Porters Ski Area

Wastewater, Stormwater and Water Supply Infrastructural Options Assessment Report

PORTERS

8 July 2010

CPG New Zealand Ltd
A subsidiary of Downer EDI Limited

Porters Ski Area
Wastewater, Stormwater and Water Supply
Infrastructural Options Assessment Report

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Job No.: 303872
Date: July 2010
Ref: 303872-Rpt100709 Infrastructural Options FINAL jd.doc

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EXECUTIVE SUMMARY

Wastewater

A community treatment plant and on-site land treatment was chosen as the one that met the environmental sustainability and development goals of the expanded Porters Ski Fields.

Peak wet weather flow was estimated to be 1,047 m^3/day. After careful consideration of a number of factors relevant to this development, a combination of primary effluent treatment and secondary treatment was selected as the most environmentally acceptable option. The high quality effluent produced would make application to land more sustainable in the long-term than would be the case with other systems.

Primary treatment will be achieved by passing the sewage through a sedimentation/interceptor tank near the source, i.e. at each dwelling or cluster. During this process, solids settle out and the effluent goes through further treatment as it passes through a coarse filter at the tank outlet.

The wastewater in the interceptor tanks is either pumped into the small diameter pipe (Sedimentation Tank Effluent Pumping (STEP)) or if site conditions are suitable, it can be conveyed through the small diameter pipes under gravity (STEG) to the treatment plant.

A recirculating textile packed bed reactor is recommended for secondary treatment because of its ability to handle varying inflows, to provide high quality effluent and its lower operation and maintenance requirements. The wastewater undergoes UV treatment prior to discharge to the ground.

In selecting the dispersal method, factors such as soil type and nature of the soil profile, soil permeability and the quality of the effluent from the treatment plant were considered. The soils were classified as Category 2 soils with a design irrigation rate (DIR) of 35 mm/week or 5 mm/day and the area required for land treatment was estimated to be 21 ha based on peak loading. Assessing nutrient loading, then using an effluent nitrogen concentration of 30 g/m^3, the area that is required is 12 ha.

Stormwater

The stormwater concept aims to provide adequate treatment of stormwater prior to discharge into the receiving environment.

There are three broad types of developed catchments:

- Village Core;
- Residential and Outlying Buildings; and,
- Roading.

Runoff from the southwestern end of the village core, the carpark building and roadway is to be directed via kerb, channel and sump to treatment and discharge in an infiltration basin constructed in general accordance with ARC TP10/NZTA (2009) design criteria. An easily maintained forebay in this basin separated from the infiltration cell by a filtering stone bund that removes coarse sediments and litter from the flow. Secondary flow in excess of the capacity of this system is conveyed over a low point in the basin bund to the Porter River via the entrance road stormwater system. The northeastern end of the village and associated roading is to be directed via kerb channel and sump to treatment and ground discharge via an infiltration basin.

The residential areas and outlying buildings such as the hot pools, gymnasium and the Crystal Hotel with its underground carpark will discharge runoff to ground and they will have raingardens. Raingardens will comprise a 1 m deep trench over variable width and length (dependent upon the area of catchment to be served) into the native silty gravel backfilled with an underdrain bedded in 150 mm of clean AP20 or similar that is overlain by a reworked native silty gravel infiltration layer capped with a local stone armouring and vegetated with local alpine herbfield plants. The raingarden trench will be set in a depression of variable depth as required for live surface storage to enable the required hydraulic performance. The underdrain will be connected to a soakpit to ensure complete drainage.

Road stormwater will drain to a 150 mm deep channel on the inside of the road bedded with crushed gravel (eg AP40) armoured with pitrun or railway ballast. This channel will be established and punctuated every 50 metres (100 m in the Crystal Village and between 20 and 50 m in the Slopeside) by an inverted concrete culvert sump excavated to the underlying silty gravels. An inverted siphon from this infiltration sump will direct overflow (in excess of the permeability of the substrate below each sump) to a drain under the road that will discharge overland to the nearest waterway via downslope ephemeral gulleys. Rip rap armouring will be employed at the head of these gulleys at the outfall of the drains.

Within the village core and in the short section of entrance roadway above the Porter River, a full kerb and channel will be employed instead of the gravel channel. In the latter sections the Porter River will be afforded further protection from potential sediment contamination. Secondary flow will be directed where possible to a final polishing in shallow vegetated depressions.

Modelling employing the Rational Method based Hydrocad package was conducted on the proposed infrastructure using the following assumptions:

- The Mt Cheesman rainfall intensities are a reasonable approximation of conditions at the Porter River valley.
- The internal roading shaded in orange and carparking is to be sealed.
- C roof, concrete and asphalt hardstand = 0.9.
- Design rainfall event for
  - Primary disposal = 5 %AEP 10 min.
  - Secondary flow = 2% AEP 10 min.
  - Waterquality = 1/3 x depth 43% AEP 24 hour (2.33 yr ARI) = 26.4 mm.
- Water Quality Volume = 10 x catchment area x 26.4 mm.
- The adopted rate of subsurface soakage is half the measured rates at location of application.

The proposed stormwater concept will achieve the water quality standards set out in Environment Canterbury's Waimakariri Regional Plan and the Proposed Natural Resources Regional Plan. This includes treatment of contaminants such as suspended solids, hydrocarbons, nutrients such as nitrates and phosphorus.

Water Supply

Using the population profiles modelled by Porters, the peak daily demand is estimated at 942 m^3/day or 11 L/s. The annual potable water requirement estimate is 204,000 m^3.

The New Zealand Fire Service Fire Fighting Water Supplies Code of Practice (SNZ PAS4509:2008) guidelines were adopted in assessing the fire fighting water requirements. It is recommended that all buildings (with the exception of single family dwellings) should have fire sprinklers in order for the development to achieve FW2 status. This would also provide maximum possible fire protection as the ski resort is a considerable distance away from major fire stations. The total storage allowance for fire fighting is 62 m^3.

While an allowance has been made for fire fighting requirements in the storage volumes, it will still be possible for fire fighting equipment to take water directly from the nearby surface waterways for fire fighting purposes as this is permitted under the RMA.

Potable water storage requirement is estimated at 550 m^3. This volume is made up of working storage, emergency storage and fire fighting requirements for both sprinklers and hydrants.

A preliminary reservoir location has been selected. The proposed reservoir location offers a number of benefits, such as reducing the pressure required to supply the individual connection points as gravity head will reduce the pumping pressure required.

Options for reservoir construction material included a circular reinforced concrete reservoir; steel tanks and lined timber tanks. A timber tank is recommended for the site.

The main purpose of the treatment plant on the proposed water scheme will be to provide a protozoal barrier. The type of treatment that can be used for providing a protozoal barrier depends on the quality of the source water. A combination of deep filtration and UV disinfection is recommended. However, bench tests are recommended to be undertaken prior to the detailed design stage to confirm the need for coagulants.

The Sustainability Principles of the Selwyn District Council

The proposed infrastructure has been selected and designed taking into account Selwyn District Council's Five Waters Strategy and where applicable the seven sustainability principles contained therein have been adopted.

As this Infrastructure Options Report is finalised, Selwyn District Council are still in the process of finalising its engineering design code of practice. As the code has not been formally adopted for widespread use, the district council engineers advise that using other available design guides such as Christchurch City Council's Infrastructure Design Standards or NZS4404:2004 is satisfactory.

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1.0 INTRODUCTION

1.1 General

Porters Ski Area Limited (Porters) proposes to redevelop the Porters Ski Area into a resort-style complex with increased skiing capacity, on-mountain facilities and accommodation.

The development will primarily comprise the creation of a village base area in the Porter River Valley adjacent to the existing ski lodge and staff accommodation, and expansion of the existing ski area from the Porters Basin into the neighbouring Crystal Basin.

The proposed expansion will increase the Ski Area into Crystal Valley and will also include associated infrastructure, facilities and accommodation to cater for a mix of people ranging from permanent residents, winter overnighters, winter day visitors, non-winter overnighters and non-winter day visitors. This proposed development will enable Porters to offer activities and receive visitors throughout the year. It is expected that when fully developed, Porters will receive up to 7,600 visitors per day at peak and just over 600,000 visitors per year.

A location map of Porters is attached in **Appendix A** and a Master Plan of the overall Porters development is attached in **Appendix B**.

1.2 Scope of Report

The objectives of this report are to:

- Assess infrastructural design options for servicing the development with sewage, stormwater and water supply; and,
- Recommend and summarise the best feasible options for sewage, stormwater and water supply.

This report is limited to preliminary designs and recommendations to accompany a Plan Change application to the Selwyn District Council (SDC). It is not intended to be a detailed design report or assessment of the environmental effects. The former will be prepared prior to construction and the latter will be prepared and attached to the relevant resource consents applications.

1.3 Selwyn District Council 5 Water's Strategy

In August 2009, Selwyn District Council released its Five Waters Strategy document which had its "vision for the future for Community Water Supplies, Wastewater, Waterraces, Land Drainage and Stormwater".

In assessing the infrastructural options for the Porters Ski Area, CPG has taken into account the initiatives described in the strategic document. These include:

- Governance issues on who manages the water resources infrastructure and SDC's role;
- Using design parameters that are acceptable to SDC to ensure a consistent and cohesive approach to water allocation, design and management;
- Accounting for supply security and water quality;
- Adoption of the seven sustainability principles in assessing different options;
- Incorporation of the effects of Climate Change by using the rainfall data adopted by SDC for stormwater design; and,
- Design the water system taking into account the Drinking Water Standards and The Health (Drinking Water) Amendment Act 2007.

2.0 SITE DESCRIPTION

2.1 Location

The Porters Ski Area is located in the Upper Porter River Valley at the southern end of the Craigieburn Range. It is the closest ski field to Christchurch, being approximately 90 kilometres distance by road to the west. Access to the Ski Area is provided by a gravel road which extends from SH73 (West Coast Highway) generally up the true-right bank of the Porter River.

Appendix A contains a location plan for the site Porters Ski Area.

2.2 Existing Landuses

Currently, Porters Ski Area comprises approximately 206 skiable hectares of the Porters Basin. The ski area operates 5 ski lifts, comprising: 3 T-bars, 1 Platter Lift and 1 Carpet Lift. These extend from the mid-mountain base at an elevation of 1,300 metres to the top of the mountain at 1,900 metres above mean sea level (amsl). Facilities at the mid-mountain base include a day lodge café, public toilets, ticket outlet and ski hire. There are two parking lots as well as staff quarters and a maintenance building.

Both the Porters Ski Club Lodge, which provides limited accommodation (42 beds), and the staff housing lodge (16 beds) are located at the bottom of the mountain at an elevation of 960 metres amsl. These facilities are immediately adjacent to the mountain access road and sit on a terrace above the Porter River.

Porters Ski Area operates throughout the ski season, which typically runs for around 90 to 100 days from the end of June through to early October.

