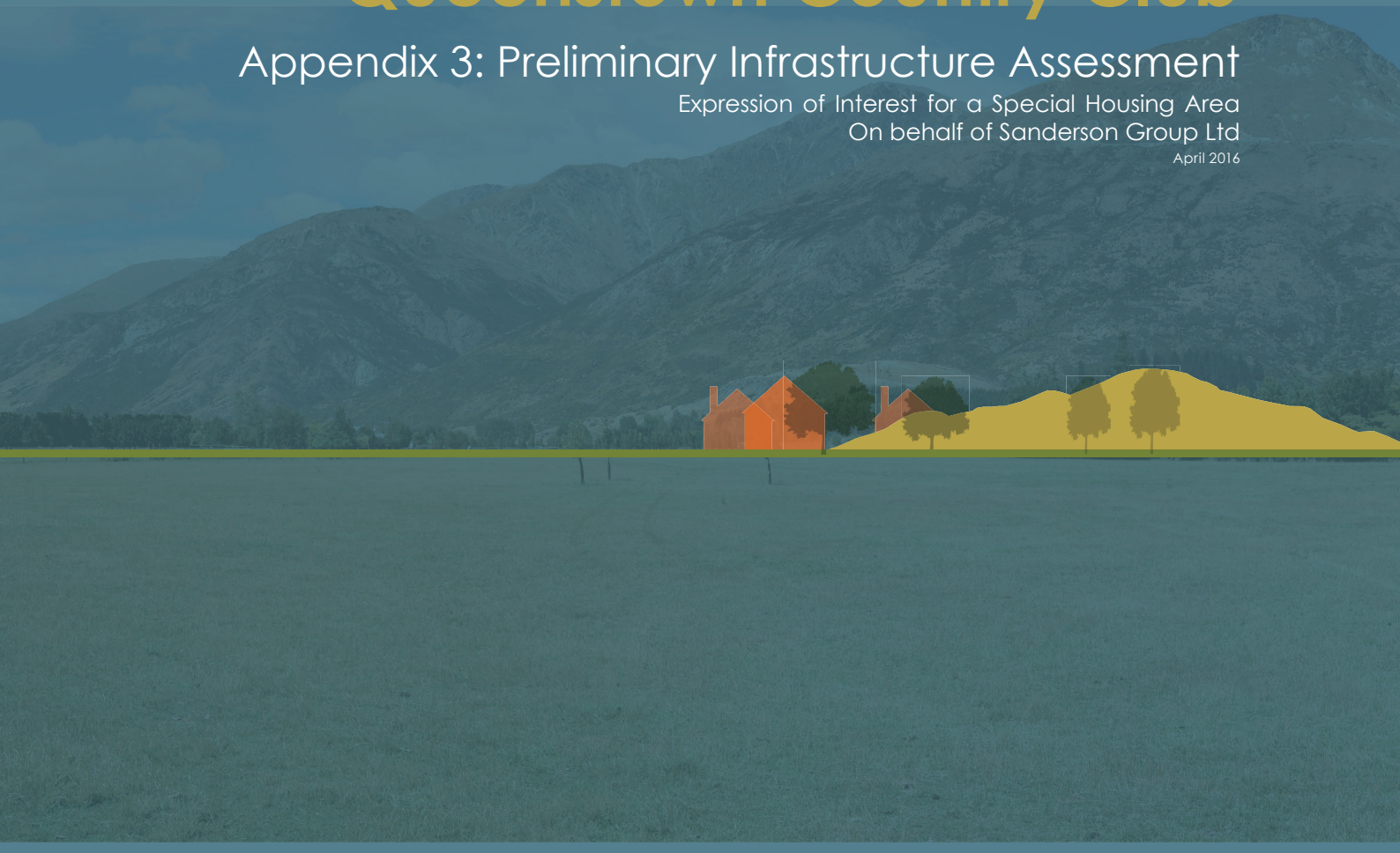
Queenstown Country Club

Appendix 3: Preliminary Infrastructure Assessment

Expression of Interest for a Special Housing Area
On behalf of Sanderson Group Ltd
April 2016



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Proposed Queenstown Country Club Retirement Village

Preliminary Infrastructure Assessment

April 2016

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Proposed Queenstown Country Club Retirement Village
Preliminary Infrastructure Assessment
April 2016

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Job No.: Q000245
Date: 8 April 2016
Reference: RP 16-04-08 ACS Q000245

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Proposed Queenstown Country Club Retirement Village
Preliminary Infrastructure Assessment
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1.0 Introduction

1.1 General

Fluent Infrastructure Solutions Limited (FS) has been engaged by the Sanderson Group to undertake a high level civil infrastructure overview of the proposed Queenstown Country Club (QCC) Retirement Village off Ladies Mile, on land lying between the residential areas of Shotover County (SOC) and Lake Hayes Estate (LHE). The brief for this engagement is to:

a. Identify potential civil infrastructure issues (capacity and demands), and

b. Confirm that the proposal is workable, identifying what further assessment or physical works / upgrades will be needed.

The report looks more particularly at the wastewater, water and stormwater infrastructure servicing for the development, with only brief commentary given to power and telecommunications servicing.

1.2 The Site

The site is located off the Frankton - Ladies Mile highway, about 5km northeast from Frankton. The site is to be developed as two sections - one lying between Ladies Mile and Howards Drive (Block 1), and the other to the south, towards the Kawarau River (Block 2). Figure 1.1 shows the location of the two sites, and their relation to the two neighbouring residential developments of Shotover Country and Lake Hayes Estate.

