**BEFORE THE HEARINGS PANEL**
**FOR THE PROPOSED QUEENSTOWN LAKES DISTRICT PLAN**

| | |
|---|---|
| **IN THE MATTER** | of the Resource Management Act 1991 |
| **AND** | |
| **IN THE MATTER** | of the Queenstown Lakes Proposed District Plan |
| **AND** | |
| **IN THE MATTER** | of Hearing Submissions Seeking Amendments to the Planning Maps covering Queenstown and Queenstown Rural (Excluding Wakatipu Basin) |

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**STATEMENT OF EVIDENCE OF CHRISTOPHER CHARLES HANSEN ON BEHALF OF**

**Jardine Family Trust**
**Remarkables Station Ltd**
**Homestead Bay Trustees Ltd**

**(Submitter 715)**

**Dated 4th June 2017**

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## 1.0 QUALIFICATIONS AND EXPERIENCE

1.1 My name is Christopher Charles Hansen. I hold a Degree of Bachelor of Surveying from Otago University. I am qualified as Licensed Surveyor and a Member of the New Zealand Institute of Surveyors and the Consulting Surveyors of New Zealand.

1.2 I have eighteen years experience as a Surveyor and Land Development Engineer. I have held positions as a Surveyor and Site Engineer in private practice within Queenstown, Whistler British Columbia, Canada & London, England. I am a partner of Clark Fortune McDonald & Associates Limited.

1.3 During this time, I have gained experience in Land Development Engineering in many residential and commercial developments. I have personally been involved with the design and construction of numerous land development projects.

1.4 I have read the Code of Conduct for Expert Witnesses in the Environment Court's Consolidated Practice Note (2014) and agree to comply with that code. This evidence is within my area of expertise, except where I state I am relying on what I have been told by another person. I have not omitted to consider material facts known to me that might alter or detract from opinions that I express.

## 2.0 EXECUTIVE SUMMARY

2.1 The proposed re-zoning of the Homestead Bay Residential Area is not considered to have any impacts on the infrastructure network. Infrastructure already exists that can be augmented as required to cater for additional demand or new infrastructure can be developed to service the residential activity proposed.

2.2 The infrastructure will be constructed and paid for the by the applicant as the development proceeds. It is anticipated that new infrastructure required would be constructed at little or no cost to QLDC. It is possible that the construction of new infrastructure required for this development could also have a wider network or community benefit by augmenting or providing additional security to existing infrastructure.

2.3 Stormwater would be managed for the development on site and is not expected to have any effects on existing infrastructure.

2.4 Other non-Council infrastructure and network utilities exist and have capacity to supply this development. Should additional capacity to accommodate the cumulative demand of the residential on the non-Council infrastructure be required, it can readily be provided.

## 3.0 SCOPE OF EVIDENCE

3.1 The purpose of this evidence is to assist the Hearings Panel within my expertise as a Surveyor and Land Development Engineering in relation to the submission lodged by Middleton Family Trust (#715) on the Queenstown Lakes Proposed District Plan.

3.2 I have been engaged to assess servicing options for a proposed rezoning on land located at Homestead Bay between Jacks Point and Lake Wakatipu. The scope of work includes examination of existing QLDC as-built records, confirmation of capacity of existing services to determine the adequacy of the existing infrastructure, and recommendation of infrastructure servicing options.

3.3 My assessment and recommendations are provided in detail within the report contained in Attachment A to my evidence. My report is preliminary and for the planning map hearings for the QLDC District Plan Review only. Further information and detailed engineering design will be required if development proceeds.

**4.0 SUBMISSION 715**

4.1 The proposal seeks to re-zone land from rural general to residential activities.

4.2 The site is legally described as Lots 6, 7 DP 504891 & Lot & 8 D.P.443832. The total site area comprises approx. 200 ha and is contained in CT's 760709, 760710 & 555575 respectively.

4.3 The site has frontage to Kingston Road (SH6) and from Maori Jack Road.

**5.0 WASTEWATER**

5.1 The Design flows are outlined in Attachment A to my evidence.

5.2 In terms of proposed servicing Lowe Environmental Impact prepared a report for wastewater options in May 2017 which is contained in Attachment B to my evidence. This report investigated 130 of the proposed 715 DE's proposed for this zone.

5.3 The report considered 4 different options each of which were considered feasible.

5.4 It is the our expectation that each of the options are still viable for the added demand by scaling up the system to cater for the added demands. It is likely that the additional dwellings will result in a better per lot cost for some of the options.

5.5 There is also considered enough suitable land available for on site disposal options within the development area for the fully developed scenario.

*Required upgrades*

5.6 If the option to connect to QLDC infrastructure was chosen, any effects on the QLDC's wider infrastructure being the Shotover Waste Water Treatment Plant will be mitigated by the imposition of headworks fees at the time of connection to Council's service.

5.7 Upgrades to the Shotover Waste Water Treatment Plant are currently under construction.

## 6.0 STORMWATER

6.1 The catchment, existing reticulation, hydrological analysis, runoff quality, management objectives and approaches are outlined in Attachment A to my evidence. In terms of storm water management options I confirm that many options are available to avoid, remedy or mitigate the adverse effects associated with residential development on receiving environments.

6.2 For the current project the recommended stormwater management strategy is to provide an integrated treatment train approach to water management, which is premised on providing control at the catchment wide level, the allotment level, and the extent feasible in conveyance followed by end of pipe controls. This combination of controls provides a satisfactory means of meeting the criteria for water quality, volume of discharge, erosion and flood control (if required).

*Concept Design*

6.3 Runoff from undeveloped areas shall be directed around the developed areas via grass swales, and then discharged to ground. This will replicate the pre development runoff scenario for the undeveloped areas. The developed areas will be serviced using a hybrid LID/SUD/Big Pipe design. This will incorporate a combination of grass swales, kerbs, pipework and detention areas.

6.4 The development area can be broken into smaller sub-catchments: Separate pipe networks are then proposed - one for each catchment. Each network will discharge to the stream, gully or directly to the Shotover River. Secondary overflow paths will be provided for in swales or road ways.

Overflows will discharge to the same locations as the pre-development scenario.

## 7.0 WATER SUPPLY

7.1 The supply design, design flows, fire fighting demand and existing infrastructure are outlined in Attachment A to my evidence. In terms of the concept design currently under development on the site is a new 300mm water bore adjoining Lake Wakatipu. Test pumping and aquifer analysis of the new bore is set to commence from 15th June. Preliminary bores and testing indicate excellent quantity of water at secure depths.

7.2 On conclusion of the aquifer analysis design for a communal water supply can be completed to supply potable water to the residential demand.

7.3 It is anticipated that an "on-demand" system similar to that at Shotover Country can be developed.

7.4 We note that the existing 300mm Shotover Country bore services approx. 950 dwellings and this residential scenario is similar in scale and nature.

7.5 The new system would also include a water treatment plant that will treat the water at the source and be pumped to areas of development and to a high level water reservoir that will buffer peak flows and provide static fire fighting reserve.

7.6 From the reservoir internal reticulation within the development would be sized accordingly but is anticipated that mains of 200mmØ would be required if arranged in ring formations where possible.

7.7 It is proposed that a new reservoir could be established on the Jacks Point hill to the west of the development at a suitable elevation to service the development. The applicant is able to provide the land necessary for the establishment of a reservoir and is able to provide the access required.

7.8 The new tank elevation will be very similar to the existing Coneburn water reservoir. There may be opportunities to link the reservoirs to provide security of supply and redundancy in the network.

7.9 Sizing of the reservoir should also be carefully considered as this could help eliminate peaks in the demand. This would then allow for a lower peak flow of water to be taken from the new system.

7.10 All new infrastructure constructed for this development could be vested in Council ownership.

7.11 QLDC may choose to pump the water further afield to meet the demands of other developments in the District.

7.12 The further design and modelling of the infrastructure would need to be undertaken closely with the QLDC to confirm availability of supply. It is anticipated that QLDC water modelling consultants will be needed to carry out this modelling at the next phase of design.

*Required upgrades*

7.13 At this stage, it is not considered likely that connection will be made to existing QLDC infrastructure. Therefore no upgrades of QLDC assets are anticipated. Similarly if a new scheme is established for this development, all costs of establishing the new infrastructure would be borne by the applicant.

## 8.0 POWER, TELECOMMUNICATIONS AND GAS

8.1 Aurora Energy has high voltage 33kVa network running south on the eastern side of SH6 Kingston Road. There is an existing High voltage underground connection that feeds into the south end of Jacks Point and the existing activities in the Homestead Bay area.

8.2 We understand that Powernet are in the process of extending network as far as Hanley Farms approx. 2km away to the north.

8.3 We consider that either network could supply suitable underground electrical supply to the proposed development. Below is a screen shot from Aurora's GIS showing the existing electrical infrastructure.

8.4 Chorus fibre optic telecommunications cables exist in State Highway 6. It is anticipated that connection to the network can be made and that the new development would be serviced with fibre to the door.

8.5 All infrastructure is underground. All necessary mains will be extended to service the development area as development proceeds. Confirmation from the network owners will be obtained at each stage of development prior to proceeding.

8.6 It is not anticipated that there will be any supply or capacity issues for these services and connection will be made available from existing infrastructure at the time of development in accordance with the relevant service provider's specifications.