2.3 Topography and Geomorphology

The Porter River Valley is over 1 km wide at the 1,100 m contour (approximately the elevation of Coleridge Pass) with a floor comprised of fans, alluvial terraces and moraines. The valley is only approximately 200 m wide and 75 m deep, and the river channel is less than 5 m wide. Thus the river is topographically well separated from most of the landscape on which development is likely to occur; except for the access road for approximately 1.6 km down valley from the present staff quarters building.

Below the Porter Basin proper is the Porter Stream Valley with a south and north ridge and slopes emanating from the Basin and with a smaller ridge in the central valley.

The Crystal Basin lies approximately 1 km north of the existing ski area and comprises two poorly defined glacial basins. The floor of the basins comprises moderate to locally steep sloping terrain covered by gravel, cobbles and boulders with limited rock outcrops at elevations between approximately 1,500 m and 1,800 m amsl.

2.4 Vegetation Cover

Vegetation in the Porters and Crystal Basins generally varies according to altitude. Vegetation types were characterised by Boffa Miskell as part of detailed ecological site surveys undertaken in 2007 and 2010 and are summarised below.

The upper slopes of the Porters and Crystal Basins comprise loose scree and occasional prominent rock outcrops with little vegetation. Below this area, slopes are more stable and are characterised by a zone of snow totara-tussock grasslands.

At around 1,250 m to 1,300 m amsl, there is a distinct transition from snow totara-tussock to a *Dracophyllum* heath, which is an abundant vegetation type and covers the lower slopes, terraces and some of the Porter River Valley floor as well as the location of the proposed Village Base development.

The tussock/*Dracophyllum* heath is regenerating from a century or more of dry stock grazing as part of a former pastoral lease.

2.5 Climate

The area has a montane-alpine climate (cool and wet), with rainfall across the Craigieburn Ecological District being 1,500 - 2,400 mm per year. The predominant winds are north-westerlies but cold fronts can bring snow at any time of the year.

Snow packs typically develop in late May to early June and thaw between September and mid-January. Frost can occur all year round and frost heave is a feature of the soils.

Meteorological data is recorded at two elevations at Porters Ski Area. The lower elevation site is located just above the base area at 1,323 m amsl (2399859.50E 5769430.99N NZMG) on a flat man-made terrace at the base of a large northeast face and includes a snow study plot and an automatic weather station (AWS). A wind fence around the snow study plot minimises the influence of the wind and provides more accurate snow measurements. The high elevation meteorological site is located on an exposed ridgeline at 1936.8 m amsl (2398298.49E, 5769768.65N NZMG) and consists of a high elevation AWS that was installed in 1996.

The low elevation meteorological site records of rainfall, air temperature, wind and RH data consist of a few winter months for 2005; all winters between 2006 and 2009; and most summer data from 2007/2008. The upper elevation meteorological site data record comprises snow pack temperatures, rainfall normals/intensity, RH data and temperature normals from all winters since 1997 and most summers since 2005/2006.

Unfortunately, the recorded rainfall intensity is too short, so the modelling in this Infrastructure Report employs interpolations of nearby data, as adopted by Selwyn District Council for infrastructure design purposes (Opus 2009). In Table 2.1 below, the rainfall record for the neighbouring Mt Cheeseman is summarised.

Table 2.1: Mt Cheeseman Design Rainfall Table (1990 - 2009) – mm/hr at Various Storm Durations

Return Period	10-min	20-min	30-min	1-hr	2-hrs	6-hrs	12-hrs	24-hrs
2.33	25	21	19	14	11	7.3	5.2	3.3
5	33	27	24	17	13	8.6	6.3	4.0
10	41	33	28	19	14	9.7	7.1	4.5
20	51	40	34	21	16	11	7.8	5.0
50	67	50	41	24	18	12	8.8	5.6
100	82	60	48	27	19	13	9.5	6.1

2.6 Surface Water

The Ski Area is situated in the upper catchment of the Porter River, which flows from Coleridge saddle, to the south of the Ski Area, into Broken River, approximately 6 km downstream of the site, and on to the Waimakariri River.

Numerous tributaries drain into the Porter River with the Porter Stream and Crystal Stream being the largest waterways within the boundaries of the Ski Area.

The Porter Stream has two branches. The true left branch is ephemeral and arises in the gully below the snow grooming garage. The true right (and the main branch) flows below the Ski Area water reservoir and car park areas adjacent to the ski road for some 1.5 km down to the Porter River.

The Crystal Stream originates in Crystal Basin, immediately to the north of the existing ski field and flows through a steep gully before levelling and splitting into multiple branches prior to entering the Porter River.

There are two springs on site. The first feeds the Porter Stream which joins the Porter River as a major tributary just upstream of the Staff lodge. The second spring discharges to a channel that runs down the centre of the main spur to the top of the terrace scarp before dropping steeply to the Porter River. The road to the Porter Ski Club lodge cuts across this channel at the base of the spur. The culvert has blocked and directs most of the small spring flow northwards along the inside of the road to the lodge.

The bulk of incident rainfall soaks into the ground on this site except where human activities have compacted the land surface in the establishment of roading and the buildings. Stormwater drainage discharges at the bridges over the Porter Stream and Porter River.

2.7 Lithology/Geology

The geology of the Porter River basin comprises a basement greywacke that are deep sea sediments (sandstones and mudstones) of Mesozoic Age (245 to 140 million years ago) and are usually referred to as Torlesse ‘greywacke’. Most of the Southern Alps are comprised of this geological formation. Whilst a strong, indurated rock, greywacke is often highly fractured and forms extensive scree deposits of angular gravel. Bedding orientations are variable (typical of Torlesse greywacke) though dips are typically moderately steep (>60°) and into the slope (i.e. to the northwest) in the vicinity of the existing Ski Area and planned areas of development. This is favourable in that it limits the potential for land sliding on bedding.

The greywacke basement strata are overlain by a sequence of Tertiary age (65 to 25 million years ago) volcanogenic sediments, sandstones and limestones in places. They have mostly been eroded from the basement and are only found in the valley floor where exposed by the Porter River and in the limestone quarry adjacent to the access road.

Overlying the basement Torlesse Greywacke are various alluvial, glacial and windblown surficial deposits infilling the valley floors. Further upslope from the valley floors, surficial deposits generally comprise eroded scree and active and rapid soil formation; characteristic of an alpine mountain environment.

The Porter River Valley is situated in an active tectonic zone and forms part of a basin created by thrusting of Torlesse Greywacke over top of the younger tertiary sediments. The Porter River Valley is bounded by the Cheeseman Fault to the north and the Torlesse Fault to the south. These are active faults capable of producing large magnitude earthquakes and consequently represent an area of high seismic hazard.

Appendix C contains logs of site test pits excavated by both mechanical digger and truck mounted auger at a number of points across the development. These soils are classified as fine sandy loams to sandy loams and they have a weak to massive structure.

Infiltration tests were conducted on the:

- Surface soil (a dark organic rich surface soil horizon of between 300 mm and 450 mm in depth) with a double ring infiltrometer; and,
- Subsurface silty gravel at an average 2 m bgl using a flooded basin technique to determine its potential for stormwater discharge below ground level. These latter tests involved the sinking of a 150 mm drainage pipe 0.5 m into the substrate via a 300 mm take point drilled by an auger at locations across the site. Bentonite was used to seal the sides of the drainage pipe. Half the contents of a 5,000 litre water truck were poured down the pipe to saturate the gravel layer and hydrate the Bentonite seal outside the pipe. The following day water was applied to the pipe at a rate that maintained a static head. This rate was taken as the soakage rate of the gravels for the small area at the base of the pipe.

The results of the surface and subsurface testing are contained in Tables 2.2 and 2.3 below.

Table 2.2: Surface Infiltration Results

Site	Topo 50 Grid Reference	Infiltration Rate (mm/hr)
1	BW20 914 073	320
2	BW20 915 074	320
3	BW20 915 077	320
4	BW20 916 075	240

Table 2.3: Pit Soakage Results

Location	Topo 50 Grid Reference	Soakage Rate in L/sec from end of 150 mm pipe (0.0177 m^2)	Derived Unit Soakage Rate (L/sec/m^2)
Porter Chalets	BW20 911 074	3.7	210
Slopeside	BW20 912 075	3.6	200
Village Centre	BW20 914 076	3.5	196
Crystal Chalets	BW20 915 079	4	226
Staff Lodge	BW20 914 075	1.9	110

2.8 Groundwater

Snowmelt and rainfall on the upper slopes of the Porter River Valley percolate into scree to emerge as discrete springs at lower levels such as the Porter Stream and the two spring seepages described in Section 2.6 above. Groundwater discharge appears to be responsible for the majority of the stream flows and this is evidenced in the exposed pylon foundation that is situated behind the current Ski Area service area. This pylon foundation is some 2 m deep and has been shored up by a concrete pipe gallery approximately 1 m in diameter. Flows at the base of this pylon can be up to 30 L/s. However, no groundwater or evidence of a seasonal watertable was encountered in the test pitting further down slope and this is consistent with the elevation of the surface excavation as compared to the elevation of the discharged stream water.

3.0 WASTEWATER

3.1 Design Principles

Porters' objective is to make the proposed development as environmentally sustainable as possible and CPG's brief was to take this into account in assessing the suitability of various service options. In considering the various wastewater design parameters, CPG has sought to ensure that this objective is met by selecting a wastewater system that performs to the highest performance standard in terms of level and consistency of treatment under intermittent loading, varying temperature conditions and general reliability.

3.2 Visitation Projections

One of the key objectives for this development is to offer year-round recreational and sporting facilities at Porters. Currently it only offers snow based recreation and this restricts visitation to the winter skiing season. Annual skier visits have varied over the years, averaging out to approximately 26,385 over the past sixteen seasons. The highest recorded skier visits were in 1996 with 35,271 people.

The proposal seeks to increase the visitations to 604,000 people a year. However, the visitations will be spread throughout the year rather than be limited to just three or four months as is currently the case. Table 3.1 below shows the projected monthly visitor numbers.

Table 3.1 Projected Monthly Visitor Numbers to Porters

Month	Permanent	Winter Overnighters	Winter – Day Visitors	Non-Winter – Overnighters	Non-Winter – Day Visitors	Total Visitors
Jan	6,200	0	0	20,000	20,000	46,200
Feb	5,600	0	0	16,200	16,200	38,000
Mar	6,200	0	0	10,050	10,050	26,300
Apr	6,000	0	0	9,300	9,300	24,600
May	6,200	0	0	8,050	8,050	22,300
June	6,000	26,000	23,400	4,000	4,000	63,400
July	6,200	66,400	67,300	0	0	139,900
Aug	6,200	58,400	57,500	0	0	122,100
Sept	6,000	14,000	12,900	0	0	32,900
Oct	6,200	2,400	2,200	4,400	4,400	19,600
Nov	6,000	0	0	8,400	8,400	22,800
Dec	6,200	0	0	19,850	19,850	45,900
Total	**73,000**	**167,200**	**163,300**	**100,250**	**100,250**	**604,000**

3.3 Design Assumptions and Flows

The population data presented in Table 3.1 was used to estimate the wastewater flows that are likely to be generated from the expanded Porters development. Table 3.2 below summarises the assumptions that were used to generate the wastewater flows.

