024	4.3	387.7
BH1	16th December 2024	13.1	360.9
	17th January 2024	14.4	360.5
BH2	17th January 2024	10.5	380.5
BH3	17th January 2024	Dry	-

Groundwater seepages were encountered in several of the test pits in the eastern area of the site. All the test pits which encountered ground water seepages are location in elevated areas of the alluvial fan adjacent to the eastern site boundary. Seepage depths are summarised in Table 7-2 below.

[120] GeoSolve. *Geotechnical Assessment for Resource Consent*. January 2025. 10.
[121] GeoSolve. *Geotechnical Assessment for Resource Consent*. January 2025.
[122] GeoSolve. *Geotechnical Assessment for Resource Consent*. January 2025. 11.

Table 7-2: Groundwater seepages as observed in test pits[123]

Test Pit	Geology at Seepage Level	Groundwater Depth (m BGL)
TP7	Alluvial Fan Deposits	4.2
TP9	Alluvial Fan Deposits	3.1
TP14	Outwash Deposits	4.0
TP40	Alluvial Fan Deposits	1.3
TP41	Alluvial Fan Deposits	2.6
TP42	Alluvial Fan Deposits	1.9
TP47	Contact between Alluvial Fan Deposits and Glacial Till	0.9*
*Test pit was located immediately adjacent to flowing creek		

The regional groundwater table is expected to lie well below the finished development. Dewatering or other groundwater-related construction issues are therefore unlikely to be extensively required. Perched seepages have been identified in some eastern areas of the site at depths of 1 m – 5 m

Design and construction will incorporate aspects to mitigate the eventuality of groundwater seepage. These include incorporation of subsoil drains or cut-off catchment drains to intercept the perched seepage and discharge into the piped stormwater drainage system.

7.4 Assessment of Foundation Bearing Conditions

Following the removal of topsoil and uncontrolled fill, it is expected that predominantly alluvial fan deposits will be encountered in Zone A, beach deposits, lake sediments or alluvial fan deposits within Zone C, and colluvium, weathered glacial till, glacial pond sediment, glacial till or outwash deposits within Zone C.[124]

7.4.1 Zone A and B

Zones A and B are expected to be underlain by beach deposits, lake sediments and/or alluvial fan deposits. These deposits will provide a reduced bearing capacity and do not meet the ‘good ground’ bearing capacity requirements as outlines in NZS3604. Specific engineering design will be required for lots containing these soils to provide appropriate foundations solutions.

Preliminary assessment indicates robust foundation slabs similar to MBIE TC2 slabs may be required, depending on further detailed assessment of the liquefaction risk.

7.4.2 Zone C

Zone C is expected to be underlain by colluvium, glacial pond sediment, glacial till or outwash deposits. These materials provide a mix of ‘good ground’ and not ‘good ground’ with respect to bearing capacity as outlined in NZS3604. Specific engineering design will be required for lots containing these soils to

[123] GeoSolve. *Geotechnical Assessment for Resource Consent*. January 2025.12.
[124] Zone designations are shown in Figure 6-1.

provide appropriate foundations solutions.

7.5 Engineered Fill

All fill that is utilised as bearing for foundations or to form batter slopes should be placed and compacted in accordance with the recommendations of NZS 4431:2022 and certification provided to that effect.

The alluvial fan deposits, outwash deposits and glacial till are suitable for used as engineered fill on site (during good weather and in accordance with an earth fill specification). Boulders and cobbles over 100 mm in size will need to be screened from engineered fill sources. An earth fill specification should be provided prior to the commencement of earthworks.

GeoSolve recommend topsoil stripping and subsequent earthworks be undertaken only when a suitable interval of fair weather is expected.

7.6 Excavations

Preliminary recommended slope batters are provided in Table 7-3 below. The recommendations will be reviewed by GeoSolve following provision of proposed earthwork plans during the detailed design stage. All proposed batter slopes higher than 4 m or required to be steeper than the recommended angles will require specific engineering assessment by GeoSolve during the detailed design stage.