**Chris Hansen**

7th April 2017

# Attachment A

Services Assessment Report

# SERVICES ASSESSMENT REPORT

Homestead Bay Residential
June 2017

CLARK FORTUNE MCDONALD & ASSOCIATES
REGISTERED LAND SURVEYORS, LAND DEVELOPMENT & PLANNING CONSULTANTS

CONTENTS

# 1 INTRODUCTION

Clark Fortune McDonald & Associates (CFM) has been engaged to assess servicing options for a proposed rezoning on land located at Homestead Bay between Jacks Point and Lake Wakatipu.

The proposal seeks to re-zone land from rural general to residential activities.

The site is legally described as Lots 6, 7 DP 504891 & Lot & 8 D.P.443832. The total site area comprises approx. 200 ha and is contained in CT's 760709, 760710 & 555575 respectively.

The site has frontage to Kingston Road (SH6) and from Maori Jack Road.

This report is preliminary and for the planning map hearings for the QLDC District Plan Review only. Further information and detailed engineering design will be required if development proceeds.

The report considers infrastructure demands based on the proposed residential activities.

# 2 SCOPE OF WORK

The scope of work includes examination of existing QLDC as-built records, confirmation of capacity of existing services to determine the adequacy of the existing infrastructure, and recommendation of infrastructure servicing options.

# 3 DESIGN STANDARDS

Site development standards include, but are not limited to, the following:

- QLDC Land Development and Subdivision Code of Practice adopted June 2015.
- NZS4404:2010
- Drinking-Water Standards for New Zealand 2005.
- NZS PAS 4509:2008, New Zealand Fire Service Fire-fighting Water Supplies Code of Practice.
- Water for Otago, Otago Regional Council regional water plan.
- Document for New Zealand Building Code Surface Water - Clause E1 / Verification Method 1.

# 4 PROPOSED REZONING

The change in zone proposes residential activities over the site in separate activity areas. The basis of the preliminary design considers a possible 715 dwelling equivalents (DE).

The following report examines the feasibility of connecting into the existing QLDC infrastructure or the establishment of new stand-alone infrastructure to service the residential demand.

The demand figures above are used in assessing demands for wastewater and water supply in the following sections of the services report.

# 5 WASTEWATER

## 5.1 Design flows – Homestead Bay Residential activities

Demand based on anticipated activities has been determined in accordance with the development standards:

Refer QLDC Infrastructure code.

| | |
|---|---|
| No of residential units/DE: | 715 |
| Average dry weather flow: | 250 l / person / day. |
| Dry weather diurnal peak factor: | 2.5. |
| Infiltration factor: | 2. |
| Occupancy: | 3 person / du. |
| ***Dry weather average daily flow:*** | ***536 $m^3$ / day.*** |
| ***Peak hour flow:*** | ***31.0 l / sec.*** |

## 5.2 Proposed Servicing for the Homestead Bay Residential activities

Lowe Environmental Impact prepared a report for wastewater options in May 2017. This report investigated 130 of the proposed 715 DE's proposed for this zone.
The report considered 4 different options each of which were considered feasible.

It is the our expectation that each of the options are still viable for the added demand by scaling up the system to cater for the added demands. It is likely that the additional dwellings will result in a better per lot cost for some of the options.

There is also considered enough suitable land available for on site disposal options within the development area for the fully developed scenario.

## 5.3 Required upgrades

If the option to connect to QLDC infrastructure was chosen, any effects on the QLDC's wider infrastructure being the Shotover Waste Water Treatment Plant will be mitigated by the imposition of headworks fees at the time of connection to Council's service.

Upgrades to the Shotover Waste Water Treatment Plant are currently under construction.

# 6 STORMWATER

The development of the site area will increase stormwater runoff and introduce contaminants into the receiving aquatic environment.

## 6.1 Stormwater Catchment Management Plan (SCMP)

It is proposed that the Homestead Bay residential area prepare and submit to QLDC a SCMP to be approved by QLDC prior to development of the site.

## 6.2 Stormwater Catchments

The topography of the development area is predominantly of gentle slopes. The site aspect is south westerly facing and falling towards Lake Wakatipu.

The development area sits between two sharply incised gullies that take stormwater run-off from the flanks of the Remarkables. The north western gully contains the discharge from Jacks Point to the north. The Southern gully is a natural boundary of the site and separates the development area from Lakeside Estates.

The total catchment or study area is approx. 240ha on the western side of SH6. Previous stormwater reporting peer reviewed by Flood Sense Ltd in October 2016 demonstrates that the Highway acts as a cut-off channel preventing flows from the Remarkables crossing the road.

The run-off from the development area will ultimately discharge to Lake Wakatipu so no downstream land is affected by the discharge from the development area.

## 6.3 Existing Reticulation

There is no existing storm water reticulation to service the property.

## 6.4 Hydrological analysis

Runoff will need to be considered based on the proposed re-zoning plan. The development area is 71 ha and presently consists mainly of pasture. The soil drainage is moderate and the development area is quite flat, so a slope correction of -0.05 would appropriately be applied to the runoff coefficient for each surface type. Runoff coefficients have been obtained from Approved Document for New Zealand Building Code, Surface Water, Clause E1. Rainfall intensity has been determined from NIWA HIRDS V3 (http://hirds.niwa.co.nz/).

Given that the discharge of stormwater ultimately is direct to Lake Wakatipu it is anticipated that the stormwater catchment management will be relatively straight forward.

## 6.5 Runoff quality

Stormwater can contain a number of contaminants which may adversely affect the receiving environment. Studies in New Zealand and abroad have identified urban development as a major contributor to the declining quality of aquatic environments. It is estimated that upwards of 40% of the contaminant content of this runoff can be attributed to run-off from roads.

At this site stormwater will be generated by run-off from the following:

- Roofs of residential buildings;
- Urban roadways;
- Footpaths; and
- Other hard-standing areas.

Based on available information it is expected that stormwater from the above named developed surfaces could contain the following contaminants:

- Suspended solids;
- Oxygen demanding substances;
- Pathogens; and
- Dissolved contaminants.

The dissolved stormwater contaminants of concern at this site can cause an aquatic risk to the ecology of the receiving environment. The parameters of concern are as follows:

(1) Hydrocarbons and Oils

These are associated with vehicle use, although there is potential for spillages of hydrocarbon products to occur. They may be in solution or absorbed into sediments. Routine stormwater discharges are likely to have low concentrations ranging between 1 and 5g/m$^3$ total hydrocarbons over each storm event.

(2) Toxic Metals

A variety of persistent trace-metal compounds are carried in stormwater in both solid and dissolved forms. The most commonly measured metals of concern are zinc, copper, and chromium (mostly associated with vehicles and roads).

(3) Nutrients

Fertiliser application and animal waste associated with the current agricultural use of the site have the potential to generate high levels of nutrients such as phosphorus and nitrogen within stormwater runoff. High nutrient levels are not anticipated within the post-development stormwater runoff as, agricultural activities, such as grazing in particular, will cease.

### 6.5.1 Expected Contaminant Levels

Ranges of contaminant levels area provided by both the Auckland Regional Council (TP 10 and 53) and NIWA (Williamson 1993). This data can be used to predict the likely contaminant loading levels associated with changes in land use. Contaminant levels anticipated for this development have been estimated from TP10 and are included in Table 1 below.

**Table 1 – Estimated Contaminant Loading Ranges for Land Use Types (kg/ha/year)**

| Land Use | Total Susp. Solids | Total Phosph. | Total Nitrogen | BOD | Lead (median) | Zinc | Copper |
|---|---|---|---|---|---|---|---|
| Road | 281-723 | 0.59-1.5 | 1.3-1.5 | 20-33 | 0.49-1.10 | 0.18-0.45 | 0.03-0.09 |
| Residential | 60-340 | 0.46-0.64 | 3.4-4.7 | 12-20 | 0.03-0.09 | 0.07-0.20 | 0.09-0.27 |
| Pasture | 103-583 | 0.01-0.25 | 1.2-7.1 | NA | 0.004-0.015 | 0.02-0.17 | 0.02-0.04 |
| Grass | 80-588 | 0.01-0.25 | 1.2-7.1 | NA | 0.03-0.10 | 0.02-0.17 | 0.02-0.04 |

### 6.5.2 Construction-Stage Stormwater

Construction stage stormwater has the greatest potential to cause discharge of sediment laden runoff to the receiving environment. We would suggest that the applicant provide details of the proposed stormwater management plan as part of the engineering design phase of the project.

The detention ponds will be designed generally in accordance with Auckland Regional Council TP10. Each pond will have a fore-bay and will be suitably vegetated. The detention ponds will provide stormwater treatment before it is discharged to ground. The primary contaminant removal mechanism of all pond systems is settling or sedimentation.

## 6.6 Stormwater Management Objectives

The following draft overall objectives should be recognised while assessing stormwater management options for the development area:

- Primary protection for 25 year ARI storms;
- Secondary protection (overland flowpaths) for 100 year ARI storms;
- Regulatory Compliance;
- Avoidance of increases in downstream peak flows resulting from the increase in developed surface areas;
- Sustainable management of the effects of the proposed development;
- Minimisation of pollution of receiving waterways through the reduction of stormwater contaminants from roadways;
- Erosion protection in the stormwater discharge zone;
- Construction and maintenance costs.