Table 3.2 – Assumptions Used in Flow Calculations

Visitor Type	Flow (L/p/d)	Reference
Permanent Visitors	250	NZS4404:2004 Land Development Design Guide
Overnighters	180	AS/NZS1547:2000
Day Visitors	45	AS/NZS1547:2000

Table 3.3 below gives the daily and annual wastewater flows expected to be produced from Porters when it is fully developed.

Table 3.3: Projected Wastewater Flows and Volumes

Period	Peak Day Wastewater Flow (m^3/d)	Wastewater Flow (m^3/year)
Peak Dry (occurs in Winter)	698	78,251[a]
Peak Wet Weather	1,047[b]	-

[a] Based on actual accumulated annual daily flows for the population; and,
[b] Peak Wet Weather = 1.5 x Peak Dry Weather.

The peak daily dry weather flow is the largest single day flow from a composite of the flows for the different visitor types. Peak wet weather flow is one and a half (1.5) times the peak dry weather flows. In open sewage systems, the peak wet weather factor is normally 2. At Porters, the system will be completely piped and sealed and therefore no infiltration and inflows from other sources are expected. To be conservative and to account for the long term deterioration of the pipe joints a conservative peak wet weather factor of 1.5 has been adopted. Therefore, it is anticipated that the proposed expansion of Porters will discharge 1,047 m^3/day.

3.4 Wastewater Treatment and Dispersal Options

3.4.1 Wastewater Quality

There are no industrial discharges at the development. However, the commercial premises proposed are likely to have stronger characteristics than typical domestic strength. The expected quality of wastewater generated and the effluent produced after at-source sedimentation is shown in Table 3.4(a) below (extracted from Metcalfe & Eddy (2003)), and Table 3.4(b) (from Butler & Smith (2003)). The commercial wastewater characteristics are likely to be towards the upper end of the ranges shown in the Table 3.4(a).

Table 3.4(a): Wastewater and Sedimentation Tank Effluent Characteristics (Metcalf and Eddy, 2003)

Parameter	Unit	Raw Wastewater [b]	Mean after rtPBR [a]	Mean after MBR [a]
BOD_5	g m^{-3}	< 450	<5 – 15	<5
TSS	g m^{-3}	< 350	< 15	< 5
Nitrate-N	g m^{-3}	< 80	1 – 25	
Total-N	g m^{-3}	40 – 100	5 – 25	10
Total-P	g m^{-3}	10 – 30	12	
DRP	g m^{-3}	-	8	
FC	cfu 100 mL^{-1}	10^6 - 10^{10}	< 10^4	< 1
FC following UV			<500	

a – from Manufacturer's literature.
b – from Rotorua trials (Schools, 2006)

Table 3.4(b): Wastewater and Sedimentation Tank Effluent Characteristics (Butler D. and Smith S., 2003)

Characteristic (mg/l)	Source	
	Crude/Raw	***Settled***
BOD	300	175
COD	700	400
TOC	200	90
SS	400	200
NH_4 - N	40	40
NO_3 - N	<1	<1

The objective of any treatment system chosen is to produce effluent of such good quality that when discharged it will have no adverse effects on the environment. Below we discuss the options that were assessed and the reasons for recommending the preferred treatment systems.

3.4.2 Options for Dealing with Domestic Wastewater

A number of wastewater treatment and dispersal options are available in New Zealand. These systems typically consist of treatment, storage, and effluent disposal to provide a complete solution. Below are some of the ways that domestic wastewater is dealt with:

- Discharging to a Council's sewer;
- Individual on-site treatment and dispersal at each lot; and,
- Community on-site treatment plant and central dispersal.

Each of these options, and their appropriateness and efficiency at Porters, are discussed below.

Discharging to Council Sewer

As there is no Council sewer within an economic distance (less than 10 km is usually achievable and possibly economically) at Porters, this option is not discussed any further.

Individual On-site Treatment at Each Lot

In this option, on-site wastewater treatment and dispersal is installed at each lot and operated and maintained by the individual home owners. The key considerations associated with this are:

- Availability of sufficient land area within each lot for wastewater to be dispersed to land effectively;
- Regulatory separation distances between boundaries and the discharge area may limit the number of lots that could be developed;
- Regulatory separation distances to waterways would also affect the number of lots that could be developed as the discharge to land needs to be at least 20 m to any waterway or take point; and,
- Individuals would be responsible for the ongoing operation and maintenance of the treatment plants.

Community On-site Treatment Plant

Wastewater from each lot would flow to one (or more) clustered community treatment plant(s). After treatment, the wastewater is then dispersed via land treatment or discharged to water.

Community on-site land treatment systems generally require large areas of land to ensure appropriate hydraulic, biochemical and nutrient loading rates.

The individual on-site wastewater treatment and dispersal was not considered suitable at Porters given some of the key features of this method and the density of the development in the village area. The community onsite treatment plant and dispersal to land method was chosen as the one that met the environmental sustainability and development goals of the expanded Porters Ski Area.

3.4.3 Wastewater Treatment Systems

A number of treatment systems were considered for the community on-site option. These included the following:

- Recirculating Textile Packed Bed Reactors (rtPBR);
- Membrane Bioreactor (MBR);
- Submerged Aerated System (SAF);
- Trickling Filters; and,
- Sequence Batch Reactors (SBRs).

After careful consideration of a number of factors relevant to this development (e.g. visitor types, seasonal distribution, temperature range, operator input requirements, sludge production) and to the site physical features, CPG considered a combination of at-source primary effluent treatment and community secondary treatment to be the most environmentally sustainable option. The high quality effluent produced means that discharge to land does not produce the range of effects that are associated with alternative treatment systems.

At-source Primary Treatment

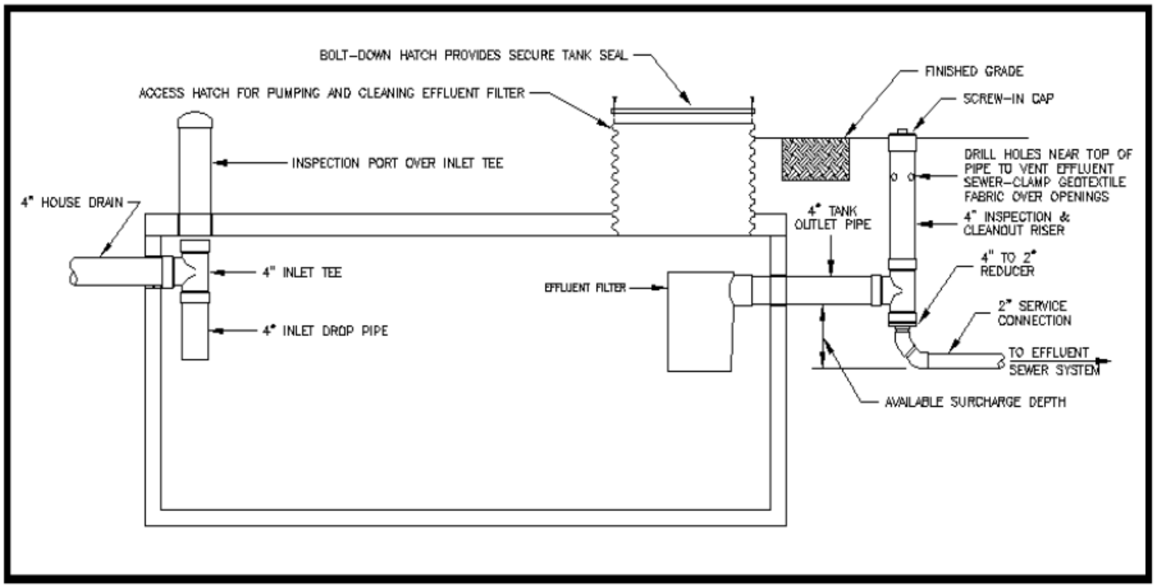
Primary treatment will be achieved by passing the sewage into a sedimentation tank near the source. During this process, solids will settle out and the effluent goes through further treatment as it goes through a filtered pump vault or filter prior to gravity flow. The sedimentation tank removes grease, suspended solids and other contaminants. The settled/sedimentation columns in Table 3.4 show the level of reduction of contaminants and nutrients after sedimentation.

The sedimentation tank can capture approximately 90% of the grease commonly found in domestic wastewater. Amino acids and proteins are broken down and the effluent is generally anaerobically stabilised ensuring a more uniform quality in the effluent conveyed to the secondary treatment plant and a low level of sludge accumulation.

Typical sedimentation tanks have a capacity of at least 4.5 m^3 for individual dwellings. This equates to at least 24 hours of storage depending on the size of the dwelling(s) being served by the tank. Larger sizes can be specified for longer storage or larger dweliings/hotels, etc. Figure 3.1 shows a typical layout of a primary treatment system.

As a guide, sludge in the sedimentation tanks builds-up at a rate of 0.08 m^3/person/year and will need to be pumped out on average every 10 -15 years.

Figure 3.1: Typical Primary Treatment System



Conveyance Options

As discussed above, a three stage (primary, secondary and soil/plant) treatment process is recommended for Porters. At-source sedimentation tanks collect the wastewater and provide treatment as discussed above. The effluent is then conveyed to the modular secondary treatment plant.

The use of pumps and gravity pressure allows small diameter pipes to be used to convey the water to the secondary treatment plant. The effluent from the sedimentation tanks may be pumped into the small diameter pipes (Sedimentation Tank Effluent Pumping (STEP)) or if specific site conditions are suitable, then gravity provides the required pressure as the wastewater is conveyed through the small diameter pipes under gravity (STEG).

Other benefits of the STEP/STEG systems are:

- They can be installed on a range of grades as pipes can follow natural ground rather than a set grade;
- Watertightness prevents infiltration and other inflows and this reduces capital and operating costs;
- No requirement for manholes to allow access to sewers; and,
- Small pumps can be used as the flows are less due to the watertightness; and
- The pumps require less maintenance than grinder or macerating pumps.

Individual pressure mains from the STEP systems will feed into sub-mains running parallel to the internal roads. The sub-mains feed into the main conveyance pipeline taking the effluent from the development to treatment plant and dispersal area.

Crossing Porter Stream

Wastewater generated from properties on the Slopeside and Porters Chalets will be conveyed by pipe across Porter Stream. It is proposed to cross Porter Stream at the existing culvert on the road crossing.