Table 7-3: Recommended maximum slope batters for temporary and permanent slopes

Material type	Recommended maximum batter for temporary slopes	Recommended maximum batter for permanent slopes up to 4 m high
Topsoil, Uncontrolled Fill, Beach Deposits	1(v):1.5(h)	1(v):3(h)
Loess, Alluvial Fan Deposits, Colluvium, Glacial Pond Sediment, Weathered Glacial Till	1(v):1.5(h)	1(v):2.5(h)
Glacial Till, Outwash Deposits	1(v):1(h)	1(v):2(h)

7.7 Aquifers

Geosolve found that no aquifer resource will be adversely affected by the development.

7.8 Neighbouring Properties

GeoSolve found that no adverse geotechnical implications apply for neighbouring properties during construction.

7.9 Conclusions

GeoSolve, following their geotechnical assessment, concludes that a mixed use development is acceptable at the Homestead Bay site from a geotechnical perspective. Standard engineering assessment and construction methods are available to construct the subdivision and develop the area.

8 Earthworks and Environmental Management

8.1 Earthworks General

8.1.1 Introduction

Large scale earthworks are required throughout the development in order to shape the existing ground to suitable grades and levels to allow for construction of the proposed infrastructure and provide useable building platforms for the planned development.

The Geotechnical Assessment for Resource Consent prepared by Geosolve (dated 21/01/25), see Appendix G , shows the site contains a variety of material types including alluvial fan deposits, beach deposits, lake sediments, uncontrolled fill, loess, colluvium, glacial pond sediment, glacial till and outwash deposits. A large volume of material in the alluvial fan deposits, outwash deposits and glacial till will be suitable to use as engineered fill. Other materials might be suitable for blending or will be cut to waste.

The earthworks operation will follow the recommendations presented in the Geosolve report and be in accordance with the project specific Construction Management Plan.

The extent of expected earthworks depths is shown on the drawings in Volume 3.

Table 8-1: Earthworks Volumes for Homestead Bay Development

Stage	Fill (m^3)	Cut (m^3)	Excess Cut (+) / Shortfall Fill (-) (m^3)
Full Site	1,052,500	1,239,400	**+186,900**

There is therefore currently an excess cut of 186,900m^3. It is expected that this excess can be reduced through a combination of amendments to finished levels during detailed design and modification to fills in non-lot areas such as the reserves and highway bunds in order to reach closer to a balance in overall earthworks. Any excess that does remain would be carted off site as clean fill.

8.1.2 Fill Earthworks

All engineered fills will be placed, compacted, and certified in accordance with NZS4431:2002.

Requirements for fill batters will be evaluated on a case-by-case basis but will generally be in accordance with NZTA F/1.

8.1.3 Excavations

Any uncontrolled fill identified during construction, typically when topsoil is removed, will be removed, and replaced with engineering fill and certified in accordance with NZS4431.

Batter slopes for temporary and permanent cuts up to 3m high are typically as shown in Table 8-2 for the soils identified on site as recommended by GeoSolve.

Table 8-2: Recommended Batters for Cuts up to 3 m in Height in Dry Ground

Material type	Recommended maximum batter for temporary slopes	Recommended maximum batter for permanent slopes up to 4 m high
Topsoil, Uncontrolled Fill, Beach Deposits	1(v):1.5(h)	1(v):3(h)
Loess, Alluvial Fan Deposits, Colluvium, Glacial Pond Sediment, Weathered Glacial Till	1(v):1.5(h)	1(v):2.5(h)
Glacial Till, Outwash Deposits	1(v):1(h)	1(v):2(h)

Steeper slopes or excavations of greater height will be specifically designed by a suitably qualified geotechnical engineer.

Perched groundwater was observed in some locations within the site so where ground water seepage is encountered during construction measures to manage groundwater to ensure stable excavations will be employed.

8.2 Construction Management (including Environmental Controls)

A draft project specific Construction Management Plan (CMP) has been prepared and is included separately in the application. This details the expected scope, staging, timing, environmental, safety and quality controls that will be in place during construction. This contains high level details of the Environmental Management that is proposed for the site and spells out the processes that will be followed for each stage of construction to ensure all works are completed with minimal impact on the internal and surrounding environment.

Stantec is a global leader in sustainable engineering, architecture, and environmental consulting. The diverse perspectives of our partners and interested parties drive us to think beyond what's previously been done on critical issues like climate change, digital transformation, and future-proofing our cities and infrastructure. We innovate at the intersection of community, creativity, and client relationships to advance communities everywhere, so that together we can redefine what's possible.

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