## 6.7 Stormwater Management Approaches

This Section of the report introduces options available for stormwater management, in particular traditional design (big pipe), Low Impact Design (LID) or Sustainable Urban Drainage (SUD) approaches.

### 6.7.1 Traditional Approaches (Big Pipe)

The traditional approach to stormwater management has been to direct all runoff from residential allotments and roadways to a pipe network which discharges to the nearest receiving water body, with minimal effort made to replicate the pre-development hydrological regime.

Arguably the big pipe approach has one advantage over LID and SUD approaches: lower construction and maintenance costs.

### 6.7.2 LID / SUD Approaches

Some LID options are presented below. These have been sourced from the *Low Impact Design Manual* for the Auckland Region TP124 (Shaver et al. 2000), the *On-Site Stormwater Management Guideline* (NZWERF, 2004) and *Waterways, Wetlands and Drainage Guide* (CCC, 2003).

- Clustering and alternative allotment configuration. Fewer, smaller allotments, with more open space. This approach is less economic for the Developer and is also at odds with some of the principals of modern urban design.
- Reduction in setbacks. Reduction in the front setback reduces the length of driveway required. Correspondingly, the total amount of impervious area within the development is reduced. This approach presents some compliance issues with QLDC District Plan rules.
- Reduction in developed surfaces. This approach applies mainly to transport related aspects of residential developments such as reduced carriageway widths, use of grassed swales as opposed to kerb & channel, and alternative turning head design.
- Vegetated filter strips and swales. Stormwater from roadways is directed through a densely vegetated strip, and then into a road-side swale. Swales are generally used for conveyance of stormwater however they do have contaminant removal properties such as sediment removal efficiency of 20 – 40% (Waterways, Wetlands and

Drainage Guide, CCC 2003). Stormwater velocity is reduced so this approach is beneficial in reducing peak flows.

- Infiltration Trench. Infiltration trenches can be constructed in place of swales if natural soils are sufficiently free draining. This is applicable to sites with limited available open space. Infiltration trenches also have the ability to store stormwater. Infiltration trenches can reduce peak flows however they present maintenance issues.
- Infiltration Basin. The suitability of this option is reliant upon free draining natural soils, adequate depth to groundwater, and sufficient open space to construct.
- Soakage chambers. These allow direct discharge of stormwater to groundwater or free drainage soils. Soakage chambers require clean, pre-treated stormwater.
- Permeable paving. This option allows stormwater to permeate directly into pavement layers, and is applicable for low traffic areas with low ground water levels and free draining non-cohesive soils. Construction and maintenance costs for this option are high.
- Detention Ponds. These are used to reduce peak discharges to pre-development levels. They allow for settlement of suspended solids by vegetation. They require sufficient open space to construct.

## 6.8 Management Options

Many options are available to avoid, remedy or mitigate the adverse effects associated with residential development on receiving environments.

For the current project the recommended stormwater management strategy is to provide an integrated treatment train approach to water management, which is premised on providing control at the catchment wide level, the allotment level, and the extent feasible in conveyance followed by end of pipe controls. This combination of controls provides a satisfactory means of meeting the criteria for water quality, volume of discharge, erosion and flood control (if required).

**Table 2 – Recommendations**

| | Recommendations | Remarks |
|---|---|---|
| **Collection** | Combinations of LID/SUD measures, kerb & channel, swales, open channels and pipes. | (1) Where allotment density allows direct roadway runoff to grass swales (primary treatment) – also for secondary overland flow during flood events.<br>(2) Where natural soils allow incorporate infiltration measures.<br>(3) Kerb & channel & pipework to provide primary protection. |
| **Treatment** | Combinations of swales, detention ponds and end of pipe structures (gross pollution traps and filters). | (1) Pipework to discharge to detention / infiltration ponds.<br>(2) End of pipe structures and fore bay bunds to provide pre-treatment of stormwater |

| | | |
|---|---|---|
| | | before infiltration to ground water. |
| **Disposal** | Use attenuation prior to discharging to watercourses. | (1) Sufficient space is available to construct detention ponds.<br>(2) Where natural soils allow incorporate infiltration ponds.<br>(3) Post development discharge not to exceed pre-development levels. |

## 6.9 Stormwater Concept Design

Runoff from undeveloped areas shall be directed around the developed areas via grass swales, and then discharged to ground. This will replicate the pre development runoff scenario for the undeveloped areas. The developed areas will be serviced using a hybrid LID/SUD/Big Pipe design. This will incorporate a combination of grass swales, kerbs, pipework and detention areas.

The development area can be broken into smaller sub-catchments: Separate pipe networks are then proposed - one for each catchment. Each network will discharge to the stream, gully or directly to the Shotover River. Secondary overflow paths will be provided for in swales or road ways. Overflows will discharge to the same locations as the pre-development scenario.

# 7 WATER SUPPLY

## 7.1 Water supply design

To assess the demand and supply requirements for the proposed Homestead Bay residential area the following aspects have been considered:

- Water demands
- Water availability
- Existing infrastructure
- Storage requirements
- Irrigation requirements

## 7.2 Design flows – Homestead Bay Residential – QLDC

Demand based on the anticipated activities for the Homestead Bay Residential area have been determined in accordance with the development standards:

| | |
|---|---|
| Refer QLDC code of practice 6.3.5.6. | |
| No of residential units: | 715. |
| Average daily demand: | 350 l / person / day. |
| Occupancy: | 3.0 person / du. |
| Peak Day factor: | 6.6. |
| | |
| ***Average Daily demand:*** | ***751 $m^3$ / day.*** |
| ***Peak day demand:*** (16 hour pumping) | ***86.0 l/ sec.*** |

QLDC Code of practice also allows for a lower demand when supported by metering data approved by QLDC. Shotover Country has completed a 12 month metering trial on 50 randomly selected houses. The trial results indicate that an acceptable demand is 350l/p/day.

## 7.3 Required Fire fighting demand

The design of the new water infrastructure will need to meet the requirements of SNZ PAS 4509 – NZ Fire Service Firefighting Water Supplies Code of Practice.

### 7.3.1 Residential fire fighting demand – reticulated supply - non sprinklered

| | |
|---|---|
| Water supply classification: | FW2. |
| Required water flow within 135m: | 12.5 l / sec |
| Additional water flow within 270m: | 12.5 l / sec. |
| Max No. of hydrants to provide flow: | 2. |
| Minimum pressure | 100kPa. |
| Minimum static storage requirement | 45$m^3$ |

## 7.4 Existing Infrastructure

There is a 250mm NB PE water pipe laid in Maori Jack Road from the intersection of the Lodge Road heading towards Lake Wakatipu approx. 600m.

This pipe could be connected to the Coneburn Water supply system and extended to service the development area. Currenlty however it is not connected to any scheme.

Further analysis would need to be undertaken of the Coneburn system to determine what upgrades might be required.

## 7.5 Concept Design

Under development currently on site is a new 300mm water bore adjoining Lake Wakatipu. Test pumping and aquifer analysis of the new bore is set to commence from 15th June. Preliminary bores and testing indicate excellent quantity of water at secure depths.

On conclusion of the aquifer analysis design for a communal water supply can be completed to supply potable water to the residential demand.

It is anticipated that an "on-demand" system similar to that at Shotover Country can be developed.

We note that the existing 300mm Shotover Country bore services approx. 950 dwellings and this residential scenario is similar in scale and nature.

The new system would also include a water treatment plant that will treat the water at the source and be pumped to areas of development and to a high level water reservoir that will buffer peak flows and provide static fire fighting reserve.

From the reservoir internal reticulation within the development would be sized accordingly but is anticipated that mains of 200mmØ would be required if arranged in ring formations where possible.

It is proposed that a new reservoir could be established on the Jacks Point hill to the west of the development at a suitable elevation to service the development. The applicant is able to provide the land necessary for the establishment of a reservoir and is able to provide the access required.

The new tank elevation will be very similar to the existing Coneburn water reservoir. There may be opportunities to link the reservoirs to provide security of supply and redundancy in the network.


Sizing of the reservoir should also be carefully considered as this could help eliminate peaks in the demand. This would then allow for a lower peak flow of water to be taken from the new system.

All new infrastructure constructed for this development could be vested in Council ownership.

QLDC may choose to pump the water further afield to meet the demands of other developments in the District.

The further design and modelling of the infrastructure would need to be undertaken closely with the QLDC to confirm availability of supply. It is anticipated that QLDC water modelling consultants will be needed to carry out this modelling at the next phase of design.