Crossing Crystal Stream
A number of options for taking the conveyance mainline across Crystal Stream were considered. These included:

- Running a pipe directly across Crystal Stream;
- Running the mainline pipe along the access road, crossing Porter River at the culvert, past the bus parking area and then crossing Porter River again directly opposite the discharge area; and,
- Running the pipe up the Crystal Stream valley and across Crystal Stream generally following a contour.

After a careful consideration of the pros and cons of each option, the proposed option is to run the mainline across Crystal Stream on a contour as shown in **Appendix D**. Some of the factors considered in selecting this option were:

- Crossing the Crystal Stream offers the most direct route compared to following the access road. It also means that the pipeline only crosses a waterway at one location and not two as would be the case if the access road route is used; and,
- Following the contour has more benefits than taking the more direct route across Crystal Stream. These include (i) avoidance of a low point under the bed of the river where effluent solids could potentially accumulate and, (ii) the degree of installation difficulty that a direct (cross-contour) crossing would cause.

Pipe Installation Options
The pipe could either be thrust or trenched across the bed of Crystal Stream. The pipe could also be incorporated into an Ecological Weir. This weir is a suggestion by the project Ecologists to protect upstream native fish from introduced predatory species, but at the time of preparing this report this has not been confirmed as part of the proposal. With the Porter Stream crossing, this will be a pipeline incorporated into the road crossing above the culvert.

During the installation of the pipelines stream, if trenching is used flow will be temporarily blocked (less than 8 hours at a time). The clean water will be separated from any sediment laden water to ensure that there is no effect on any fish populations. Ideally, this work will be undertaken during periods of low flows in both streams

The effluent conveyance submains and mainline pipe will be polyethylene pipes. These are cost efficient and provide flexibility in design, ease of laying and longevity. They can be manufactured and supplied in long rolls (18 - 200 m lengths depending on the pipe diameter and pressure rating) that minimise the number of joints and hence the possibility of inflow/infiltration when the pipe starts to age. The pipe diameters and design pressures will be established at the detailed design stage. However, it is envisaged that the submains will be in the order of 50 – 100 mm diameter.

Based on the 1:50,000 topographic maps, there appears to be sufficient head for the effluent to be conveyed to the treatment plant without a separate booster pump. Detailed topographic surveys will be undertaken prior to the detailed design phase to confirm the elevations. If a booster pump is required, this can be installed within the village area and will comprise a wet well and two pumps; in a duty:standby configuration.

The development concept at Porters also proposes a café and service facilities in the Crystal Basin. The population to be served by these facilities is included in the population figures used to estimate the amount of wastewater generated. Effluent from this area will be conveyed through a pressurised small diameter pipe to the Village where it will join into the submains. The conveyance pipeline will run within the service tracks that will be developed for the construction of the snowmaking storage reservoir.

Effluent from the existing service area up at the Mid-Mountain Base within the Porters area will also be conveyed to the village and into the sub-mains as shown in **Appendix D**.

Secondary Treatment

Of the systems listed above, the two leading secondary treatment systems were the rtPBR and MBR. These two treatment systems are well established in New Zealand, give high quality effluent and function well under fluctuating loads. These systems are commonly used for community on-site wastewater where a high level of treatment, nitrogen reduction and the removal of pathogens are important considerations.

The expected effluent discharge quality from these two systems is summarised in Table 3.5 below.

Table 3.5: Expected Final Effluent Quality

Parameter	Unit	Raw Wastewater Value[b]	Value after rtPBR System [a]	Value after MBR [a]
BOD_5	g m^{-3}	< 450	<5 – 15	<5
TSS	g m^{-3}	< 350	< 15	< 5
Nitrate-N	g m^{-3}	< 80	1 – 25	
Total-N	g m^{-3}	40 – 100	5 – 25	10
Total-P	g m^{-3}	10 – 30	12	
DRP	g m^{-3}	-	8	
FC	cfu 100 mL^{-1}	10^6 - 10^{10}	< 10^4	< 1
FC following UV			<500	< 500

a – from Manufacturer's literature.
b – from Rotorua trials (Scholes, 2006)

Re-circulating textile packed bed reactors consist of two main components - a re-circulation tank and a packed textile bed. Effluent, having been treated in a primary settling tank (at-source primary treatment) enters the re-circulation tank and mixes with already treated and oxygenated wastewater. Discharge from the re-circulation tank is pumped into the packed bed reactor. Effluent is sequentially applied to the top of the reactor. Percolate is collected in an under drain system. Suspended solids are removed by mechanical straining and biological films, which form on the textile. Intermittent application of wastewater and venting of under drains helps maintain anaerobic conditions followed by aerobic conditions throughout the system. This encourages BOD_5 reduction and nitrification/denitrification of nitrogen. The effluent from the under drain is collected and part is re-circulated back to the textile packed bed reactor. The recycle ratio is usually 3–4 recycled back to 1 treated wastewater to dispersal (irrigation) at design flow loadings. During average flows, re-circulation will increase, providing increased nitrogen removal.

A MBR system is a combination of the activated sludge process (a wastewater treatment process characterised by a suspended growth of biomass) with a micro or ultra-filtration system that rejects particles. MBRs have two basic configurations: (1) an integrated configuration that uses membranes immersed in the bioreactor, and (2) a recirculating configuration where the mixed liquor circulates through a membrane module situated outside the bioreactor. The membrane filtration system replaces the traditional gravity sedimentation unit (clarifier) in the activated sludge process. In addition to removing biodegradable organics, suspended solids, and inorganic nutrients (such as nitrogen and phosphorus), MBRs retain particulate and slow growing organisms, thereby allowing treatment of more slowly biodegradable organics. They also remove a very high percentage of pathogens (protozoa, bacteria, and viruses), which alleviates the need for additional disinfection processes.

The MBR generally produces better quality effluent than the rtPBR as shown in Table 3.5. However, MBRs produce a larger volume of sludge, have higher maintenance requirements and operating costs than rtPBRs. The rtPBRs are able to handle varying inflows, compared to the MBRs, whilst providing high quality effluent using simple systems that require low operation and maintenance requirements. The rtPBR treatment system is modular and can be installed incrementally as development progresses.

Other positive benefits of the rtPBR are (please note that some of them do apply to the MBR):

- Not highly temperature dependent;
- Remote system servicing and troubleshooting;
- Low odour production;
- Low noise;
- Low sludge production;
- Low power requirements;
- Low visual impact as the system will be buried below the ground;
- Good for anaerobic pre-treated settled wastewater;
- Suitable, after disinfecting, for a water reuse system; and,
- Simple to operate with minimal operator knowledge required.

In addition to the above the rtPBR has features that enable shock variable loads to be managed:

- During periods of low flows, effluent from the recirculation tank continues to be dosed on to the media bed within a closed loop system i.e. 100% recirculation rate occurs when incoming flow is zero. This maintains the population of beneficial microorganisms; and,
- Sudden large flows are not dosed onto the media immediately. They are intercepted in interceptor tanks and the recirculation tanks that release a fixed amount of water irrespective of influent flow rates. These tanks provide a buffering function. The dosing rate onto the media is controlled by timers which control the pumps in the system which means a set volume of wastewater is applied to the media over a 24 hour period.

For these reasons, CPG considers the rtPBR to be more suitable for Porters Ski Area. An example of an installed rtPBR is shown in **Appendix E**. In addition, there are two major suppliers of this technology in NZ, with one of these locally based.

Treatment Plant Footprint

The approximate footprint of the treatment plant is 50 m x 50 m for a system sized for 1,047 m^3/day and with 24 hours emergency storage. The treatment plant may be located anywhere within the treatment area (**Appendix D**).

The treatment plant will be operated from a central location. Usually it is housed in an enclosed control-shed which can range in size from 9 m^2 and it may be up to 2.5 m high. The proposed re-vegetation in the LTA will take years to fully conceal the control house, as a result Porters propose to plant partly grown vegetation and trees around the treatment plant controls shed. Alternatively, the control shed can be incorporated into an underground concrete tank, however, this makes access more difficult.

3.4.4 Land Application System

Discharge Location

The potential discharge area or the land treatment area (LTA) has been identified and is shown in **Appendix F**. URS undertook a study (**Appendix G**) to ascertain the actual area that could be utilised for land application. This took into account the slope stabilities, waterways and rock outcrops.

The gross area available for land treatment ranges from 24 - 31 ha (refer to URS Drawing PS087-WW001, **Appendix G**) depending on set back distances of 5 - 45 m from waterways.

Land Application Method

In the preliminary assessment phase, CPG considered the suitability of a number of discharge to land options including:

- Discharge to land via infiltration trenches (LPED-T);
- Discharge to land via infiltration beds (LPED-B);
- Discharge to land via mounds (LPED-M);
- Discharge to land via evaporation assisted beds (LPED-ETA);
- Discharge to land via surface irrigation methods (LT-S); and,
- Discharge to land via drip irrigation (LT-SS).

In selecting the application method, factors such as soil type and soil profile, soil permeability and the quality of the effluent from the treatment plant were considered.

Table 4.2B2 in the ANS1547:2000 summarises common site and soil constraints and provide guidance on the suitability of land application systems. Table 3.6 below is a modified extract from Table 4.2B2.

Table 3.6 Summary of Common Site and Soil Constraints (modified extract of Table 4.2B2 ASN1547:2000)

Land Application System	Slope (%)	Soil Depth (m)	Soil Category Number	Comments – Suitability of System at Porters
Infiltration Trenches	<25%	> 1.2	5 - 6	While the system would meet most of the design criteria, it has a high environmental risk to pollute surface run-off.
Infiltration beds and evaporation beds	< 5%	> 1.2	4 - 6	A large proportion of the land delineated for land treatment has slopes greater than 6% the maximum recommended for beds. Therefore not suitable at Porters.
Mounds	< 15%	Not Important	1 - 3	Has high risk of toe seepage of steep slopes. Slopes in parts of LTA > 15%.
Surface irrigation systems	< 6%	> 0.4	Any	Some slopes in the LTA are > 6%.
Subsurface drip irrigation	< 25%	> 0.4	Any	Operates under most conditions and meets the conditions at Porters best.

The soils in the proposed LTA were hand-augured and logged. The soils in the LTA were classified as fine sandy loams to sandy loams, weakly to massive structured. A more detailed description of the soils is given in Section 2.7. Soil Infiltration tests were carried out on the proposed land dispersal area and these are presented in Tables 2.2 and 2.3. The average infiltration rate across the three sites tested was over 300 mm/h indicating that the soils were well drained to rapidly drained. The soils in the proposed LTA were therefore on average classified as Category 2 soils.

From the foregoing and from Table 3.6 above, the subsurface drip irrigation system is the most suitable system to use at Porters as it meets the site and soil constraints of the LTA.

Discharge Area Required

Hydraulic Loading Rate

Table 4.2A4 of the A/NZS1547:2000 gives the recommended Design Irrigation Rate (DIR) for different soil categories. Category 2 soils have a DIR of 35 mm/week or 5 mm/day. Therefore, the subsurface drip irrigation will be designed to apply 5 mm/day.