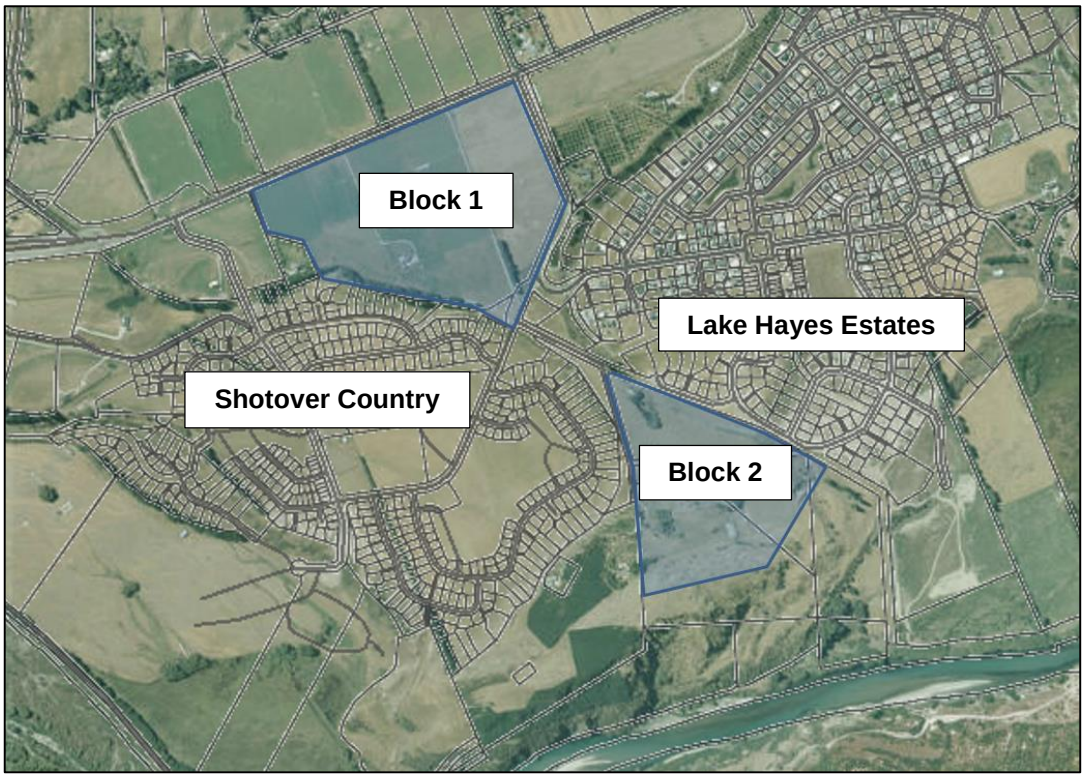

Figure 1.1: Site Location

The total area of the proposed development is approximately 0.5km^2. Block 1 is relatively flat, gently sloping towards Lake Hayes Estate. The northern part of Block 2 is also relatively flat, with the area to the south increasing in steepness up the slopes of Jones Hill that lies to the south by the Kawarau River.

The soils in the area are characterised predominantly as silty loams with slow to moderate permeability.

Blocks 1 and 2 are situated between the established subdivisions of Shotover Country and Lake Hayes Estates, each serviced with their own wastewater, water supply, and stormwater systems. This report examines the feasibility of connecting into these systems (based on their design capacities) and/or explores alternative options for the management of these services.

2.0 The Proposed Development Plan

Figure 2.1 shows the general layout of the proposed subdivision. The proposed development on which this preliminary infrastructure assessment has been undertaken comprises:

Block 1
153 Retirement Villas
72 Bed Rest Home
36 Apartments
7 Workers Accommodation Blocks
3 Commercial Facilities (e.g. Medical Centre, Pharmacist, Dentist)
Club House

Block 2
74 Retirement Villas
36 Apartments
10 Individual Allotments

The layout shown and the above scope of development is indicative and subject to change according to infrastructure and other requirements.

Figure 2.1: Proposed Development Plan

3.0 Wastewater

3.1 Background

Currently, both of the neighbouring Lake Hayes Estate and Shotover Country residential developments are serviced by the Queenstown Lakes District Council (QLDC) sewer system. Each development collects and then individually pumps its wastewater up to sewer lines on Ladies Mile, as described further below.

3.2 Wastewater Flows

The following design flows (Table 3.1) have been established for the proposed QCC development, the basis of which is described further below:

Table 3.1: Wastewater Design Flows

Unit Type	No. of Units	Max No. of People / Unit	WW - L/p/d	WW - m^3/d	Net WW PF	Peak WW - L/s
Block 1						
Villas	153	2	250	77	5	4.5
Rest Home - Residents	72	1	250	18	5	1
Rest Home - Staff	20	1	50	1	5	0.1
Apartments	36	1.5	250	13.5	5	0.8
Worker Blocks	7	8	250	14	5	0.8
Commercial - Medical	1	15	50	1	10	0.1
Commercial - Dentist	1	10	50	0.5	10	0.1
Commercial - Pharmacist	1	10	50	0.5	10	0.1
Club House	200	1	50	10	10	1.2
		Sub Totals		**136**		**8.5**
Block 2						
Villa	74	2	250	37	5	2.1
Apartments	36	1.5	250	13.5	5	0.8
Individual Lots	10	3	250	7.5	5	0.4
		Sub Totals		**58**		**3.3**
		Totals Blocks 1 and 2		**194**		**12**

As the development is not a conventional residential subdivision, it is not directly covered by QLDC's standard subdivision guidelines. The design criteria for the development has therefore generally been established from first principles, but with reference as appropriate to Queenstown Lakes District Council's 2015 Land Development and Subdivision Code of Practice. Average dry weather design flows are based on 250L/p/d, with a peaking factor of 2.5 for the dry weather diurnal and a dilution/infiltration factor of 2 for wet weather, as for residential properties.

The 250 l/p/d design flow has been applied to all residential properties except the worker blocks, which are to be housing units with workers out of the homes during the work day. A peaking factor of 5 has been allowed for all residential cases, but for commercial development, and non-resident staff, a higher peaking factor of 10 has been used to reflect the shorter daily period over which these flows occur.

The assessment identifies a combined (Blocks 1 and 2) peak wastewater design flowrate requirement of 12L/s.

3.3 Existing QLDC Infrastructure

The main sewer infrastructure servicing the Ladies Mile is a 300mm uPVC sewer rising main conveying wastewater along Ladies Mile direct to the Queenstown Wastewater Treatment Plant from Arrowtown and Lake Hayes. When the Lake Hayes Estate (LHE) development was designed in the early 2000's it was established that this main did not have sufficient capacity (along Ladies Mile) to accept wastewater flows from LHE. A second 150mm diameter sewer rising main was therefore laid along the road to join the 300mm main to where the road drops down to cross the Shotover River.

That 300mm rising main is a replacement for the original 375mm Arrowtown - Queenstown pipeline that had been suffering regular pipe failures and had reached the end of its serviceable life. A small section of this original pipeline is, however, retained and used at the western end of Ladies Mile, west of the new Stalker Road roundabout, as described below.

The arrangement of sewer pipelines along Ladies Mile is relatively complex and has been recently upgraded to better accommodate sewer flows from LHE, SOC and Bendemeer developments in association with the construction of the Stalker Road roundabout. Figure 3.1 present an overview of the main sewer infrastructure servicing LHE and SOC and Figure 3.2 presents a schematic diagram of the pipework towards the western end of Ladies Mile where it is proposed that the QCC development would likely connect - see Section 3.3 following:

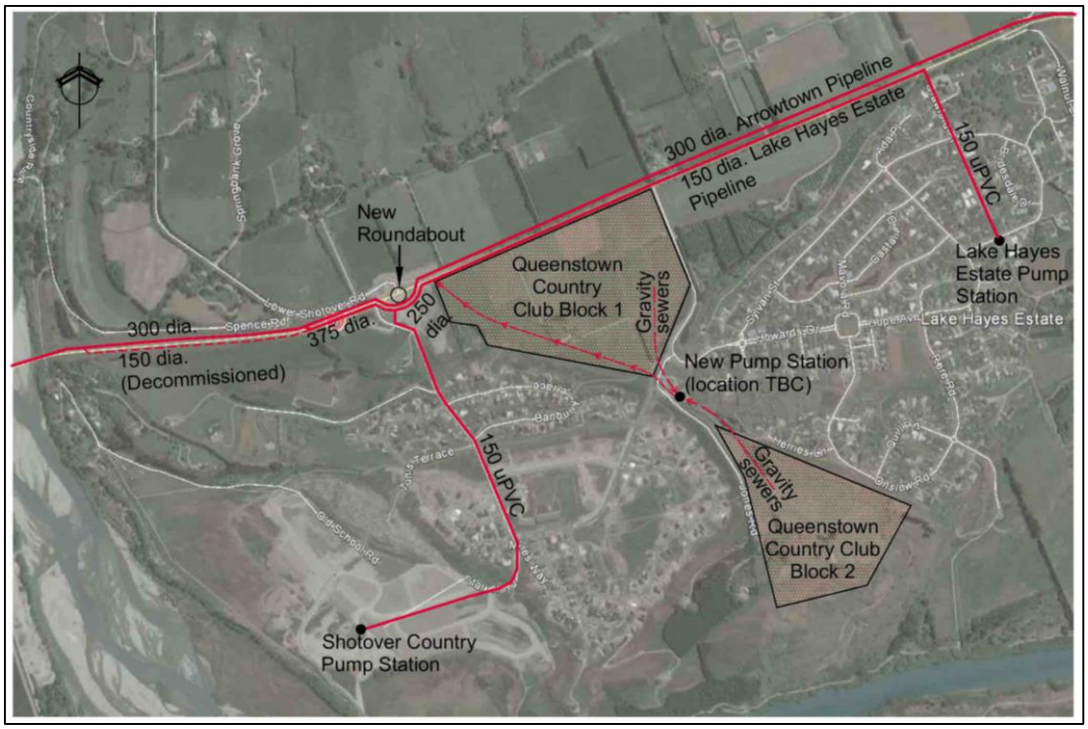

Figure 3.1: Schematic Overview of Existing Sewer Services Serving Shotover Country and Lake Hayes Estate

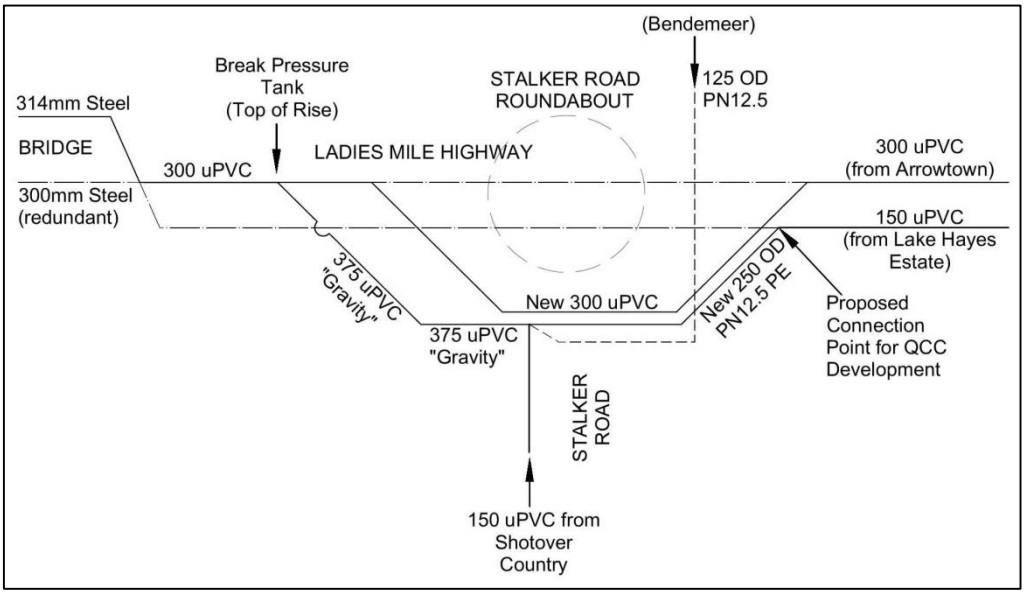

Figure 3.2: Schematic Diagram of Sewer Pipework at the Western End of Ladies Mile

The LHE rising main along Ladies Mile has been recently reconfigured to enter a break tank, near the Stalker Road roundabout, where it joins flows from the Bendemeer and Shotover Country sewer pumpstations. From this point wastewater enters a sealed falling main that runs down and across the Shotover River Bridge to the Shotover wastewater treatment plant. The original 150mm LHE rising main in the vicinity of the new Stalker Road roundabout has been recently increased in size to 250mm to help address capacity issues at the LHE pumpstation.

3.4 Wastewater Servicing for the Proposed Development

A report (letter) by Rationale addressed to the Queenstown Lakes District Council and dated 16 March 2015 considers the capacity of the LHE sewer system and that of the receiving sewer system in Ladies Mile to accommodate the new Bridesdale Development located to the south east of LHE. That report provides key information relevant to the proposed QCC development, namely:

a. The main LHE sewer pumpstation has recently been upgraded to a capacity of 25L/sec to address previous capacity issues at that location.

b. The recently reconfigured sewer network at the Stalker Road roundabout and the pipeline leading down the hill and across the Shotover River bridge is assessed as having a capacity in excess of 150L/sec. This exceeds the current operating flows from the communities currently discharging into that system.

From the investigations undertaken and enquiries made, it is clear that the SOC and LHE sewer systems have no capacity to accept sewer flows from the proposed QCC development. The QCC development will be required to reticulate its sewer networks to a new sewer pumpstation that will then deliver the wastewater from the development to the upgraded sewer system at the Stalker Road roundabout.

Whether the connection point from QCC to the existing sewer system is the new 250mm sewer east of the roundabout (as shown on Figure 3.2), or the break pressure tank further west, will be subject to a more detailed analysis of the existing sewer capacities and the impact on upstream rising mains and pumping stations.

The downstream reticulation leading to and across the Shotover River bridge, as noted above, has the capacity to receive flows using either option.

4.0 Water Supply

4.1 Water Supply System Design

The design, sizing and layout of the water supply network to service the proposed QCC development depends on the population served, the facilities to be provided and the water required to maintain the site landscaping. The following aspects relating to the water supply have been investigated to assess water supply requirements:

- Population (i.e. the number of dwellings and population figures)
- Water demands - both peak and fire fighting requirements
- Water supply availability
- Water pressure requirements
- Water storage requirements
- Landscaping irrigation requirements
- Water quality requirements

4.2 Water Demand Assessment

4.2.1 Retirement Village Residential and Commercial Demands

The proposed QCC retirement village is different to a conventional residential subdivision in regard to both domestic / commercial water demands and irrigation requirements. For a residential subdivision the property occupancy varies from house to house and can vary seasonally. Water for irrigation use is in the hands of individual households and is largely uncontrolled. For this reason Queenstown Lakes District Council in its 2015 Land Development and Subdivision code of Practice for the purpose of design sets a design house occupancy of 3 and sets criteria to cover irrigation requirements on a per capita basis (at 700L/p/d).