## 7.6 Required upgrades

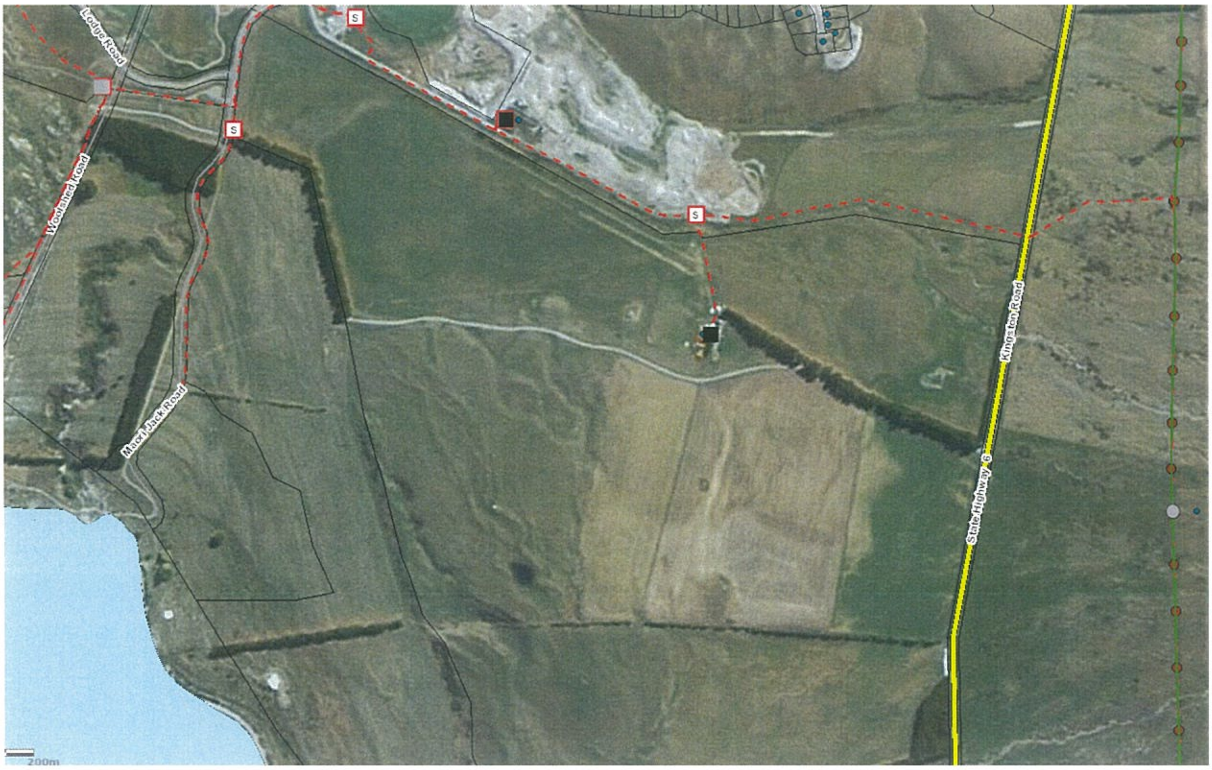
At this stage, it is not considered likely that connection will be made to existing QLDC infrastructure. Therefore no upgrades of QLDC assets are anticipated. Similarly if a new scheme is established for this development, all costs of establishing the new infrastructure would be borne by the applicant.

# 8 POWER, TELECOMMUNICATIONS AND GAS

Aurora Energy has high voltage 33kVa network running south on the eastern side of SH6 Kingston Road. There is an existing High voltage underground connection that feeds into the south end of Jacks Point and the existing activities in the Homestead Bay area.

We understand that Powernet are in the process of extending network as far as Hanley Farms approx. 2km away to the north.

We consider that either network could supply suitable underground electrical supply to the proposed development. Below is a screen shot from Aurora's GIS showing the existing electrical infrastructure.




Chorus fibre optic telecommunications cables exist in State Highway 6. It is anticipated that connection to the network can be made and that the new development would be serviced with fibre to the door.

All infrastructure is underground. All necessary mains will be extended to service the development area as development proceeds. Confirmation from the network owners will be obtained at each stage of development prior to proceeding.

It is not anticipated that there will be any supply or capacity issues for these services and connection will be made available from existing infrastructure at the time of development in accordance with the relevant service provider's specifications.

## 9 CONCLUSION

The proposed re-zoning of the Homestead Bay Residential Area is not considered to have any impacts on the infrastructure network. Infrastructure already exists that can be augmented as required to cater for additional demand or new infrastructure can be developed to service the residential activity proposed.

The infrastructure will be constructed and paid for the by the applicant as the development proceeds. It is anticipated that new infrastructure required would be constructed at little or no cost to QLDC. It is possible that the construction of new infrastructure required for this development could also have a wider network or community benefit by augmenting or providing additional security to existing infrastructure.

Stormwater would be managed for the development on site and is not expected to have any effects on existing infrastructure.

Other non-Council infrastructure and network utilities exist and have capacity to supply this development. Should additional capacity to accommodate the cumulative demand of the residential on the non Council infrastructure be required, it can readily be provided.

# Attachment B

Homestead Bay Wastewater Treatment
Options Report

# Homestead Bay Wastewater Treatment Options Report

Prepared for

**Murphy's Developments Ltd**

Prepared by

Lowe Environmental Impact

May 2017

# Homestead Bay Wastewater and Land Treatment

# Options Report

This report has been prepared for **Murphy's Developments Ltd** by Lowe Environmental Impact (LEI). No liability is accepted by this company or any employee or sub-consultant of this company with respect to its use by any other parties.

| Quality Assurance Statement | | |
|---|---|---|
| Task | Responsibility | Signature |
| Project Manager: | Rob Potts | |
| Prepared by: | Terry Hughes | |
| Reviewed by: | Rob Potts | |
| Approved for Issue by: | Rob Potts | |
| Status: | Draft | |

**Prepared by**: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# 1 INTRODUCTION

## 1.1 Overview

Murphy's Developments Limited (MDL) proposes to develop an area of land for residential use, located on the eastern side of Lake Wakatipu, south of Queenstown, in an area known as Homestead Bay. It is located approximately 8.5 km south of Queenstown Airport and directly to the south of Jacks Point residential area and golf course. Current access to the proposed Homestead Bay subdivision is via the airstrip access road from State Highway 6. The location is denoted by NZMG reference 5560044.9N, 2174807.2E. The Lake Wakatipu locality is an area of cultural, natural, historic, recreational and commercial importance with high value placed on both lake water quality and the natural environment.

The Homestead Bay development area is currently zoned under the Queenstown Lakes District Plan for rural activity. MDL aims to change the activity status to residential via a plan change.

MDL has approached LEI via Clark Fortune McDonald & Associates to prepare this "Options Report" which assesses the viable methods of wastewater (sewage) treatment and disposal or reuse options. This includes assessing effluent land application, the recommended sites, loading rates, land uses, set-backs, management constraints, potential for staging, and pumping to the QLDC Municipal Plant. LEI has also provided rough order costing undertaken to allow comparison of options. The costing includes the major components and likely annual operating costs via a Net Present Cost (NPC) analysis.

## 1.2 Project Scope

Lowe Environmental Impact (LEI) has been engaged by MDL to provide technical support for the treatment and dispersal of water for the Homestead Bay community. This "Options Report" provides MDL with information on onsite wastewater treatment and effluent land application, discharging to the QLDC Municipal Treatment Plant or to the Jacks Point community treatment plant, along with operating and capital cost expenditure.

The aim of this report is to provide MDL with sufficient information to assess which options are available and economically viable to support a Plan Change. It can also be used to make an informed decision, as to the most suitable method, for the treatment and either disposal or dispersal of effluent to land from the Homestead Bay community.

# 2 COMMUNITY WASTEWATER TREATMENT OPTIONS

## 2.1 Population and Design Flow Rates

The Homestead Bay development population and design flow rate is based on 130 dwelling equivalents.

The design of the neighbouring Jacks Point community wastewater treatment scheme based the peak occupancy ratio on 5 people per household. This was derived from a Kingston Morrison population survey (150 houses – approximately 10 % of total properties) over the peak summer weeks in Wanaka for Queenstown Lakes District Council in 1995/1996. The survey showed permanent residents averaged at 1.6 people per household and occupancy peaked at just over 5 people per household for 5 days and over 4 people per household for 16 days.

The Queenstown Lakes District Council (QLDC) Community Plan (2004), states that in 2001 Wanaka had 3,300 permanent residents living in 1,400 dwellings. This equates to an average of 2.3 people per household. In addition to these occupied dwellings, there were around 1,100 dwellings that were not occupied on a permanent basis. During busy periods (summer and winter) the population numbers grow significantly and estimates suggest that residents and visitors could total up to 12,000 people on a peak day. Based on the total dwelling figures this equates to an average of 4.8 people per household.

The average household size in Queenstown Lakes District is 2.5 people, compared with an average of 2.7 people for all of New Zealand (NZ Statistics, 2006 Census). NZS 4404:2010 Land Development and Subdivision Infrastructure recommends that the design flow shall be calculated by the method nominated by the territorial authority. In the absence of such information, then the number of people per dwelling should be based on 2.5 to 3.5 along with the average dry weather flow being between 180 to 250 L/person/day.

AS/NZS 1547:2012 "*On-Site Domestic Wastewater Management*" recommends a typical domestic wastewater flow allowance of 200 L/person/day for reticulated community or a bore water supply.

Table 2.1 summarises the recommended design flow rate and population/flow allowances for Homestead Bay using two methods.

**Table 2.1: Design Flow Rate**

| Number of Dwellings | Population per Dwelling (people) | Flow Allowance (L/person/day) | Design Flow Rate ($m^3$/day) | Annual Flow Rate ($m^3$/year) |
|---|---|---|---|---|
| 130 | 5 | 200 | 130 | 16,624 [a] |
| 130 | 2.5 | 200 | 130 [b] | 23,725 |

(a) Based on 5 people for 5 days, 4 people for 16 and 1.6 people for the remainder
(b) Flows from NZS4404 with 2 x peaking factor for WWF.