The peak daily and annual flows from the Ski and Village Base Areas when fully developed have been estimated in Section 3.3 above. Table 3.7 below shows the net LTA estimates for peak dry weather and peak wet weather conditions.

Table 3.7: LTA Estimates under Dry and Wet Weather Conditions

	Condition	
Parameter	**Peak Dry Weather**	**Peak Wet Weather**
Flows (m^3/d)	698	1,047
Annual Wastewater Volume (m^3/year)	78,251	117,376
Peak DIR (mm/day)	5.0	5.0
Net LTA Required (ha)	**14**	**21**

Therefore 21 ha will be required to meet the peak daily discharge at Porters Ski Area.

Nutrient Loading Rate

Table 3.5 gave the total nitrogen concentration from a rtRBR as given by the manufacturers. We have adopted a more conservative figure of 30 g/m³ from the Rotorua trials (Scholes, 2006) as it is difficult to remove nitrogen in colder periods. Based on a nitrogen concentration of 30 g/m^3 and based on an average annual volume, the area that will be required to keep the nitrogen loading below a generally accepted value of 200 kg/ha/yr is 12 ha.

As discussed above, the gross area available is 24 – 31 ha, with a further 18.3 likely to become available. Therefore, there is enough suitable land area to be used for wastewater treatment and dispersal. The dispersal to land system will be designed to irrigate 21 ha (net) based on the hydraulic loading constraints. This will result in a nitrogen loading of 114 kg/ha/yr.

Irrigation System

As discussed above, drip irrigation will be used to disperse the treated effluent to land. It is proposed that subsurface Netafim pressure compensated Bioline AS dripline be used (a product specification sheet of the type of system described is included in **Appendix H**).

Dripper irrigation is relatively easy to install and irrigation lines can be easily configured to the land contours, making use of unusual shaped land areas and sloping sites. Low application rates (from 1.6 L/h to 2.2 L/hr) allow for good final treatment of effluent within the soil matrix.

As the dripline is pressure compensated, it minimises the effect of steep slopes thus giving more flexibility to the layout of the dripline. The emitters are pressure compensated to 35 m and each line must be fully pressurised before discharge can occur from any emitter.

The irrigation system will be designed to be self-draining (i.e. without anti-drain CNLs) to ensure that is free of effluent between application doses. This will act as a frost protection measure. In addition, as the vegetation canopy and litter layer develops over the drip lines with time, this will also provide additional frost protection

The dripline will be spaced 1 m apart and the drippers will be 0.6 m apart. The dripline will be placed approximately 200 mm below the ground surface to protect public health and to minimise risk of frost damage to the irrigation system. This treatment method is well proven in cold climates, for example, a similar system was installed at Jacks Point in Queenstown in 2004 and this system has been operational ever since and is performing as expected.

Irrigated Crop/Vegetation

The dispersal area will be under Mountain Beech and Red Tussock. It is envisaged that vegetation growth will improve and provide a healthy and natural vegetation cover while providing a nutrient removal function from the discharged wastewater.

3.4.5 Summary of Preferred Treatment Systems

The alternatives that were not selected were either technically inappropriate or unnecessarily expensive.

The proposed wastewater treatment and dispersal system will consist of the following:

- Primary treatment of raw wastewater via individual or shared at-source sedimentation tanks and a coarse filter system;
- Secondary treatment of wastewater via a re-circulating textile packed bed reactor treatment plant;
- UV treatment is proposed to reduce the faecal coliform numbers; and,
- Land treatment via sub-surface drip irrigation at a maximum application rate of 5 mm/day.

The preferred conveyance options are:

- A pipeline across Porter Stream at the existing culvert above the bed within the road; and,
- A pipeline will follow the contour from the development across Crystal Stream to the treatment plant. The pipeline will either be trenched across Crystal Stream or thrust under the river bed. This may be incorporated into an ecological native fish protection weir.

The preferred treatment and dispersal system will provide:

- High quality treated wastewater;
- High degree of storage during outages;
- Reasonably consistent effluent quality throughout the year;
- Low annual operating costs;
- High degree of automation;
- A sustainable discharge system;
- Reuse of the effluent for irrigation;
- Likely to meet the expectations of future regulatory changes;
- Low maintenance requirements; and,
- Ease of operation.

3.5 Resource Consents

The preferred options involve the discharge of treated wastewater to land at a scale above that of permitted activities in the Waimakariri Regional Plan, the Transitional Regional Plan and the Proposed Natural Resources Regional Plan, therefore discharge consents (air and contaminants to land) from ECan will be required. A resource consent will also be required for works associated with installing the conveyance mainline across Porter Stream and Crystal Stream. These resource consent applications for the proposed discharges to land and air and associated stream crossings are part of this application

3.6 Operation and Maintenance

For the system to operate successfully, appropriate operation and maintenance requirements will need to be adhered to. It is envisaged that a suite of resource consent conditions will stipulate the basic regulatory requirements. Other necessary performance requirements will be in-built into the design and installation of the system and the preparation of an Operation and Maintenance Manual. Examples of the performance requirements are:

(i) Measures to reduce freezing of the wastewater pipes and infrastructure in winter. To mitigate against this a number of measures will be implemented in the design and installation of the system, examples include:

- Installing conveyance pipes below the seasonal frost line;
- Pipe insulation where necessary;
- Electric heat trace where necessary;
- Water circulation; and,
- Layout to allow lines to drain between dosing.

(ii) Automation of pumping units and level controls in storage facilities for safety reasons.
(iii) A remote alarm system will be installed, to warn of any problems with water levels or failure in the treatment plant.
(iv) There will be at least 24 hours emergency capacity (combined at sedimentation tanks and the treatment plant) as a contingency plan should power supply fail or the system break down.
(v) A back up power supply source such a generator will be installed. This will supply power in case of normal power supply interruptions.
(vi) The development and use of an Operations and Maintenance Plan (the supplier of the system will provide this) for the proposed wastewater treatment system. This will address the operation of the treatment plant and will detail procedures for maintaining the process unit and will involve a management contract with the suppliers of the treatment technology or suitably trained contractors or maintenance staff. Typical operation and maintenance requirements for a rtPBR treatment system are likely to include:

- Two monthly visual inspections of the plant and monitoring and recording processes stated on relevant consent conditions.

- Individual sedimentation (STEP/STEG) tanks require annual scum level measurements and checks of pump alarms and floats with de-sludging and cleaning of filter occurring as needed.

- Inspection, cleaning, and operation of components within the recirculation tanks are required two monthly.

- The rtPBR requires two monthly lateral operating pressure checks and annual flushing and flow rate checks.

- Treated effluent tanks require two monthly checks of operating systems such as pumps, alarms, floats and control panel. Sludge levels can be checked annually and sludge removed as necessary.

- The land treatment areas should be inspected every two months with irrigation lines flushed annually.

4.0 STORMWATER

4.1 Design Objectives

The overall stormwater objectives are:

- To minimise the pollution of receiving waterways by contaminants in road stormwater;
- To prevent erosion of the slopes where discharges are directed; and,
- To attenuate peak flows, where necessary, from additional runoff derived from the increased impervious area post development.

4.2 Developed Site

The resort development has been divided into three broad types of developed catchments as shown Plan 1 in **Appendix I**. These developed catchments are:

- Village Core;
- Residential and Outlying Buildings; and,
- Roading.

4.2.1 Village Core

As shown in Plan 2 of **Appendix I**, runoff from the area shaded in yellow comprising the southwestern end of the village core, the carpark building and roadway is to be directed via kerb, channel and sump to treatment and discharge in an infiltration basin constructed in general accordance with ARC TP10/NZTA(2009) design criteria.

An easily maintained forebay in this basin separated from the infiltration cell by a filtering stone bund that removes coarse sediments and litter from the flow.

- A splitter box on the inlet directing above the basin volume to a soakpit.
- A bed comprising 150 mm layer of native soil established in local alpine herb field vegetation caps a 1 metre depth of 100 mm Porter quarry stone or railway ballast placed over the native silty gravel.

Secondary flow in excess of the capacity of this system is conveyed over a low point in the basin bund to the Porter River via the entrance road stormwater system.

The area shaded in pink comprising the northeastern end of the village and associated roading is to be directed via kerb channel and sump to treatment and ground discharge via an infiltration basin located in the triangular area east of buildings 14 and 9. This basin will be constructed to the same general design as that receiving the runoff from the southwestern end of the village core. A concept design is depicted by Plan 3 of **Appendix I**. Secondary flow will be directed east to the Porter River via the general flow path indicated.

A vegetated cutoff swale drain will cross the toe of the slope immediately above the village core for discharge to the spring outflow channel. Plan 4 of **Appendix I** shows the outfall of this swale will feature a surge chamber (vertical 1 m diameter concrete culvert) sunk into the permeable gravels with a submerged outlet to prevent scour damage.

4.2.2 Residential and Outlying Buildings

The residential areas and outlying buildings such as the hot pools, gymnasium (Building C) and the Crystal Hotel with its underground carpark will discharge runoff to ground according to the generic conceptual design contained in Plan 5 of **Appendix I** which features:

- A raintank for each roof with soakpits constructed in general accordance with the design shown in Plan 6 of **Appendix I** and proportional to the area of each roof and the measured subsurface soakage rate at that location; and
- Hardstand runoff being directed to an adjacent infiltration basin or raingarden.

Raingardens will comprise a 1 m deep trench over a of variable width and length (dependent upon the area of catchment to be served) into the native silty gravel backfilled with an underdrain bedded in 150 mm of clean AP20 or similar that is overlain by a reworked native silty gravel infiltration layer capped with a local stone armouring and vegetated with local alpine herb field plants. The raingarden trench will be set in a depression of variable depth as required for live surface storage to enable the required hydraulic performance. The underdrain will be connected to a soakpit to ensure complete drainage.

As shown in Plan 1, secondary flow from the above will be directed to the stormwater system in the adjacent road system or directly to the nearest site waterway via downslope ephemeral channels.

4.2.3 Roading

Plan 7 of **Appendix I** is a generic conceptual design for the roading stormwater system subcatchment that is shaded in orange. Within these areas the road will drain to a 150 mm deep channel on the inside of the road bedded with crushed gravel (e.g. AP40) armoured with pitrun or railway ballast. As shown on Plan 1 this channel will be established and punctuated every 50 metres (100 m in the Crystal Village and between 20 m and 50 m in the Slopeside) by an inverted concrete culvert sump excavated to the underlying silty gravels. An inverted siphon from this infiltration sump will direct overflow (in excess of the permeability of the substrate below each sump) to a drain under the road that will discharge overland to the nearest waterway via downslope ephemeral gulleys. Rip rap armouring will be employed at the head of these gulleys at the outfall of the drains.