For the QCC retirement village proposal, dwelling occupancy numbers are known with reasonable accuracy, as are other occupancy numbers relating to staffing and hospital beds. Domestic / commercial water demand for this development is therefore assessed on a more direct first principles approach.

In regard to irrigation requirements, the landscaping and maintenance of the retirement village grounds will be the responsibility of the Body Corporate and not individual home (villa) owners. Accordingly, irrigation requirements need to be assessed separately to the domestic demands.

For the domestic water demand a per capita flow of 250L/p/d has been adopted (same as the wastewater demands). Other water demands have been assessed in regard to more specific activities within the retirement village.

Table 4.1 sets out the assessed domestic / commercial demands for the proposed development, and the assessed irrigation requirements. For this preliminary study, the same peaking factors used for the wastewater assessment have been used for the peak water demand assessment. These factors are likely to be high and are subject to further review at the more detailed assessment stage. They are however considered appropriate for this preliminary analysis.

Table 4.2 sets out the assessed irrigation requirement. For the purposes of this preliminary assessment a weekly averaged irrigation application rate of 3mm/d on lawns and landscaped areas has been adopted. This is a modest allowance and requires application away from the hot parts of the day when evaporation losses are high. Irrigation, then, will be on a managed basis over a 12 to 16 hour period per day, generally overnight, and more particularly avoiding peak domestic water demand periods during the day. This means that the daily irrigation demand will be relatively constant and not subject to the peaking characteristics typical of the domestic demands.

Two peaking factor scenarios have been considered:

- Case 1 – maximum domestic PF, with no irrigation
- Case 2 – 50% of maximum PF plus irrigation over 12 hours

Table 4.1: Assessed Water Supply Design Volumes and Flows

Unit Type	No. of Units	Max No. of People / Unit	WS - L/p/d	WS - m^3/d	Case 1 Net WS PF (Peak Domestic)	Case 2 Net WS PF (50% Peak)	Case 1 Peak WS - L/s (Peak Domestic)	Case 2 Peak WS - L/s (50% Peak)
Block 1								
Villas	153	2	250	77	5	2.5	4.5	2.2
Rest Home - Residents	72	1	250	18	5	2.5	1	0.5
Rest Home - Staff	20	1	50	1	5	2.5	0.1	0
Apartments	36	1.5	250	13.5	5	2.5	0.8	0.4
Worker Blocks	7	8	250	14	5	2.5	0.8	0.4
Commercial - Medical	1	15	50	1	10	5	0.1	0.1
Commercial - Dentist	1	10	50	0.5	10	5	0.1	0
Commercial - Pharmacist	1	10	50	0.5	10	5	0.1	0
Club House	200	1	50	10	10	5	1.2	0.6
Sub Totals (Domestic)				**136**			**8.7**	**4.2**
Block 2								
Villas	74	2	250	37	5	2.5	2.1	1.1
Apartments	36	1.5	250	13.5	5	2.5	0.8	0.4
Individual Lots	10	3	250	7.5	5	2.5	0.4	0.2
Sub Totals (Domestic)				**58**			**3.3**	**1.7**
Total Blocks 1 & 2 (Domestic)				**194**			**12**	**6**
Add Irrigation Daily Volume				**615**			**0**	**14**
Total Assessed Domestic + Irrigation Demand				**809**			**12**	**20**

*Irrigation figures developed from Table 4.2 below

Table 4.2: Irrigation Assessment

Site	Full Area (ha) of Developed Area	Irrigation %	Irrigated Area (ha)	Daily Volume @ 3mm/d (m^3/d)
Orchard	5.9	100	5.9	177
Block 1	17.7	40	7.1	212
Block 2	13.7	55	7.5	226
TOTAL				**615**

From Tables 4.1 and 4.2, the following water demand requirements (excluding fire fighting) have been established:

Peak Day Demand	810m^3/d
Peak Day Peak Period Demand - Domestic Peak - No Irrigation	12L/s
Peak Day Peak Period Demand - Reduced Domestic Peak + Irrigation	20L/s

4.2.2 Fire Fighting Demands

The design of the water supply system is also required to meet the fire fighting flow and pressure requirements of SNZ PAS 4509 - NZ Fire Service Firefighting Water Supplies Code of Practice.

All facilities, apart from the individual villas, will be serviced by a sprinkler system. This means that all properties fall under the FW2 water supply classification (Table 2), requiring a minimum fire fighting supply (Table 2) of a total of 25L/s from two hydrants, at a minimum pressure of 100kPa.

4.3 Existing Water Supply System

Originally the LHE and SOC water supplies were separate individual systems and quite independent of the main Queenstown water supply system. The water supply to these two areas has recently been modified such that both LHE and SOC are now served by a common system supplied by a single water bore located near the edge of the Shotover River. This bore supply feeds a reservoir located on nearby Jones Hill, which in turn feeds back down to the LHE and SOC residential developments.

The reservoir sits at approximately RL382m, which compares to ground levels over the QCC land generally in the RL360 to 370m range. This means that the reservoir is not sufficiently high to provide the required water pressures to the proposed QCC development without booster pumping.

Planning is underway, however, to upgrade the existing LHE and SOC water supply with additional bores and additional pumping capacity for an “on-demand” type system. The preliminary design for this is currently being undertaken by Fluent Solutions, raising the

possibility of modifying the intended design to accommodate the QCC development - if approved. This is discussed further in Section 4.4 following.

In parallel with the planned extension to the existing LHE / SOC borefield system, the intention is to also develop a second set of bores in the same area that will supplement the Queenstown Water Supply System that would back feed into the Frankton and wider Queenstown areas over the Shotover River Bridge. The schematic arrangement of these two supplies in shown on Figure 4.1

With regard to the capacity of the Queenstown and LHE / SOC water supplies, reference is made to three Tonkin and Taylor reports / letters:

i. "Shotover Borefield Design Flows and Volumes", dated 14 October 2015.

ii. "Water Modelling Results for the Proposed Bridesdale Development, and Connection of Lake Hayes Estate to Shotover Country Pressure Zone, V2", dated 05 March 2015.

iii. "Queenstown Water Supply Future Options - High Level Concept for Shotover Bore Supply", dated September 2015.

The following key information is summarised from these reports in Table 4.3 together with the estimated QCC flows.