There has been a significant amount of population data collected for the Queenstown Lakes District and the design specifications, shown in Table 2.1, are in line with the Kingston Morrison Survey, NZ Statistics data, the Queenstown Lakes District Council (QLDC) Community Plan and NZS4404.

## 2.2 Sewer Reticulation Options

LEI considers there to be four available sewer reticulation options for the Homestead Bay community wastewater treatment system, as follows:

1. Sedimentation Tank Effluent Pumping (STEP) system;
2. Sump and grinder pump pressure sewer system;
3. Modified gravity system; and
4. Vacuum sewer.

The following sections detail the four reticulation options.

### 2.2.1 Option 1 – STEP System

Wastewater from each dwelling is collected in an on-lot Sedimentation Tank Effluent Pumping (STEP) Unit. This is a specialist onsite sedimentation (or interceptor) tank fitted with a pumping assembly which will pump liquid waste (effluent only, no solids) to the communal treatment system via the effluent sewer network.

Each interceptor tank would be connected to the wastewater main effluent sewer line via a service connection. This service connection protects the house from back-pressure and allows the house to be isolated from the effluent sewer in an emergency.

Typically the main collection lines will be 63 mm diameter medium density polyethylene. No manholes or minimum gradients are required. The pipe work is generally buried in a common services trench at least 450 mm below ground level at variable grade, i.e. it can follow the contour.

By removing the solids from the wastewater prior to transporting, the collection pipes can be smaller (e.g. 63 mm diameter) and can be laid in shallow trenches without the requirement for minimum gradients and velocities. The system will be effectively sealed meaning the treatment plant can be sized considerably smaller since it does not have to cope with large wet weather flows. A shallow system is desirable in areas of high groundwater.

There are two options available for the installation of a STEP system, as follows:

1. Shared STEP unit per two households.
2. One STEP unit per household.

Savings can be made, without compromising the system performance, by installing one STEP unit for every two dwellings; however, MDL should be aware of the following issues:

1. **Ownership**: Is the sedimentation tank owned equally by each residence, or owned by MDL, or Body Corporate, or vested to QLDC?
2. **Power:** How much of the Power does each residence pay; is this divided on a prorata basis or 50/50 split?
3. **Maintenance:** If maintenance is required due to a system failure, who takes responsibility for the cost of such maintenance. This is important if the failure is a result of poor management on the part of the occupants of one dwelling only. Who pays for septage pump-out at about 10 yearly intervals?
4. **Location:** Which property is the STEP unit sited on, or is it in public areas (roadside)?

It is recommended that a suitable management plan be prepared and that a copy be made available to each household.

### 2.2.2 Option 2 – Sump and Grinder Pump/Pressure Sewer

Pressure sewerage systems consist of a network of on-lot grinder pumps and medium to high pressure pipes, which integrate to form a collection system.

Gravity lateral pipes from any dwelling connects to an on-lot sump containing a purpose built pump and grinder unit. Wastewater is then discharged in the form of watery, finely ground slurry into small-diameter pressure piping. In a completely pressurised collection system, all the piping downstream from the pumping unit will normally be under pressure (45 m or less). Pipe sizes will start at 40 mm outside diameter polyethylene for property discharge lines.

Polyethylene pipe is usually used for the pipe network, which is fully sealed by electrofusion welding of joints or couplings. Depending on the topography, size of the system and planned rate of build out, appurtenances may include isolation valves, flushing points, air release valves at significant high points (if required), and check and stop valves on the property boundaries at the junction of each property connection with the main.

This system provides watertight reticulation and is similar to that of Option 1 in most facets. Primary treatment can take place at the treatment plant and if required the primary tank can be used as a carbon source for enhanced nitrogen removal. Ownership and maintenance issues are similar to STEP tanks but without the need for a 10 – 15 year pump-out.

### 2.2.3 Option 3 – Modified Gravity and Central Pumping

Wastewater is reticulated via gravity, from each dwelling, to one or more pump stations (this potentially can be at the sewage treatment plant). This option results in no solids removal prior to the treatment plant, thus pipes need to be larger and laid at sufficient gradient to convey solids. However, the system is modified from that of a conventional sewerage system; the modified sewers involve smaller diameter flexible pipe systems with limited manholes compared to conventional systems.

Modified gravity systems can be prone to stormwater ingress because, whilst utilising flexible pipe and fewer manholes over that of a conventional gravity system, they are not completely sealed and therefore can potentially result in a wet weather in-flow requiring a larger capacity wastewater treatment plant. However, wet weather flows are generally less than conventional gravity systems.

### 2.2.4 Option 4 – Vacuum Sewer

Vacuum systems operate under the principle of differential air pressure as the driving force. The sewer lines are under a vacuum of -50 kPa to -70 kPa, created by vacuum pump/s located within a vacuum pump station.

The pressure differential between the atmospheric pressure and the vacuum in the sewer lines provides the energy required to open the vacuum interface valves and to transport the sewage. Sewerage flows by gravity from homes into a collection sump. When 40 L accumulates in the sump, the vacuum interface valve located above the sump pneumatically opens and differential air pressure propels the sewage through the valve and into the vacuum main. Sewage flows through the vacuum lines and into a collection tank at the vacuum station. Sewage pumps transfer the sewage from the collection tank to the wastewater treatment facility. There are no electrical connections required at the home. Power is necessary only at the vacuum station.

The differential air pressure propels the sewage at velocities of 4 – 6 m/s, disintegrating solids while being transported to the vacuum station. The valve stays open for 4 – 6 seconds during this cycle. Atmospheric air used for transport enters through the 100 mm screened air inlet on the gravity line. There are no odours at this air inlet due to the small volumes of sewage and short detention times in the sump.

## 2.3 Wastewater Treatment Plant Options

Three treatment systems were considered for the treatment of the Homestead Bay wastewater. These include the following.

1. Community/decentralised treatment on-site using a package treatment plant, such as a Recirculating Textile Packed Bed Reactor;
2. Connect to the Queenstown Municipal Treatment Plant;
3. Connect to the Jacks Point Community Recirculating Textile Packed Bed Reactors.

There are several types and numerous suppliers of package treatment plants in New Zealand. They are generally variants of activated sludge technology and all meet secondary treatment quality standards. We have used two examples here – Recirculating Packed Bed Reactors (rPBR) and Sequencing Batch Reactor (SBR).

rPBR's are well established in New Zealand for small communities, giving a high-quality effluent and generally function well under fluctuating loads. This type of system is commonly used for community on-site wastewater where a high level of organic treatment, nitrogen reduction and the removal of pathogens are important considerations. An earlier version of what is now available is installed at neighbouring Jacks Point.

Gunn (2012) discusses the option of utilising a SBR; whilst this type of treatment technology could be employed for treatment of the Homestead Bay wastewater and does have advantages over other systems e.g. small foot print and can produce high quality effluent, LEI considers that it is not ideal for the following reasons:

- High volume of sludge production;
- High operation and maintenance requirements; and
- High operating costs.

Table 2.2 provides a summary of the advantages and disadvantages of the SBR and rPBR systems. Each system has been awarded a score of between 1 and 3 (3 indicating most desirable, 1 indicating least desirable).

**Table 2.2: Summary of Wastewater Treatment Options** (3 = Best, 1 = Least Desirable)

| Parameter | SBR | | rPBR | |
|---|---|---|---|---|
| | Description | Score | Description | Score |
| Capital expenditure | Moderate | 2 | Moderate | 2 |
| Running costs | High | 1 | Moderately Low | 3 |
| Additional carbon dosing | Yes | 1 | Usually not | 2 |
| Power requirement | High | 1 | Low | 3 |
| Maintenance requirement | High | 1 | Moderate | 2 |
| Sludge production | High | 1 | Low | 3 |
| Suitable for intermittent flow regimes | Yes if in parallel or balance tank | 3 | Yes | 3 |
| Noise | Moderate | 2 | Low | 3 |
| Remote servicing and trouble shooting | No, needs operator | 1 | Yes | 3 |
| Visual impact | Moderate | 2 | Low | 3 |
| Operation simplicity | Needs frequent operator input | 1 | Low operator | 3 |
| Odour production | Moderate | 2 | Low | 3 |
| Reliability | Moderate | 2 | High | 3 |
| Effluent treatment | High | 3 | High | 3 |
| **Total Score** | | **23** | | **39** |

SBR technology requires a high 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 effluent quality. SBR's are an aerated technology and therefore requires a high power input, significantly exceeding that of a rPBR system; as a result of the high level aerobic microbial activity a large volume of sludge is produced requiring disposal. rPBR units are able to handle varying inflows through a high recycle ratio, whilst providing high quality effluent using simple systems that require low operation and maintenance requirements.

For the above reasons, LEI has not considered SBR technology further. Other package plants are available, such as submerged aerated filter systems. These have similar advantages and disadvantages as the SBR.

### 2.3.1 Option 1 – On-site Recirculating Textile Packed Bed Reactor (rPBR)

The recirculating packed bed reactor is a multiple pass packed bed aerobic wastewater treatment system. The packed bed media is an engineered textile, which has a high void capacity allowing for a large surface area. Wastewater enters a processing tank (recirculating tank) where anaerobic digestion and suspended solids removal can take place. Effluent is then pumped to the secondary treatment chamber where it percolates down through a textile media and is collected in the bottom of a filter pod. This process does not utilise forced aeration. From the filter pod, the flow is split (diverted) between the processing tank and the final discharge.