Within the village core and in the short section of entrance roadway above the Porter River, a full kerb and channel will be employed instead of the gravel channel. In the latter sections the Porter River will be afforded further protection from potential sediment contamination. Secondary flow will be directed where possible to a final polishing in shallow vegetated depressions to be created at the locations shown in Plan 1 of **Appendix I**.

4.2.4 Proposed Service Facilities On-Mountain

Stormwater from any future developments on the mountains will be discharged to ground via soakpits. The stormwater will be primarily from roofs and from hardstanding around the service areas. It should be noted that there will be no parking and regular vehicle access and therefore contaminants such as hydrocarbons will be negligible.

4.3 Quality of Stormwater Discharge

4.3.1 Types of Contaminants

The runoff from the roading and carparking of this development is likely to contain the following contaminants:

- **Suspended Solids**

  This is derived from site erosion, the detritus of decomposing vegetation and road surfaces, especially car parking areas that accumulate the by-products from vehicle wear and tear and combustion. This is easily removed from sumps and the beds of infiltration basins and raingardens.

- **Oxygen Demanding Substances**

  Oxygen demanding substances are comprised of soil organic matter and plant detritus that reduce the oxygen content of water when they are broken down by chemical action and bacteria. This is not normally a soil contaminant.

- **Pathogens**

  Pathogens are disease causing bacteria and viruses in urban stormwater, usually derived from animal faeces and endemic sources such as soil. Stormwater pathogens are mostly consumed by other organisms within the upper layers of the soil and are therefore not regarded as a significant contaminant.

- **Litter**

  Litter in stormwater is often referred to as gross pollution. It has a high visual and amenity impact, but only limited effect on public health and ecological standards. It will easily removed from the proposed stormwater infrastructure and it is not a significant soil contaminant.

- **Hydrocarbons and Oils**

  These are typically associated with vehicle use (e.g. oil leaks) although spills of hydrocarbon products from other sources can occur. They may be in the form of a free slick, oil droplets, and oil emulsion and in solution or absorbed to sediments. These contaminants are usually readily visible and thus can be easily removed from sumps, and the beds of the infiltration basins as part of a maintenance programme.

- **Other Toxic Organics**

  A variety of other toxic organic substances are occasionally present in stormwater. These typically originate from garden or agricultural chemicals such as weed killers or pesticides. No application of garden or agricultural chemicals is contemplated within this site.

- **Toxic Metals**

  A variety of trace metal compounds are carried by 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). Metals are predominantly in the solid state either sump as part of their proposed maintenance and from the beds of infiltration basins and raingardens as is shown to be necessary by programmed monitoring of soil quality there.

- **Nutrients**

  As animals are prohibited to be introduced to the site, this is not anticipated to be a significant contaminant type.

Table 4.1 shows a possible range of selected site annual contaminant loadings derived from the results of surveys of a variety of urban catchments by Williamson (1992).

Table 4.1 - Predicted Annual Site Contaminant Loadings

Contaminant	TSS	TP	TN	BOD	Zn	Cu	Hydro-
Urban Loadings kg/ha							
Lower	60	0.4	2.5	12	0.25	0.02	4
Mean	375	0.8	8	20	0.75	0.09	
Upper	1750	1.6	11	33	2	0.2	20
Rural Loadings kg/ha							
Lower	103	0.01	1.2		0.02	0.02	
Mean							
Upper	583	0.25	7.1		0.17	0.04	

*(*Figure derived from CCC (2003))*

The risk to soil from hardstand runoff in residential areas is considered to be low due to the intermittency of residence (low levels of vehicle movement and parking) especially during the summer. The runoff from roofs (zincalume prohibited) may contain small amounts of suspended sediment and bird droppings but is considered to be relatively uncontaminated.

The runoff from carparks and roading is likely to contain a higher content of road borne contaminants because traffic volumes and parking duration there will be relatively higher than in the areas of private residences. The soil contamination risk in these latter areas is appropriately mitigated by the operation and maintenance of the proposed stormwater management strategy, in particular the replaceable filter beds of the infiltration basins and raingardens.

4.4 Sizing of Infrastructure

Because so much of the resort layout and thus catchment areas are a flexible concept, the design sizing calculation shown in Table 4.2 was carried out to provide the minimum dimensions for a hypothetical unit (100 m^2 catchment) infiltration basin and raingarden that will enable engineering plan compliance with the design criteria. The Unit sizing can be pro rata increased for the size of catchment where such technologies are proposed to be applied. They can also be incorporated into the consent conditions. The formulae will be especially useful for the developers of private residences and lodges as it will allow flexibility in area of roof and hardstand if sufficient stormwater management capacity is provided.

Table 4.2: Calculations for a "Unit" Infiltration Basin Raingarden and Soakpit

INFILTRATION BASIN for 100 m^2 unit area	**Quantity**	**Units**
Surface area requirement (As)		
A_{eff}	0.010	ha
d_{ff} (1/3 of 2yr 24 hr rainfall depth)	26.4	mm
WQV=10*Aeff*dff	3	m^3
f_d Measured infiltration rate/2	0.16	m/hr
i hydraulic gradient	1	m/m

INFILTRATION BASIN for 100 m^2 unit area	Quantity	Units
t discharge time	6	hrs
p water quality precipitation depth	0.0264	m
Unit Surface Area As = WQV /(fd*i*t) - p)	3	m^2
Calculate trench depth to provide storage for 37% of volume required to be infiltrated		
Vr= void space ratio of Porters quarry stone	0.35	
Trench Volume Vt = 0.37(WQV+pAs)/Vr	3	m^3
dmax = fd*(t/Vf)	3	m
Rock filled Trench depth below bed d = Vt/As	1	m
Design is sufficient where d<dmax		
RAINGARDEN for 100 m^2 unit area		
Aeff	0.01	ha
dff (1/3 of 2yr 24 hr rainfall depth)	26.4	mm
WQV=10*Aeff*dff	3	m^3
Live storage requd = V live = 0.4*WQV	1	m^3
Assuming		
drg (depth of soil filtration medium)	0.85	m
k (1/2 measured infiltration rate)	3.84	m/day
h (operating depth) 1/2 max water depth of 300 mm	0.15	mm
trg (residence time)	1.5	days
Required Raingarden Unit surface area Arg = (Vwq)(drg)/k(h+drg)trg	0.390	m^2
Compare with Available Raingarden Area to see if sufficient		

Table 4.3 shows the required and proposed size of infiltration basins and raingardens for the village core and commercial areas which are reasonably settled in their catchment area.

Table 4.3: Ground Discharge Infrastructure – Required and Proposed Dimensions

(a) Infiltration Basins

Catchment and Area	Water Quality Volume (m^3)	Total Live Storage (m^3)	Basin Bed Area (m^2)	Trench Depth (m)	Prop. Basin Storage (m^3)	Basin Depth (m)
Northeast Village Core (0.89 ha)	275	299	252	1	889	1
Southwest Village Core, (1.8665 ha)	235	255	528	1	376	1

(b) Raingardens

Catchment	Water Quality Volume (m^3)	Raingarden Bed Area (m^2)		Total Live Storage (m^3)	
		Required	Proposed	Required	Proposed
Crystal Hotel (0.139 ha)	37	5	70	15	40
Gym (0.0475 ha)	12	5	40	2	23
Spa carpark (0.05 ha)	13	5	50	2	29

(c) Roof and Road Soakpits

Soakage Test pit Area	Ridge Top	Slopeside	Village Core	Crystal Resid. and Hotel	Staff Lodge (Lower Village Core)
Adopted Local Soakage rate (L/s/m^2)	105	100	98	113	55
Roof Peak Discharge from 100 m^2 for 10%AEP 10 min rainfall (L/s)	1.03				
Hydraulic Capacity 300 mm Auger Hole (L/sec)	7.4	7.1	6.9	8.0	3.9
Road Peak Discharge from 600 m^2 for 5% AEP 10 min rainfall (L/s)	7.7				
Hydraulic Capacity Vertical 1050 mm Concrete Pipe (L/s)	91	87	85	98	48

It is intended that soakpits for other hardstand areas are likely to be excavated pits sized according to catchment size and location.

4.5 Construction Period Discharges

As the site is located within a sensitive alpine environment, extra care will need to be taken during the development phase to minimise the area susceptible to erosion. The stormwater erosion and sediment control management plan (ESCMP) for the site contained in **Appendix I** of this document has been prepared according to the following key principles of Environment Canterbury Erosion and Sediment Control Guidelines (2007):

- Control run-on water.
- Separate 'clean' water from 'dirty' water.
- Protect the land surface from erosion.
- Prevent sediment from leaving the site.

The ESCMP covers best management practice measures, such as revegetating or covering areas that will be exposed for extended durations, and the use of sediment traps around the perimeter of work areas to aid in preventing any sediment loaded runoff from exiting the site.

4.6 Modelling of Discharges

The stormwater infrastructure has been preliminarily designed based on the following assumptions:

- The Mt Cheesman rainfall intensities are a reasonable approximation of conditions at the Porter River valley.
- The internal roading shaded in orange and carparking is to be sealed.
- C roof, concrete and asphalt hardstand = 0.9.
- Design rainfall event for:
  - Primary disposal = 5 %AEP 10 min.
  - Secondary flow = 2% AEP 10 min.
  - Waterquality = 1/3 x depth 43% AEP 24 hour (2.33 yr ARI) = 26.4 mm.

- Water Quality Volume = 10 x catchment area x 26.4 mm.
- Surface infiltration = ½ measured rate = 160 mm/hr.
- The adopted rate of subsurface soakage is half the measured rates of application (Section 2.7).

The stormwater system concepts have been designed based on ARCTP10 criteria. Without exception they show that at half the measured rates of surface infiltration and deep soakage as applied, discharge to ground is a technically feasible solution for stormwater treatment and disposal on this site. Table 4.4 and Table 4.5 summarise the dimensions of the proposed infrastructure for the treatment and disposal of the stormwater through the infiltration basin and raingardens.

Table 4.4: Infiltration Basins and Raingardens - Hydraulic Performance

Basin / Raingarden and Catchment Area	10 min Incoming Discharge (L/s)	10 min Peak Level (m)	Critical Duration (min)	Critical Duration Incoming Discharge (L/s)	Critical Duration Peak Level (m)
Northeast Village Core Infiltration Basin (0.89 ha)	111	0.151	444	22	0.767
Southwest Village Core, Infiltration Basin (1.8665 ha)	234	0.134	444	46	0.681
Crystal Hotel (0.139 ha)	17.4	0.084	204	4.6	0.172
Gym (0.0475 ha)	6.0	0.042	30	4.2	0.072
Spa carpark (0.05 ha)	6.3	0.032	30	4.3	0.055

Table 4.5: Roading SW Channel for a 6 m wide carriageway 1 in 10 grade

Design Channel Depth (mm)	Design channel Width (m)	Channel Batter	Road Grade	Design Length of Road between Sumps (m)	Effective Catchment Area (m^2)	5 %AEP10 min Peak Discharge (L/s)	Mannings Number *	Hydraulic Capacity (L/s)
180	1	5	10	100	600	8	0.035	288
							0.04	252

- *Table 22.1 from CCC (2003) for an open channel with a rock bed.*

The hydraulic loss capacity of the base of the 1,050 mm concrete pipe sump set into the native soil assuming a conservative 100L/sec/m^2 subsurface permeability (see Table 2) is 87 L/s.