Table 4.3: Water Demand Summary for Queenstown, LHE / SOC, and the Proposed QCC Development

Period	Parameter	Queenstown		Lake Hayes Estate Shotover Country (2)	Demand Estimate for QCC
		Total (1)	New Shotover Bore Contribution		
Current Flows	Peak Day Flow (m^3/d)	24,146	-	345	-
	Peak Hour Flow (L/s)	280	-	12	-
30-year Design Flows	Peak Day Flow (m^3/d)	49,855	18,320	10,200	810
	Peak Hour Flow (L/sec)	577	240	155	20

(1) Table 5, T & T Report (iii)

(2) Table 3, T & T Letter (i)

In considering the supply of water to the proposed QCC development (Section 4.4) an important consideration is the potential use of soon to be redundant water treatment plant infrastructure installed originally to service the LHE subdivision. This infrastructure includes a booster pumping facility as well as a $210m^3$ reservoir and is ideally located midway between the two proposed Blocks 1 and 2 (see Figure 4.1). The possible use of this infrastructure is discussed below, noting however that its availability for such re-use has not yet been canvassed with the Queenstown Lakes District Council.

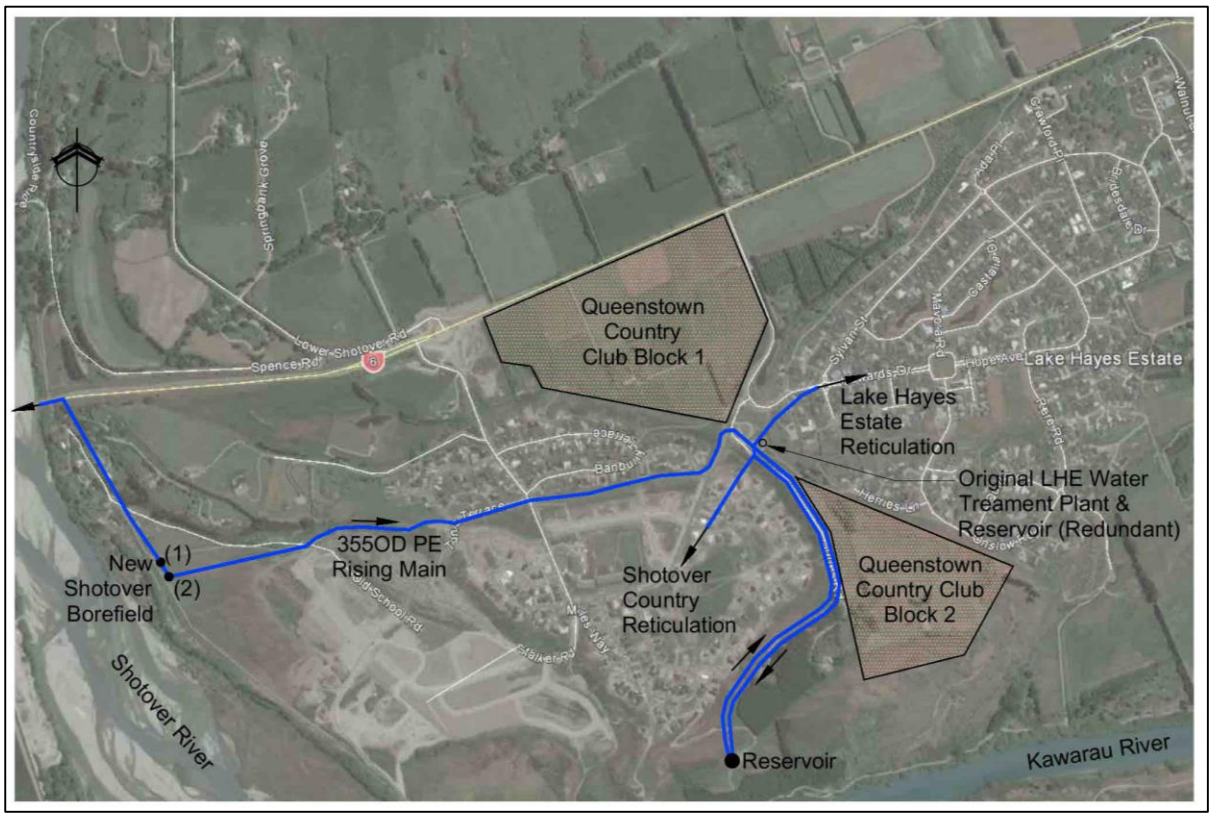

Figure 4.1: Conceptual Outline of Proposed Water Supply Upgrade to Queenstown and LHE / SOC (Subject to Confirmation)

Water drawn from the Shotover borefield is of sufficient quality to be treated by UV disinfection in order to meet NZ drinking water quality requirements. The supply is currently treated this way at the original LHE treatment facility, but under the future arrangement the UV treatment system is to be considered for relocation to the borefield site.

4.4 Water Servicing Requirements for the Proposed Development

The water demand for the proposed QCC development (from Table 4.3) equates to approximately 8% of the protected future daily demand for the LHE / SOC area, and approximately 1.5% of the total Queenstown area demand. Given that the proposed new Shotover borefield development has significant capacity (subject to Resource Consent) to supply water demands beyond that currently envisaged, it is concluded that subject to appropriate planning and design the daily water requirements for the proposed QCC retirement village can be easily met.

The conveyance of the additional water supply to the QCC development is also reasonably achievable, and is largely a case of closer technical scrutiny at the time of more detailed design. Particular considerations to be taken into account in future planning are summarised as follows:

A) Peak Hourly Demand

The 20L/s assessed peak day peak hourly water demand can be provided in a number of ways, including:

(i) By a commensurate increase in the pumping rate from the borefield (155L/sec to 176L/sec); this simply requires larger capacity pumps or more pumps to be installed in the borefield.

(ii) By a reduced increase in the pumping rate from the borefield together with the provision of reservoir storage used to help satisfy peak hourly demand. Given the existence of a 210m^3 reservoir that is potentially available to QCC, this option appears more favourable than (i), but needs further study.

(iii) A third possible option is to take the QCC supply off the higher pressure feed from the new Shotover borefield to the Queenstown system. This would, however require running a water main back from the Shotover River bridge to the QCC property beyond the Stalker Road roundabout. The cost effectiveness of this would need to be compared to that of booster pumping and reservoir storage.

B) Maintaining Supply Pressure

The minimum supply pressure for a reticulated water network is 300kPa or approximately 30m as stipulated in QLDC's design standards. At present the existing reservoir is not able to supply this pressure to the QCC land. Options to provide the required service pressure include:

(i) Increase the delivery head of the bore pump (i.e. bigger bore pumps), though this is not favoured because about 90% of the water supply is boosted to unnecessarily high pressures, leading to related energy wastage issues.

(ii) Install a booster pumpstation to boost pressure in the QCC development only; this is the more favoured approach, particularly given the potential re-use of some existing redundant infrastructure.

(iii) The alternative higher pressure supply option off the planned new pipeline back to Frankton is a further option to be considered.