**Effluent Quality**

The expected effluent quality from a rPBR wastewater treatment plant is summarised in Table 2.3.

**Table 2.3: Expected Final Effluent Quality**

| Parameter | Typical Domestic Raw Wastewater | rPBR[(1)] |
|---|---|---|
| Biological Oxygen Demand (BOD, mg/L) | < 450 | < 20 |
| Total suspended Solids (SS, mg/L) | < 350 | < 25 |
| Total Nitrogen (TN, mg/L) | < 70 | < 35 |
| Total Phosphorus (TP, mg/L) | < 30 | < 5 |
| Faecal Coliform (cfu/100 ml) | $10^3 - 10^6$ | $< 10^4$ |

[(1)] Effluent quality gauged from supplier literature and Rotorua OSET Trial data.

Note that the colder temperatures in Central Otago means that nitrogen reduction in winter is difficult without significant heating or additional removal systems. Land based loading of N should be based on a mean concentration of 50 mg/L.

The rPBR effluent is considered to have been treated to a suitably high standard and is accepted by regulatory authorities as being suitable for land application.

### 2.3.2 Option 2 – Connect to the Queenstown Municipal Treatment Plant

This option requires the Homestead Bay wastewater to be pumped to the Queenstown Municipal Treatment Plant located on the true right bank of the Shotover River, between the river and the Airport Terrace.

Assuming a pressure sewer (no wet weather flow) to a main pump station, then there is a requirement for a flow rate of 4.5 L/s (peaking factor of 2.5 as per NZS4404) and approximately 10 km pipe run, between Homestead Bay and a manhole near Kelvin Heights, approximately 35 m of headloss across the pipe can be expected for a 100 mm diameter PVC pipe. Assuming a motor and pump efficiency of 60% and the density of wastewater being similar to water at 1000 kg/m$^3$; the pumping power required would be 3 kW. A duty-standby pumping system would be required, thereby having one pump on standby.

If the nearby Hanley Downs development is reticulated to Queenstown, then there is a possibility of conveying to Hanley Downs and joining that system; a distance of around 4 km.

This has not been discussed with Hanley Downs.

### 2.3.3 Option 3 – Jacks Point Community Treatment Plant

This option requires Homestead Bay wastewater to be pumped to the Jacks Point Community Treatment Plant located less than 500 m to the north. Only primary settled effluent can be exported to this system, therefore primary treatment would need to occur onsite utilising a STEP system. A community pump station would therefore not be required.

Flows would be buffered by the on-site tanks and would be less than that in Section 2.3.2 above.

This has not been discussed with Jacks Point.

## 2.4 Available Land Treatment Area Options

Should the option to install a community/decentralised wastewater treatment plant for the development be selected, then land treatment options need to be assessed.

The LEI site investigation looked at the potential land treatment area soils in detail. Sites A, B and C provides 3.4 ha of land usable land. The areas are all on the eastern boundary of the site. Based on the soil types, hydraulic conductivity, available area and terrain LEI considers all the areas identified (Areas A, B and C) to be suitable locations for effluent dispersal for land treatment.

### Soil Infiltration

Results of the testing for $K_{-40mm}$ are given in Table 2.1. The reported field measurements refer to clean water irrigation and are not considered to be suitable for continuous and sustained applications of wastewater.

In consideration of a wastewater application rate suitable for the investigation area, a conversion should be made to allow for the application of "enriched" water which has elevated levels of constituents (cations, anions, complex organic molecules). A value of 30% of the $K_{-40mm}$ has been adopted in-line with the recommendations of Crites and Tchobanoglous (1998) to provide a Design Irrigation Rate (DIR); which has been calculated for each site. Average results are presented in Table 2.1.

**Table 2.1: Soil Hydraulic Conductivity and Design Irrigation Rate**

| Sample ID | Soil Type | Phase | $K_{sat}$ (mm/h) | $K_{-40mm}$ (mm/h) | DIR (mm/d) (1) |
|---|---|---|---|---|---|
| Site 1 | Wakatipu sandy loam | Rolling | 76 ±19 | 3.3 ±1.8 | 24 |
| Site 2 | Eely sandy loam | Undulating | 226 ±64 | 4.5 ±2.4 | 32 |
| Site 3 | Wakatipu sandy loam | Undulating | 246 ±100 | 3.2 ±1.7 | 23 |

(1) Design daily irrigation rate based on soil hydraulic conductivity only (30% of $K_{-40mm}$)

The DIR for the site ranges from 23 mm/d to 32 mm/d. The adoption of the lowest DIR for the entire site is recommended. This will further protect the groundwater beneath the development and adjacent waterways including Lake Wakatipu.

LEI considers a design irrigation rate of an average of 5 mm/day to be acceptable and this will result in a land treatment area requirement of 2.6 ha (based on a peak rate of 130 $m^3$/d). This rate allows the system to be dosed every 3 -4 days at a higher rate, then rested.

## 2.5 Land Dispersal Options

Based on soil type and soil profile, soil permeability, groundwater levels, required treatment outcomes, the potential quality of the effluent from a secondary treatment plant, and the proposed end use for the land, LEI considers that subsurface irrigation is the most appropriate for the land application of the Homestead Bay effluent.

### 2.5.1 Land Treatment Area Vegetation

Effluent passing through a soil matrix is subjected to plant and microbial uptake, filtration, sorption and biological and chemical process; all of which reduce the contaminant constituents prior to leaching to groundwater. Plant uptake results in a reduction of nitrogen and phosphorus; both of which are required for plant growth. Nutrients if allowed to enter water, in excess of naturally occurring concentrations, can result in nuisance periphyton growth and potentially eutrophication. An important part of any land application design is choosing the correct vegetation type and maintenance of the established crop. Factors to consider when selecting a vegetation type are:

- Short rotation crops;
- Climatic conditions;
- Soil types;
- Environmental constraints;
- Effluent chemical composition;
- Effluent application system;
- Aesthetic requirements;
- Land use; and
- Nutrient and water uptake requirements.

Plant uptake will be higher during juvenile growth when nutrient requirements peak, therefore managing any crop to maintain this phase is essential. When selecting a plant species consideration must be given to the environmental conditions as well as the hydraulic loading and chemical composition of the effluent. Not all plant species require the same hydraulic or nutrient input for growth; therefore, fast growing species (short rotation crops) that require a high nutrient input is preferable.

Landuse of the land treatment area will generally be via the following three methods stated in order of preference:

1. Cut and Carry.
2. Sheep grazing.
3. Cut and Leave.

**Cut and Carry**

"Cut" refers to mowing grass or grass type crops, tree felling (replanting with juvenile plants) or pruning vegetation back to stimulate regrowth; "carry" refers to removing plant material off site for sale or grazing elsewhere. If vegetation is not removed offsite, biological decay will result in the transfer of nutrients held within the plant back into the soil matrix, with the net plant uptake being near zero. The most common form is the making of hay, silage or baleage.

**Sheep grazing**

Sheep grazing removes dry matter (and thus nutrients) and converts it to wool and meat but recycles some back to the soil store; in theory the net input of nutrients from sheep urine and faeces will be less than that carried offsite in a cut and carry regime. Sheep are generally rotated around the site to optimise grazing and vegetation removal.

**Cut and Leave**

This option is generally only applied to sites that are not easily accessible and for which vegetation removal will be difficult, or are managed as turf areas, e.g. golf courses, bowling greens, etc. The net result is limited nutrient removal offsite; the plant life cycle of regeneration and decay will inevitably result in most nutrients taken up by the plants, re-entering the soil matrix during the decay phase. However, plant uptake will slow the rate of nutrient leaching and nitrogen losses occur due to soil organic matter accumulation and biological denitrification, in addition, evapotranspiration will reduce hydraulic pressure on the soils.

### 2.5.2 Land Use and Buffers at Homestead Bay

MDL would prefer to have the final landuse as open space pasture. They also accept cut and carry, i.e. shutting up the area to make baleage or similar, with no stock (although putting sheep in following harvesting for a day or two to tidy up around the fence lines is acceptable).

Buffer distances to boundaries are not generally required for subsurface drip, however, to be conservative, a 5 m buffer has been allowed to all external eastern boundaries. Buffer distances to ephemeral waterways of at least 50 m have been allowed for. These reduce the available land area down to 3.04 ha. This is greater than the area required for hydraulic loading.

This results in a nitrogen loading in the order of 395 kg N/ha/yr. Cut and carry systems generally have N loading in the order of 450 – 600 kg N/ha/yr, so the loading should be acceptable for consenting purposes with Otago Regional Council.

## 2.6 Wastewater Disinfection

Generally, UV disinfection is not a requirement if the method of effluent dispersal is via subsurface land application.

Soils have the ability to reduce pathogens a 1 log cycle for every 150 – 200 mm passage through the soil matrix.

# 3 COST ESTIMATION

This section provides the approximate capital and operational and maintenance expenditure required for the reticulation and treatment of the Homestead Bay wastewater for each option. UV disinfection has not been allowed for due to the application being subsurface.