Without exception the results show that at half the measured rates of surface infiltration and deep soakage as applied, all the proposals for discharge to ground will meet the treatment and hydraulic design criteria.

4.7 Summary

The stormwater infrastructure has been designed based on the best practices. The system will be able to cope with stormwater from the developed sites and appropriate secondary flow paths have been provided.

The proposed system is also consistent with Selwyn District Council's Five Water Strategy.

5.0 POTABLE WATER SUPPLY

5.1 Introduction

The following sections describe the option assessments undertaken for supplying potable water to the proposed Porters development. It includes the following:

- A description of the existing water supply on site;
- An estimate of the water demand by the fully developed Ski Area and associated services;
- A description of the water supply options;
- An estimate of the reservoir storage required;
- A brief description of the relevance of the Drinking Water Standards and the Health (Drinking Water) Amendment Bill to the water supply for this development; and,
- A description of the treatment options and a recommendation of the preferred option.

A brief description of the snow-making and its water requirements and storage is also provided.

5.2 Existing Water Supply Situation

5.2.1 Existing Resource Consents

The Ski Area has several existing consents. These consents are summarised in Table 5.1 below.

Table 5.1: Existing Water Related Resource Consents at Porters

Consent Number	Purpose	Flow Rate (L/s)	Daily Volume (m^3/day)	Expiry Date
CRC992163	Take surface water for domestic use and to prevent pipes from freezing.	1.4 L/s	121	14/05/2034
CRC991129	To dam and take surface water for snowmaking purposes.	19.6 L/s	1,693	09/02/2034
CRC991131	Discharge excess water from a storage pond to an unnamed tributary of the Porter River.	< 19.6 L/s	-	09/02/2034
CRC991132	To discharge contaminants to land and water for snowmaking purposes.	< 19.6 L/s	< 1,693	09/02/2034
CRC930705	Take water from an unnamed tributary of the Porter River for the generation of electricity.	-	-	22/09/2028
CRC930706	To discharge water into an unnamed tributary of the Porter River, following the generation of electricity.	< 20 L/s	< 1,728	30/06/2028

5.2.2 Existing Water Supply Infrastructure

There are three existing potable water supply systems within the Porters Ski Area. The first is a domestic take that supplies the skifield café and service facilities. Water for this take is sourced from a spring on the slopes just above the café area. The water is piped to a small storage take from where it is reticulated to the points of use. This take does not have a consent as it was probably a permitted use under the Transitional Regional Plan (<100 m^3/day).

The second potable water scheme takes water from Crystal Stream and supplies the Porter Ski Club clubhouse. This take is consented under CRC992163 to take 1.4 L/s or 121 m^3/day. The scheme conveys water using a small diameter pipeline which runs to the clubhouse. This take runs continuously to a small tank at the clubhouse, with the overflow soaking away below the clubhouse.

A third take is from a subsurface gallery near the upper learner's platter tower. This is a take for snow-making (Consent CRC991129). The water flows via gravity to a 20,000 m^3 storage reservoir below the current day facilities.

5.2.3 Water Quality

Water samples were collected from two points on the Porter River, as well as single points on Crystal Stream and Porter Stream. The locations of the four sites that were sampled are shown in the map provided in **Appendix J**. The results of the water quality results from the four sites are also given in **Appendix J**. The sample results show that the water quality is good with very low levels of suspended solids, hydrocarbons, nutrients and other contaminants. A discussion of the suitability of the water for drinking purposes is provided in Sections 5.8 and 5.10 below.

It should be pointed out that the existing water sources are not treated and water is taken directly from source to point of use and no health issues associated with drinking this water have been reported.

5.3 Water Demand Estimates

5.3.1 Snow-making and Storage

Water requirements for snow-making were estimated by Ski Industries Ltd. The water requirements and the design were based on putting 30 cm of snow on all the trails in 65 hours to open the Ski Area by early June each year and putting another 30 cm on all trails by end of June, and finally another 30 cm July/August to allow for freshening trails, melt and evaporation.

From these figures, the approximate total volume of water required each year is 180 thousand cubic metres. To meet these requirements a minimum take rate of 20 L/s is required during the period from May 15th to end of August. The reservoir size recommended by Ski Industries Ltd is 90,000 m^3. However, a higher flowrate with the potential for a smaller reservoir is desirable, hence the take application of up to 40 L/s for snow-making will be made to the Regional Council.

A reservoir location has been identified at around 1,720 m elevation between Trails 9, 11 and 12. This position will allow trails below elevation 1,500 m to be gravity fed and is a very energy efficient solution.

Appendix K shows the location of the reservoir and the reticulation from the pumping point to the reservoir.

5.3.2 Potable Water Demand Assumptions

The population data (Table 3.1) supplied by Porters was used to estimate the potable water demands. Table 5.2 summarises the assumptions that were made to determine the water use at Porters.

Table 5.2: Potable Water Demand Assumptions

Visitor Type		Unit	Source of Assumption
Summer Overnighters	250	L/p/day	NZS4404:2004
Winter Overnighters	200	L/p/day	Assumed 80% of NZS4404:2004
Permanent Residents	250	L/p/day	NZS4404:2004
All Day Visitors	50	L/p/day	Existing Porters water use data

In addition to normal individual water demands and fire-fighting requirements, water will also be required for watering gardens (minimal as will mainly be natural tussocks, etc.), cleaning public areas (e.g. hotels, bars etc) and in swimming and spa pools. Water use assumptions used for estimating these other requirements are summarised in Table 5.3 below.

Table 5.3: Water Use Assumptions for Services

Use	Allowance	Unit	Source of Assumption
Hotels/Apartments	2	L/m^2/day	CCC Estimates
Retail and Commercial	2	L/m^2/day	CCC Estimates
Pool/Spa Complex	5	L/m^2/day	Proposed design baseline
Lawns and Gardens - based on 10% of GFA	5	L/m^2/day	5 mm/day ET for Canterbury

- *Figures based on Gross Floor Area (GFA)*

5.3.3 Fire-fighting

Although the New Zealand Fire Service Fire Fighting Water Supplies Code of Practice (SNZ PAS4509:2008) is a discretionary standard, it is generally accepted as being a *de facto* obligation. It is intended for the proposed Porters development to meet the fire fighting standards.

According to the SNZ PAS4509:2008, "all structures with a sprinkler system installed to an approved Standard" will have fire water classification number FW2. Single family homes without sprinklers are classified FW2. Therefore, if all structures, except single family homes, are installed with sprinklers the fire water requirements for the development could be based on FW2 which requires reticulated water supply systems to have one hydrant no more than 135 m away from a building and an additional hydrant within 270 m of that same building, with both hydrants providing at least 25 L/s, or for non-reticulated water supplies a storage volume of 45 m^3 would be required.

Based on the foregoing, it is recommended that an allowance for fire fighting storage of 45 m^3 for use from the fire hydrants be made. Since it is recommended that buildings be sprinkled (with the exception of single family dwellings), it is also estimated that a storage allowance of 17 m^3 for sprinklers be made. The allowance for sprinklers is based on the following assumptions:

- Average sprinkler coverage of 9 m^2 (NZS4541:2007).
- Gross Flow Area of 27,420 m^2 as specified.
- Average flow rate of a sprinkler = 1,350 L/min (NZS4541:2007).
- Sprinkler running time = 30 minutes (NZFS PAS 4309:2008 for FW2).
- 50% of the sprinklers would operate at any one time. This is conservative as it assumes an unlikely scenario where half of the GFA would be under fire.

Therefore the total storage allowance for fire fighting is 62 m^3 (45 m^3 + 17 m^3).

Section 14 of the Resource Management Act states that: *no person may take, use, dam or divert any water unless the taking, use or damming is either: --*

(a) *For reasonable domestic and stock-water needs, with no adverse effects on the environment, or*
(b) *For fire fighting purposes where water is either taken or used (does not apply to damming or diverting); or*
(c) *It is expressly allowed by a rule in a regional plan or in any relevant proposed regional plan or a resource consent has been granted.*

Thus while an allowance has been made for fire fighting requirements in the storage volume, it will still be possible to take water directly from the nearby surface waterways for fire fighting purposes as this is permitted under the RMA. However, as the area is remote from the nearest fire station, it is recommended that buildings be installed with sprinklers and the recommended storage provided.

5.3.4 Summary of Future Demands

Table 5.4 below summarises the design water requirements for the expanded Porters development.

Table 5.4: Water Demand Summary

User/Use			DEMAND					
	Maximum Daily No. of People	**Demand**	**ADD**		**PDD**		**PHD (5 x PDD)**	
		L/p/day	m^3/day	L/s	m^3/day	L/s	m^3/hour	L/s
	Winter period (120 days)							
Winter Overnighters	2600	200	279	3.2	520	6.0	108.3	30.1
Permanent Residents	200	250	16	0.2	50	0.6	10.4	2.9
Winter Day visitors	4800	50	68	0.8	240	2.8	50.0	13.9
Retail/Services			5	0.1	15	0.2	3.2	0.9
Apartments/Hotels			31	0.4	93	1.1	19.4	5.4
Pool/Gym/Spa			3	0.0	10	0.1	2.1	0.6
Winter Totals	**7,600**		**402**	**5**	**928**	**11**	**193**	**54**
	Non-Winter period (245 days)							
Non-winter overnighters	750	200	102	1.2	150	1.7	31.3	8.7
Permanent Residents	200	250	34	0.3	50	0.6	10.4	2.9
Non-winter day visitors	750	50	20	0.2	38	0.4	7.8	2.2
Retail/Services			10	0.2	15	0.2	3.2	0.9
Apartments/Hotels			63	1.1	93	1.1	19.4	5.4
Pool/Gym/Spa			7	0.1	10	0.1	2.1	0.6
Gardens and Lawns			5	0.2	14	0.2	0.6	0.2
Summer Totals	**1,700**		**240**	**3**	**370**	**4**	**75**	**21**

- *Gardens will be irrigated predominantly in summer. It is assumed that there will not be significant lawns as part of residential landscaping.*
- *ADD – average daily demand, based on average of all annual daily requirements*
- *PDD – peak daily demand, based on full population at the design rates*
- *PHD – peak hourly demand, based on the 24 hour demand occurring over 5 hours.*

Table 5.5 below summarises the winter, non-winter and annual volume of potable water used based on the assumptions used to estimate the water demand in Table 5.4 above.