C) Fire Fighting Supply

A fire fighting supply of 25L/sec at a pressure of 100kPa is required. Some areas of the QCC development may achieve this from the existing reservoir, but not all. Booster pumping will be necessary, noting that a potential fire booster pump already exists in the soon to be redundant LHE water treatment complex.

From the foregoing assessment it is concluded that a suitable water supply can be provided to meet the demands of the proposed QCC development, but further more detailed analysis is required to establish the best way this can be achieved. Discussions would be required with the Queenstown Lakes District Council concerning the potential availability of the old LHE water treatment plant infrastructure. Additionally, water supply modelling by Council's water modelling consultants Tonkin & Taylor will likely need to be carried out once the more detailed design phase is reached.

5.0 Stormwater

5.1 Introduction

This section of the report describes the stormwater runoff conditions that exist on the QCC Retirement Village development site and the stormwater management concepts proposed for the development.

5.2 Stormwater Catchment Characteristics

The topography of the land form between the left bank of the Shotover River and Lake Hayes means that most of Block 1 and all of Block 2 drain to the Kawarau River that flows parallel to and south of the Frankton - Ladies Mile Highway. The small corner area of Block 1 that does not drain to the Kawarau River is identified as the "Special Stormwater Management Area" in Figure 5.1. The Special Stormwater Management Area drains via the new Shotover Country subdivision area toward the confluence of the Shotover and Kawarau rivers.

Prior to construction of the LHE subdivision, the natural flow paths from most of Block 1 and the greater part of Block 2 flowed down gullies onto and then over a series gently sloping grazed river terraces in shallow watercourses down to the Kawarau River. The overland flow across the grazed terraces below Blocks 1 and 2 would have moderated the flow rates from the upstream gullies discharging from Blocks 1 and 2 before the residual flows reached the Kawarau River.

Block 1 slopes from the Frankton - Ladies Mile Highway toward LHE and also east towards Lake Hayes. The Frankton - Ladies Mile Highway has normal road water tables and is also effectively a catchment boundary for flows arriving from the north and therefore the Block 1 site is not affected by stormwater from up-gradient of the site.

The part of Block 2 that does not drain to LHE drains to the south more directly to the Kawarau River via an existing drain.

As noted previously, the soils in the area are characterised as predominantly silty loams with slow to moderate permeability. The soil type and land use type areas determine the runoff rates and volumes for the pre-development and post development runoff flow scenarios.

5.3 Existing Stormwater Systems

There is no existing stormwater infrastructure on either Blocks 1 or 2.

5.3.1 LHE Stormwater Network

The LHE piped stormwater collection system was implemented in approximately 2010. In 2010, under the QLDC' 2015 Land Development and Subdivision Code of Practice, the LHE stormwater system is expected to have been designed to convey a 5year Average Return Interval (ARI) flow through the LHE residential area. The LHE piped primary stormwater system discharges to an existing open drain that conveys stormwater from LHE and part of Block 2 to the Kawarau River. Since the LHE stormwater system was constructed the QLDC

has issued a revised "Land Development and Subdivision Code of Practice" referred to further here as the "2015 Subdivision Code".

An inspection of QLDC services plans available on-line suggests that there is no stormwater detention facilities in the LHE stormwater system. The LHE primary flow path pipelines have therefore been constructed to provide stormwater drainage for the post development catchment condition without ponding for a 5 year ARI rainfall event. Where flows occur in excess of the capacity of the piped system, the excess flow is conveyed via secondary flow paths in LHE that are typically the residential streets.

5.3.2 Existing Connection Option

There is a 300mm diameter connection to the LHE stormwater network located on the access road from LHE to the Shotover Country subdivision that may have been provided to take runoff from Block 1. There is no apparent connection point for Block 2.

From QLDC's GIS stormwater reticulation information the 300mm diameter connection flows into a section of the LHE stormwater network that has pipe sizes up to 450mm diameter but is then connected to the trunk main network via a 225mm diameter pipe. Unless the infrastructure data is incorrect, the 225mm diameter link would severely restrict any discharge from Block 1.

There is also a 200mm diameter connection from the existing water treatment plant to the LHE stormwater system. It is presumed that this pipe is an emergency water supply discharge facility and is therefore is not available for routine stormwater discharges.

Although it cannot yet be confirmed, the LHE stormwater system would appear to have been designed for flows in excess of the 5 year ARI 60 minute storm rate of runoff from undeveloped land. In the absence of stormwater detention facilities, the LHE stormwater system has likely been designed for the developed LHE catchment condition. Even if the LHE stormwater system is designed for the developed catchment condition, the capacity of the 300mm diameter connection to LHE is substantially less than the 20 year ARI predevelopment flow for Block 1.

It is concluded that the LHE stormwater network has no capacity to accept runoff from the proposed QCC development.

5.4 2015 Code of Subdivision Requirements

5.4.1 Key Clauses

The 2015 Subdivision Code (Cl 4.3.5) requires that a primary stormwater system be designed to convey a minimum 20 year ARI flow taking into account climate change. Where a secondary flow path is available, the secondary flow path is required to convey the balance of a 100 year ARI flow without damage to property and with freeboard. If a secondary flow path is not available, the primary system is required to convey a 100 year ARI flow with freeboard.

Further, where discharge to an existing reticulated QLDC network (such as the LHE network) it is proposed that the flow rate shall be no greater than what would have occurred for the undeveloped catchment for a 60 minute 5 year ARI storm.
Other 2015 Code of Subdivision requirements include:

- Clause 4.3.7.1 expresses QLDC's preference for low impact design (LID) stormwater control solutions but qualifies the preference by requiring the approval of maintenance requirements before submitting LID proposals for acceptance.
- Clause 4.3.7.4 states that "Detention ponds shall only be used with prior approval from Council."

5.4.2 Implications of the 2015 Code

The implications of the 2015 Code of Subdivision for QCC Blocks 1 and 2 are that the QCC stormwater reticulation is required to convey the flow generated by a 20 year ARI design storm without causing surface flooding. The 20 year ARI flow requirement is a greater than the capacity of the LHE stormwater system. The Code of Subdivision would also require that the discharge from a QCC system to the existing LHE system must be limited to the rate of runoff for a 5 year ARI 60 minute rainfall event on the undeveloped (pre-development) catchment.

5.5 Otago Regional Council Requirements

The stormwater discharge from Blocks 1 and 2 would be to the Kawarau River downstream of the confluence of the Shotover and Kawarau rivers. The Shotover River following rainfall events, that trigger runoff in its catchment, carries significant volumes of sediment. The dry weather flow rate in the Kawarau River at the site is typically in excess of 150 cubic metres per second (m^3/s) and therefore a stormwater discharge from QCC development of the order of $1m^3/s$ from the site would be rapidly diluted.

Stormwater runoff from Blocks 1 and 2 would be discharged to the Kawarau River via what is referred to as a "drain" in this report that also conveys water from the LHE stormwater system to the Kawarau River. If the drain is considered to be a "river" then work on the drain to accommodate stormwater discharges from the Blocks 1 and 2 involving disturbing the bed of the drain would require consent from the Otago Regional Council because the drain catchment is greater than 50 hectares.