The cost estimations summarised in Tables 3.1 to 3.3 have been split into three different categories, as follows:

1. Table 3.1: Land application options:

2. Table 3.2: Within-site reticulation and treatment options:
   - Presents all within-site reticulation and treatment plant options.
   - Includes the land application option (as per Table 3.1) allowing for a complete expenditure analysis.

3. Table 3.3: Within-site reticulation and pumping off-site to QLDC municipal, or Hanley Downs.
   - Presents all within-site reticulation options (as per Table 3.2).
   - Presents the option of discharging to QLDC municipal or Hanley Downs rather than onsite treatment and land application.

Tables 3.2 and 3.3 also provide the 20 year net present cost (NPC) of each option based on a discount rate of 8.75%. The NPC is useful in allowing for a comparison between high capital expenditure/low operating costs and low capital expenditure/high operating cost options.

Note that the NPC analysis assumes that reticulation and treatment are installed on day 1, then annual operating and maintenance costs. However, in reality with the systems proposed, significant staging savings can be made as the systems are generally modular. This may not be the case with the pumped option to the QLDC municipal plant.

It should be noted that only conceptual development plans are available at this early stage in the development process; therefore, the cost estimates provided are a guide with a likely ±30% level of accuracy.

**Table 3.1: Homestead Bay Land Application Capital Expenditure**

| **Land Application ($)**<br>**Description** | **Area 4** |
|---|---|
| Subsurface drip irrigation ($50,000/ha) | 150,000 |
| Total (Capital Expenditure) | 150,000 |
| Capital Expenditure per Lot | 1,150 |
| Annual Pumping ($/year) | 1,100 |
| Maintenance – annual flushing/replacement ($/year) | 3,500 |
| Total (Running Cost, $/year) | 4,600 |
| Running Cost per Lot ($/year) | 35 |

Notes:

(1) Pumping costs based on $0.25/kW and 6 hours pumping day.

**Table 3.2: Homestead Bay Wastewater Onsite Reticulation and Onsite Treatment**

| Description | STEP | Pressure Sewer | Gravity | Vacuum Sewer |
|---|---|---|---|---|
| **Onsite Reticulation ($)** | | | | |
| Grinder tank/pump | | 884,000 | | |
| STEP tanks | 845,000 | | | |
| Boundary connection | 45,500 | 78,000 | 84,500 | 297,000 |
| Monitoring system | | | | 65,000 |
| Low pressure sewer | 121,000 | 121,000 | 363,000 | |
| Vacuum sewer | | | | 262,500 |
| Pump Station | | | 200,000 | 185,000 |
| Contractors P&G, design, mark-ups etc. | 20,000 | 20,000 | 97,500 | 65,000 |
| Total Reticulation (Capital Expenditure) | 1,031,500 | 1,083,300 | 745,000 | 874,500 |
| Total (per Lot) | 7,935 | 8,331 | 5,731 | 6,727 |
| **Onsite rPBR Treatment Plant ($)** | | | | |
| Primary treatment | | 236,710 | 236,710 | 236,710 |
| Pre-anoxic process | 56,000 | 56,000 | 56,000 | 56,000 |
| rPBR | 540,550 | 540,550 | 540,550 | 540,550 |
| Post anoxic process | 57,000 | 57,000 | 57,000 | 57,000 |
| UV disinfection | 0 | 0 | 0 | 0 |
| Total Onsite Treatment (Capital Expenditure) | 653,550 | 890,260 | 890,260 | 890,260 |
| Total (per Lot) | 5,027 | 6,848 | 6,848 | 6,848 |
| **Land Application/Treatment ($)** | | | | |
| As per Table 3.1 | 150,000 | 150,000 | 150,000 | 150,000 |
| **Annual Operations and Maintenance ($/year)** | | | | |
| Carbon Dosing (if required) | 2,880 | 1,440 | 1,440 | 1,440 |
| Pumping Power Cost | 1,463 | 2,966 | | 2,281 |
| UV disinfection (power/tube replacement/maintenance) | 0 | 0 | 0 | 0 |
| rPBR power costs | 4,745 | 4,745 | 7,120 | 4,745 |
| Pump station maintenance | n/a | n/a | n/a | 3,300 |
| Major service maintenance | 3,335 | 3,335 | 3,335 | 3,335 |
| Tank desludging | 3,900 | n/a | n/a | n/a |
| Reticulation maintenance | 2,200 | 2,200 | 26,000 | 2,200 |
| Treatment plant maintenance | 13,000 | 13,000 | 13,000 | 13,000 |
| Miscellaneous (consent compliance, grounds up keep etc.) | 15,000 | 15,000 | 15,000 | 15,000 |
| Land Application/Treatment Area (as per Table 3.1) | 4,600 | 4,600 | 4,600 | 4,600 |
| Total ($/year) | 51,123 | 47,286 | 70,495 | 49,901 |
| Total (per Lot) | 393 | 364 | 542 | 384 |
| Total Capital Expenditure ($) | 3,040,050 | 3,328,260 | 2,990,260 | 3,119,760 |
| Total Operations and Maintenance ($/year) | 51,123 | 47,286 | 70,495 | 49,901 |
| 20 year NPC ($) | 3,515,163 | 3,767,710 | 3,645,407 | 3,583,519 |

**Table 3.3: Homestead Bay Wastewater Onsite Reticulation and QLDC Municipal Treatment**

| Description | STEP | Pressure Sewer | Gravity | Vacuum Sewer |
|---|---|---|---|---|
| **Onsite Reticulation (as per Table 3.2) ($)** | | | | |
| Total (Capital Expenditure) | 1,031,500 | 1,083,000 | 745,000 | 874,500 |
| Total (per Lot) | 7,935 | 8,331 | 5,731 | 6,727 |
| **Pumping/Reticulation to QLDC Municipal ($)** | | | | |
| Reticulation to QLDC Municipal (10 km) | 900,000 | 900,000 | 900,000 | 900,000 |
| Pump station (duty standby 3 kW pumps/electrical/building) | 140,000 | 140,000 | 0 | 140,000 |
| QLDC Municipal connection fees | 913,900 | 913,900 | 913,900 | 913,900 |
| Total (Capital Expenditure) | 1,953,900 | 1,953,900 | 1,813,900 | 1,953,900 |
| Total (per Lot) | 15,030 | 15,030 | 13,953 | 15,030 |
| **Annual Operations and Maintenance ($/year)** | | | | |
| Pumping costs | 2,190 | 2,190 | 4,380 | 2,190 |
| Pump station maintenance | 5,000 | 5,000 | 7,500 | 5,000 |
| Major service maintenance | 8,000 | 8,000 | 8,000 | 8,000 |
| Tank desludging | 3,900 | n/a | n/a | n/a |
| Reticulation maintenance (including onsite) | 52,200 | 52,200 | 76,000 | 52,200 |
| Miscellaneous (consent compliance, grounds up keep etc.) | 5,000 | 5,000 | 5,000 | 5,000 |
| QLDC wastewater charges | 71,500 | 71,500 | 71,500 | 71,500 |
| Total ($/year) | 147,790 | 143,890 | 172,380 | 143,890 |
| Total (per Lot) | 84 | 82 | 99 | 82 |
| Total Capital Expenditure ($) | 2,985,400 | 3,036,900 | 2,558,900 | 2,828,400 |
| Total Operations and Maintenance ($/year) | 147,790 | 143,890 | 172,380 | 143,890 |
| 20 year NPC ($) | 4,358,891 | 4,374,146 | 4,160,918 | 4,165,646 |

(1) Estimate is based on topographical maps and further design information and can be considered to have a ±30% error.

(2) Development costs and rates charges have been provided by QLDC (2013) based on current rates and an evaluation of the proposed development. The development contribution is estimated at $7,030/lot. The annual rates contribution was estimated as being between $500 – $600; $550 has been used for the cost estimation shown.

If the wastewater is pumped to Hanley Downs rather than the QLDC reticulation system, then the above capital costs reduce by an estimated $500,000 primarily associated with the reduced reticulation distance.

The STEP system provides primary treatment and therefore the QLDC connection fee and rates charges may potentially be lower than estimated in Table 3.3; however, it cannot be stated with certainty at this early stage in the development process. The STEP primary treatment, will effectively buffer flows and mitigate many of the blockage issues associated with (non-primary treatment) sewer systems, allowing for a smaller rising main pipe diameter. However, the above are not likely to significantly change the outcome shown in Table 3.3.

Whilst the vacuum system does have a relatively low capital expenditure, there are a number of unknowns and it does not have a fully proven track record within NZ due to only two operational schemes having been installed. Experience within Christchurch suggests that the vacuum system may be prone to cost overruns from that stated in Tables 3.2 and 3.3 that were sourced from the equipment supplier. The vacuum system also provides very little in the way of attenuation;

therefore, should a fault arise, the reason for the fault and location must be determined immediately with a quick maintenance response or the system valve pits may overflow.

Table 3.3 specifies a pump station with low pressure reticulation, discharging wastewater to the QLDC municipal treatment plant. It should be noted that a pressure system could be utilised and if the headloss is below 55 m then potentially no pump station is required reducing capital and maintenance expenditure. However, the overriding factor in determining the cost of piping to the QLDC municipal treatment plant are development fees and rates charges.

# 4 SUMMARY

A number of options are suitable and viable for wastewater within the Homestead Bay site.

Based on the environmental conditions within the Homestead Bay vicinity, the required capital expenditure and operational and maintenance costs, initial investigations indicate that all within-site reticulation is feasible, however, due to issues with the vacuum systems installed in Christchurch City, vacuum is not recommended. LEI recommends either STEP or pressure systems are installed, as a gravity system requires designing the plant and land treatment area for wet weather flows.

Pumping to the QLDC reticulation system has a higher NPC by around $400,000 to $700,000 than onsite treatment and the construction cannot be easily staged; however, it should be noted that potential additional costs such as plant failures, land application issues, landuse for future subdivision, etc. are mitigated by this option and responsibility for treatment is no longer a local community responsibility. It should also be noted that the above costing does not take into account staging and this may change the NPC costing for within-site reticulation and onsite treatment/land application because staging can potentially reduce the initial capital expenditure for some of the options.

It is considered that the area of available land for effluent dispersal, their soils types, slope and depth to groundwater are suitable for effluent land treatment and management should be relatively straight forward. Development can also be undertaken in a staged manner. Generally, Regional Councils consider that a Council run sewerage system is usually the best outcome for the community, as maintenance and ownership issues are easily dealt with.

Should onsite treatment and effluent dispersal be preferred, LEI recommends the option summarised in Table 4.1. LEI consider that the onsite option is consentable through Otago Regional Council.

**Table 4.1: Recommended Homestead Bay Community Wastewater Scheme**

| | |
|---|---|
| **Vacuum or pressure sewer** | STEP or pressure sewer reticulation to treatment plant. |
| **rPBR treatment plant** | A recirculating packed bed reactor. |
| **UV disinfection** | Not necessary. |
| **Land treatment area** | Areas A, B and C all used with 5 m buffer to boundary. |
| **Irrigation method** | Subsurface drip irrigation placed 150 – 200 mm beneath the surface. |
| **Vegetation** | Grassed and maintained on a cut and carry basis. |

# 5 REFERENCES

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# Attachment C

Peer Review Stormwater Calculations

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# JARDINE LANDFILL: PEER REVIEW STORMWATER CALCULATIONS

## 1. INTRODUCTION

Flood Sense Ltd has been engaged by Clark Fortune McDonald to carry out a peer review of a stormwater report pertaining to a landfill proposal as outlined on CFM Plans 12471. The report and plans were produced in CFM's Queenstown office by Craig Woodcock, BSurv, MNZIS.

The approach taken by Flood Sense in this review has essentially been to view the proposal "from scratch", and to compare our assessments with those produced by CFM. This required a site visit, carried out on Saturday 1 October.

The inspection confirmed that the CFM report accurately described the location and purpose of the proposed works. The site exists at the head of a minor gully that drains an area of relatively gently rolling pasture between SH6 and the south arm of Lake Wakatipu. Historically, the gully would also have drained a small catchment located above SH6 on the considerably steeper slopes of the Remarkables. This subcatchment has long since been cut off from the lower catchment with the construction of SH6. The water table running parallel to the highway is intended to be adequate to prevent flows encroaching across the road, with a large culvert collecting the runoff from a wider area of the Remarkables and conveying the flow via a large and continuous gully to the lake.

The CFM report appears to assume that runoff from the upper subcatchment will be substantially or entirely cut off by the highway drainage system, and will not therefore contribute to the cleanfill area. The assumption, while appearing not unreasonable, is not well justified in the report. This review looks at the matter in rather more detail.

It is confirmed therefore that a lower flat and well vegetated catchment contributes to the cleanfill area, but that the potential contribution of runoff from above the highway should be more fully investigated.

## 2. DESIGN RAINFALLS

The report assumes a 100-year design return period, and a10 minute design rainfall intensity. The design rainfall is stated as being derived from NIWA's High Intensity Rainfall Distribution tabulations. We have confirmed that the assessed design rainfall intensity of 54mm/hr is of the right order (our calculations produced 57mm/hr), but we consider that the selection of a 10 minute design storm may be unrealistically short, leading to an overestimate of design rainfall intensities.

We have therefore produced an alternative series of runoff assessments. Our summary of findings is as follows:

**Scenario 1:** The lower catchment

We have estimated the area of the lower (i.e. below SH6) catchment to be of the order of 20 hectares. This is significantly greater than the 13 hectares assumed by CFM. It is conceded that the actual catchment is difficult to accurately determine, as the catchment boundaries are indistinct, even with the reasonably good contour information provided. Our estimate is conservative, in keeping with our approach to ensure that any error is on the side of safety.

Using standard BIA E1/VM1 methods, we have calculated a length of overland flow of approximately 660m, and an average surface slope of approximately 0.04, leading to a time of concentration of 35 minutes. Reducing the ToC to 30 minutes (also conservative) and using the projected HIRD's intensities for a 2 degree temperature increase (again conservative), we have produced a design rainfall intensity of 26mm/hr for a 10-year rainfall, leading to a design flow of 289 l/s. This compares with the 585 l/s derived by CFM for a 100-year flow.

It is considered that a 10 year flow is the more appropriate design parameter for swale design. Purely for purposes of comparison, our calculation for a 100myear flow is 522 l/s, approximately 10% lower than the CFM derivation. The similarity is essentially a consequence of what we would consider to be compensating challengeable assumptions in the CFM report.

**Scenario 2:** The upper catchment. The upper catchment (above SH6) is obviously steeper and, in its higher elevations, comparatively devoid of vegetation. We assumed a higher runoff coefficient of 0.60 for the upper reaches, and 0.35 overall. The overall slope is of the order of 14%, giving a ToC of 12 minutes. Allowing for a climate change adjustment of plus 2 degrees, we have assessed that the upper subcatchment will deliver a peak flow of approximately 545 l/s to the roadside water table.

We would routinely expect the highway authorities would ensure that the adjacent water table would be designed to ensure that such cross country flows can be accommodated without road damaging overflow occurring. The roadside water table is of uniform cross section, typically having a bottom width of at least 300mm and batters of around 1.5:1 on average. The channel slope was determined from the provided contours to be approximately 3%. The channel was observed to be in stable condition and lightly vegetated. Using a Mannings value of 0.04 (again, a conservative estimate), we derived a channel capacity of at least 1.5 $m^3/s$ (1,500 l/s). This is a full 3 times the assessed 100-year contribution from the above-road catchment of interest in this case.

## 3. THE GRAVEL FAN

A succession of gravel fans have formed above the State Highway as narrow gorges in the uooer levels of the Remarkables disgorge on to much wider expanses a few hundred meters above the road. These fans appear stable and well-vegetated. The State Highway appears to have generally been built below the lower extent of the fans. The fans could conceivably become active in the event of prolonged intense rainfall, but the proposed swale is located a further 600 meters below, across gently sloping pasture. No direct impact of gravels originating upstream of the State Highway is therefore envisaged.

More worthy of consideration is the possibility of fan-based gravels washing into the water table adjacent to the State Highway. The resultant blockage (or partial blockage) could direct limited flows from the upper catchment towards the proposed swale, but such blockage would be expected to be expeditiously cleared by the roading authorities to ensure that the road remained open. Flows would then return to the main drainage path through the 1500mm culvert that drains towards Lake Wakatipu well to the south of the proposed swale. This scenario may have been considered in the CFM report (its paragraph 3) that refers to issues developing and being remediated as necessary. The report might have been more specific, in our opinion.

## 4. FLOW CAPACITY OF THE PROPOSED SWALE

The CFM report describes the swale as being grassed, having a 800mm base width, and 2:1 batters. A Mannings n value of 0.35 was assumed. This value is confirmed a appropriate for mid-length vegetation. Our calculations demonstrate that such a swale at the advised slope of 1% will convey 713 l/s at a depth of 400mm. This compares well with the 720 l/s derived by CFM and, quite significantly, allows a freeboard of a further 400mm. If the swale were to flow at bankfull level, we have calculated that the swale capacity would increase to in excess of 3,000 l/s.

This demonstrates to us that the swale capacity as designed is more than adequate, even in the most extreme of super-design situations conceivable.

## 5. CONCLUSIONS

1. The CFM report underestimates the catchment area contributing to the proposed swale.

2. The report adopts a 10-minute design rainfall period. This is considered too short, leading to a significant overestimate of design rainfall intensities.

3. The variances summarised in conclusions 1 and 2 are to a considerable degree mutually compensating.

4. The report also fails to consider any possible impacts of runoff from above the State Highway.

5. Notwithstanding the above, we have independently concluded that the swale as designed is adequate to accommodate a 100 year, 30 minute storm runoff with in excess of 400mm freeboard (this incorporating an allowance for climate change impacts).

6. At bankfull, the swale will accommodate in excess of 4 times the 100 year design flow.

7. *Direct* impacts on the proposed swale of gravel flows in the event of fan instability are not considered possible.

8. *Indirect* impacts on the swale of any flow re-direction caused by gravel flows are possible, although extremely unlikely, but are likely to be short-term. The bankfull capacity of the swale appears adequate to ensure that any resulting temporary flow increase should be dealt with without difficulty.

9. To summarise, it is considered that the proposed swale as described in the CFM report is easily adequate to accommodate any foreseeable contributing flows, even allowing for climate change impacts and gravel fan instability. Ongoing maintenance of the swale is assumed.

**Neil Johnstone BEng, MIPENZ**
**Director**
**Flood Sense Limited**