Since a reticulated stormwater system would discharge to the Kawarau River then there is a requirement to comply with the permitted activity rules for stormwater discharges or potentially a requirement to obtain consent for a discretionary activity.

To comply with stormwater discharge requirements in the ORC Regional Plan-Water there must be no visible evidence of contamination after reasonable mixing. If the drain were considered to be the receiving waters and the drain has ecological values then a stormwater discharge is very likely to require consent for discretionary activity. If the receiving waters for the stormwater discharge is the Kawarau River then it is expected that the stormwater discharge would be a permitted activity.

5.6 Proposed Stormwater Management Concept

5.6.1 Stormwater Management Options

The primary drivers for stormwater management works are:

i. Protection of property against flood damage - within the proposed area and downstream of the development.

ii. Environmental effects. The adverse effects to be avoided or mitigated are typically the management of contamination and the prevention of erosion and sedimentation.

iii. Practicability in terms of construction and sustainable operations and maintenance practice. Practicability and sustainable operation defines the lifetime cost of the system.

Guidance for responding to these drivers is provided in the QLDC code and ORC Water Plan requirements. The stormwater management options considered in arriving at the stormwater management concepts described below were as follows:

1. Stormwater Infiltration
   The upper soil horizon soils are described silty loams. Underlying the loams there is likely to be silty sandy gravels. Based on general knowledge of the Frankton Flats geology 3 kilometres to the west, infiltration rates are relatively slow and therefore infiltration options would require considerable subsurface storage for a 20 year ARI storm event and hence the cost would be at the high end.

2. Direct Runoff
   The direct runoff option would require designing the open channel or pipe reticulation system within each of the two village blocks and the trunk mains to the Kawarau River to convey a 100 year ARI storm event. This direct runoff option would have major construction costs.

3. Open Channel Mains
   The proposed location of the discharge main from Block 1 to the drain at the Kawarau River would be close to the top edge of a high terrace and given the fine non cohesive nature of the soils there would be a significant risk of failure for an open earth channel and therefore is unlikely to be a practical option.

4. Discharge to LHE
   As discussed above the LHE discharge option would require further investigation with regard to capacity before it would be considered a practical option. This option is not likely to be feasible.

5. Stormwater Detention
   Stormwater detention at a central location within each residential sub-catchment for a 20 year ARI event with surface ponding within each block for up to a 100 year ARI event is proposed. If the proposal goes to a detailed feasibility stage then other detention options such as LID retention/rain-gardens in the street scape and in tanks within the residential building sites should be considered to reduce stormwater collector costs and maximise ground area utility. Subsurface storage at central locations should also be considered.

There was no site specific geotechnical information considered in the course of preparing this report. A detailed geotechnical site assessment may lead to some of the options discussed above being re-assessed.

5.6.2 Proposed Concept

Since the disposal of stormwater from Blocks 1 and 2 via the LHE stormwater collection and disposal system places limitations on the rate of design discharge and poses a potential risk of flood damage to residential property in LHE, a separate stormwater collection and disposal route is proposed. The proposed stormwater management concept, illustrated in Figure 5.1, includes the following elements:

1. Piped primary stormwater collection system receiving runoff from roofs, pedestrian road pavements and vegetated areas (lawns and gardens). The collection system would be designed to convey 20 year ARI rainfall events without causing surface flooding.
2. Secondary overland flow paths would convey flows in excess of the primary piped stormwater system capacity to the detention area locations.
3. Detention storage would be provided to moderate the increased rate of runoff resulting from the relatively high density of development and associated impervious areas back to the predevelopment runoff rate for the 20 year ARI and 100 year ARI rainfall events.
4. To accommodate the discharge from a 100 year ARI rainfall event, the detention storages for the 20 year ARI flows would be surcharged and local flooding would occur. The discharge in the trunk mains to the stormwater outfall would increase slightly and the additional volume of water would pond in designated areas without resulting in significant overflows to the LHE and Shotover Country urban areas.
5. The collection system, and, or the detention storage areas would be designed to remove floatable debris, oil and grease and coarse sediments before discharge to the Kawarau River.
6. Primary discharge stormwater mains would convey stormwater from detention areas in the Block 1 and Block 2 areas to an outfall main pipeline that would discharge to an existing drainage channel that conveys stormwater runoff from the LHE stormwater system to the Kawarau River.
7. Upgrade the existing drainage channel to the Kawarau River, if required, to take the additional direct runoff flow from Blocks 1 and 2.

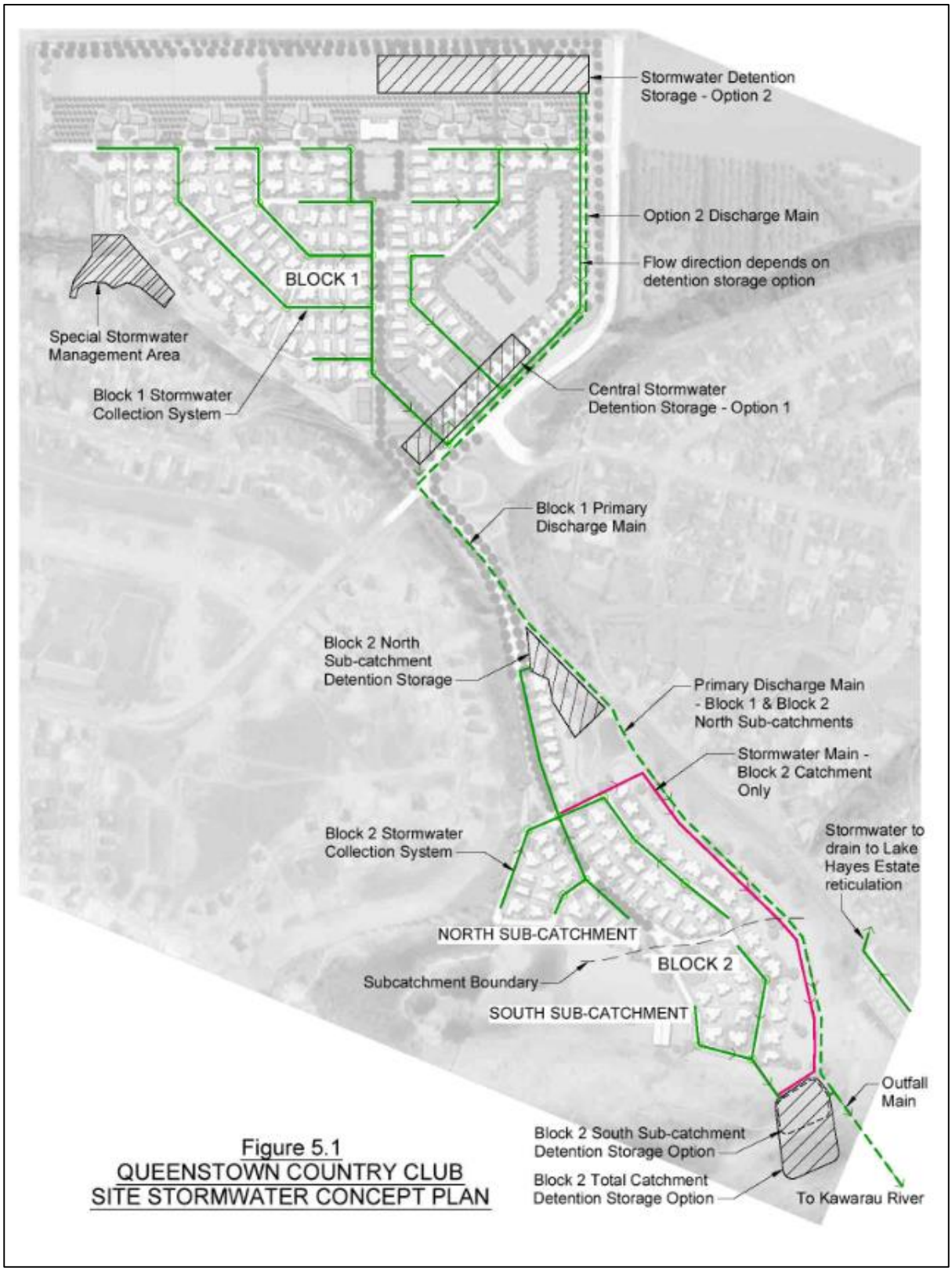

Figure 5.1: Queenstown Country Club Site Stormwater Concept Plan

5.7 Preliminary Stormwater Element Design

Preliminary hydrological calculations have been completed to identify the expected sizes of the stormwater management elements.

5.7.1 Preliminary Hydrological Calculations

HEC HMS hydrological modelling software was used to quantify the stormwater flows leaving the site in both the pre and post-development scenarios. This program utilises rainfall input data to generate flow outputs from the site, based on specific site characteristics.

Rainfall information for the site was derived from rainfall depth-duration-frequency data taken from the HIRDS V3 database, and a rainfall hyetograph (rainfall intensity versus time graph) was developed for the 1 in 20 year, 24 hour design storm. A 2ºC climate change allowance was included in the design rainfall distributions.

Table 5.1: Stormwater Flow Summary

Catchment	Average Return Interval (Years)	Predevelopment Flow (and Maximum Detention Area Discharge Target) (m^3/s)	Post Development Direct Runoff Flow (m^3/s)
Block 1	**20**	**0.40**	**1.2**
	100	**0.8**	**1.8**
Block 2	**20**	**0.17**	**0.9**
	100	**0.36**	**1.3**

5.7.2 Stormwater Element Capacities

The indicative sizes, and options for stormwater management in some cases, are provided in Table 5.2.

Table 5.2: Indicative Sizes of Stormwater Elements

Item	Element Description	Location	Indicative Size	Options / Comments
1	Stormwater Collection Pipe Network	Block 1	225mm diameter up to a maximum of 1000mm diameter pipes	If detention in the form of rain gardens, local detention tanks were considered an advantage then the maximum pipe size could be reduced.
		Block 2	225mm diameter up to a maximum of 600mm diameter	As for Block 1 above.
2	Detention Areas	Block 1	4000 cubic metres	a. The detention volume could be reduced if local detention storages in a variety of forms were provided. b. The detention storage area would be partitioned to treat the "first flush" to manage the collection of debris and other contaminants. The balance of the detention storage would then be inundated infrequently and could then be used more effectively for recreation. c. Two detention storage location options are shown in Figure 5.1. The construction cost for Location Option 1 is expected to be less than that for Location Option 2.
		Block 2	3500 cubic metres	Comments a. and b. as for Block1 apply for Block 2 plus: The Block 2 north sub-catchment detention storage option enables a single common Primary Discharge Main to be used for the Block 1 discharge and the North sub-catchment area of the Block 2. If all of the Block 2 stormwater runoff is conveyed to a single detention storage area at the southern end of Block 2 then a stormwater collector main and the Discharge Main for Block 1 would be required - this would impose a higher construction cost.

Item	Element Description	Location	Indicative Size	Options / Comments
3	Primary Discharge Mains	Block 1	600mm diameter for Block 1	
		Block 2	700mm diameter pipe for the combined Block 1 and northern sub-catchment of Block 2	Relies on adoption of the Block 2 North Sub-catchment Detention Area option.
4	Outfall Main	Combined Block 1 and Block 2 Detention Area discharges	600mm diameter	The diameter is less than the Discharge Main diameter required for Blocks 1 and 2 because of the steeper slope of the Outfall main alignment.
5	Drain Section to the Kawarau River	Outfall Main to the Kawarau River	Open channel approximately 2 metre bottom width and 1.5metres deep - likely to be wider and deeper than the existing outfall drain.	Assuming that the existing drain has been in existence since before the LHE stormwater system was constructed, then previously the runoff discharges from Blocks 1 and 2 would have been moderated by the undeveloped river terraces. The more direct runoff from the developed Blocks 1 and 2 may result in the need for some upgrading of the drain. The cost is expected to be modest.

6.0 Power and Telecommunications Infrastructure

Formal responses have not been received from Delta, Powernet and Chorus regarding power and telecommunications capacity in the LHE / SOC area, but given the significant development already planned in the area over the coming years it is likely that the planned QCC development can be readily accommodated along with this other growth.

7.0 Summary

A high level civil infrastructure overview of the proposed Queenstown Country Club Retirement Village development has found that all infrastructure requirements for the development can be met by existing and new services.

Wastewater servicing will be met by an internal gravity sewer collection network within each Block that will run to a wastewater pumpstation delivering to a connection point to existing sewer reticulation in the vicinity of the new Stalker Road roundabout on Ladies Mile.

Water demand can be met by supply from the proposed upgraded Shotover borefield development to the west of Shotover Country. Particular measures, such as booster pumping, will be necessary to provide sufficient service and firefighting pressures to the elevated QCC sites. Peak hourly demands will likely be met by a combination of direct injection to the reticulation and reservoir storage.

The disposal of stormwater from Blocks 1and 2 via the LHE stormwater collection and disposal system places limitations on the rate of design discharge and poses a potential risk of flood damage to residential property in LHE. Therefore a separate stormwater collection and disposal route discharging to an existing drainage channel that conveys stormwater runoff from the LHE stormwater system to the Kawarau River is proposed. On-site stormwater detention will be required to meet local stormwater standards and reduce peak discharge off the site. Such detention can be achieved in various ways, but this has yet to be established.

Regarding power and telecommunications servicing, given the significant development already planned in the area over the coming years it is likely that the planned QCC development can be readily accommodated along with this other growth.