Table 5.5: Potable Water Annual Volumes

	Annual Volume Based on Average Daily Demand (m^3)	**Annual based on Peak (m^3)**
Winter (120 days)	63,487	113,082
Non-Winter (245 days)	76,396	90,613
Annual	**139,883**	**203,695**

The rates of take and volumes on the existing consents are not enough to meet the water demands at Porters Ski Area when it is fully developed. Porters is in the process of applying for resource consents to increase both the rates of take and the annual volumes so as to meet the new water requirements.

5.4 Reservoir Storage

5.4.1 Storage Volumes

There is no New Zealand standard for the amount of water storage to be provided for a new development. The following section rationalises the storage capacity that should be provided for Porters Ski Area.

Treated water reservoir storage is required for the following reasons:

- To provide operating storage to cover the peak hourly demand periods during the day; operating storage is determined from the volume difference between the water entering the reservoir and water leaving the reservoir;
- To provide emergency storage in case of water supply system failures upstream of the reservoir, e.g. water treatment plant failure, power loss or a burst trunk main;
- To provide fire-fighting reserves of water; and,
- To provide adequate chlorine contact time (if chlorine is used) before the water enters the reticulation. This criterion is, however, met easily as the other criteria noted above provide more than adequate storage to meet the contact time requirements.

Ideally, 24 hours water storage should be provided in the reservoir. However, 24 hours is viewed as an unrealistic target by many water suppliers and they aim for lower volumes. Selwyn District Council is in the process of developing its engineering standards. The following assumptions were made in sizing the proposed potable water reservoir at Porters:

(i) 4 hrs x PDD (working) + 8 hrs x PDD (emergency) + Fire (hydrants) + Fire (sprinklers)

PDD = Peak Daily Demand;
Fire = Fire storage for hydrant supply and for sprinklers. Refer to Section 5.3.3 above.

Table 5.6 below summarises the potable storage requirements for the expanded Porters development.

Table 5.6: Potable Water Storage Requirements

	Working Storage (4*PDD) m^3	Emergency Storage (8*PDD) m^3	Fire Suppression m^3	Fire Sprinklers m^3	Total Storage m^3
Total Storage Volume	157.1	314.1	45.0	17.0	533.2

We propose that the potable water storage reservoir should have a capacity of 550 m^3.

5.5 Proposed Abstraction Rates and Take Locations

To meet the water demands estimated above, Porters will submit resource consent applications for a new consent for potable water and to increase the rate of take of water for snowmaking.

5.5.1 Ecological Requirements

In addition to the water quality data the CPG sampled, flow data was also collected at locations shown in **Appendix J**. Boffa Miskell carried out ecological studies on the Porter Stream to ascertain the ecological value of the stream. Using the flow measurements supplied by CPG, Boffa Miskell estimated the minimum flow needed to be maintained within the stream to sustain the ecological values. Table 5.7 shows the flow measurements carried out by CPG, the residual flows required as estimated by Boffa Miskell and the available flow after accounting for the ecological requirements. For example, during the months June-August at least 30 L/s should be left in the stream below the point marked "Ecological Flow" shown in **Appendix L**.

Table 5.7: Ecological Flows on Porter Stream

Month	Flow Measurements by[a] CPG (L/s)	Residual Flow Estimated by Boffa Miskell (L/s)[b]	Available Flow (L/s)
January	220	100	120
February	220	100	120
March	220	100	120
April	220	100	120
May	600	60	540
June	240	30	210
July	120	30	90
August	400	30	370
September	220	60	160
October	240	100	140
November	250	100	150
December	220	100	120

a – Location shown in Appendix J
b – "Ecological Flow" shown in Appendix L

5.5.2 Points of Take

Two points of take were identified and these are the existing dam/weir ("the lower point") from where the snowmaking consent is consented to take water and the second point ("the upper point") is at the pylon foundation next to the existing "Beginners' Area". These two points are marked on the plan appended in **Appendix L**.

As discussed in Section 5.3.1 it is proposed to take up to 40 L/s for snowmaking. Table 5.4 shows that the peak daily demand for potable water is 11 L/s, however to reduce the size of the potable water reservoir, it is proposed to take drinking water at 30 L/s. Therefore a total 70 L/s will be required.

The “upper point” would be the preferred point of take for both snowmaking and potable use because:

- It is at a higher elevation than (> 240 m above) the “lower point” and therefore would require smaller pumps to pump to the snowmaking storage reservoir;
- Water would be conveyed to the potable water storage under gravity;
- The water comes straight out of the ground and is therefore less exposed to contamination than the surface water flow at the dam. Therefore the treatment requirements of this water supply would be less.

However, the flow from the “upper point” has been estimated at only 25 L/s. Part of the reason for the estimated low flow of only 25 L/s is that only a small area of the potential groundwater flow path is being tapped. All site indications and hydrogeological thinking lead to the conclusion that if a larger area is exposed by installing a gallery that runs further up (20 m) and across (30 m) the slope, in a T-shape, the take point would yield more than 100 L/s (70 L/s for the two water uses and 30 L/s for the ecological requirements in the months June – August for example). The gallery would be designed and constructed to capture as much of the available flow as possible up to the 70 L/s required for the two uses.

However, until the gallery is constructed the maximum available flow at different times of the year will not be known. Therefore, it is proposed to consent the “lower point” so that the required water (for both snowmaking and potable use) can be taken from either points of take depending on availability.

To ensure that the minimum ecological flows are maintained, flow metering will be required downstream of each take point. The flow meters and the pumps should be linked and automated so that as soon as the flow downstream of the take point reaches the flows shown in Table 5.7 the pump above shuts down. In the case of the “upper take” point, the flow meter could be located at the “Ecological Flow” point identified by Boffa Miskell (**Appendix L**).

In the meantime, CPG recommends that flow measurements on Porter Stream be continued on a monthly basis to collect as much data as possible.

5.5.3 Water Balancing

As discussed in the previous section, 40 L/s is required for snowmaking and 30 L/s for potable water supply.

The 40 L/s for snowmaking is only required during three months of the skiing season. Therefore, for the rest of the year, the only abstraction will be for potable water supply.

Table 5.4 shows that 11 L/s and 4 L/s are required for potable water for the winter and non-winter months respectively. The proposed potable water reservoir is 550 m^3. If potable water is abstracted at the proposed rate of 30 L/s, the reservoir will fill from empty in just over 5 hours. Most of the time the reservoir will not be filled from empty and so may only take 2-5 hours to fill.

There is, therefore, enough flexibility in the system to balance between the snowmaking requirements and the potable water requirements. For example, if flows are low and only enough to meet the needs of one use or the other, the first 2 – 5 hours of the day could be used to fill the potable water reservoir, with the remaining 19 – 22 hours used to fill the snowmaking reservoir. This adds an extra 3 – 5 days to the time required to fill the snowmaking reservoir. The scenario also represents the worst case scenario as it assumes that snowmaking will be required and peak water use during that period.

5.6 Storage Location

Appendix L shows the proposed location of the storage tank. The location does offer a number of advantages and these include:

- Reducing the pressure required to supply the individual connection points as gravity will reduce the pumping pressure required;
- Provision of some degree of cover so that the storage reservoir is not highly visible; and,
- Keeping the distance between the take point and the reservoir to a minimum while achieving the two goals above.

Appendix L shows the envisaged pipe layouts from the point of take on Porter Stream to the reservoir.

5.7 Water Supply for On-Mountain Facilities

It is planned that when fully developed the Crystal Basin Ski Area will have a day lodge, including a café and associated facilities on the mountain.

The water supply for the Day Lodge will be drawn from the pipe supplying water to the storage reservoir for snow-making. A pipe will be run from near the reservoir to the Day Lodge and water will be conveyed by gravity. The Day Lodge is likely to be sited approximately 200 – 300 m below the storage reservoir. Therefore there will be a need to install regulators to control the pressure. The pipe will also be sized appropriately.

Appendix L also shows the envisaged pipeline from the water sources to the proposed service facilities. The routes shown are only approximate and will be finalised when detailed topographical surveys are completed.

5.7.1 Reservoir Material

Options for reservoir construction material include:

- Timber tanks are available and have a significantly lower capital costs;
- Circular reinforced concrete reservoir; and,
- Steel tanks which are also available but do not offer the life span of the reinforced concrete construction.

A timber reservoir is recommended for the site as it is cost effective and would be easier to install than the other available options. Figure 5.1 shows an example of a locally manufactured timber tank. However, Selwyn District Council suggested that a concrete tank would the preferred option if the system was to be vested with council. Porter Ski Area Limited has expressed willingness to adopt Selwyn District Council's preferred option of a concrete tank.

Figure 5.1 – Timber Reservoir (taken from www.timbertanks.co.nz)

5.8 Freezing Protection

Freezing of the water supply infrastructure will be an issue in winter. To mitigate against this requires a combination of strategies; examples include:

- Installing water pipes below the seasonal frost line;
- Pipe insulation;
- Electric heat trace;
- Water circulation; and
- 'Bleeding water to waste' as is currently employed with Porters Ski Club water supply.

These methods will be effective in dealing with potential freezing of the water supply infrastructure.

5.9 Drinking Water Standards

5.9.1 Compliance Criteria

The Drinking Water Standards for New Zealand 2005 (Revised 2008) (or DWS) outline five compliance requirements that must be met to comply with the standard. They are:

Bacterial Compliance: The easiest way to comply with this criterion is to include chlorination as part of the treatment process and to ensure that a contact time of at least 30 minutes is provided. Sampling of the water reticulation is also required on a regular basis to ensure that a chlorine residual remains in the reticulation system. It would also be possible to comply with this criteria through the use of UV or ozone treatment without any chlorine but with additional *E-coli* sampling. The Ministry of Health do however favour the use of chlorine and if a chlorine residual was not provided, this would be reflected in the grading provided for the water system. There are many examples of supplies that prefer not to use chlorine and are willing to accept a slightly lower grading for this reason.

Cyanotoxin Compliance: Cyanobacteria are aquatic organisms that are not a health hazard in themselves and are found in many waterways. However, when they exist in large numbers or blooms (e.g. blue-green algae) the toxins they produce (cyanotoxins) can be a health hazard. There have been various cases reported in the media in recent years describing the death of dogs that have drunk from water containing high levels of cyanotoxins. It is not expected that Cyanotoxin compliance will be difficult to achieve at this site.

Chemical Compliance: There are a large number of chemicals listed in the Drinking Water Standards with maximum acceptable limits. Testing for all of these chemicals is generally not undertaken due to the cost associated with the laboratory analysis. Some contaminants have been tested for and these have been presented above (Section 5.2.3). The results showed no chemicals of concern.

The preferred point of take for the potable water is the "upper take point". The water quality is expected to be better than that from the sampled sites (**Appendix J**) and will have no turbidity issue. As a precaution, water from this take point will only need UV treatment or chlorination.

Radiological Compliance: Naturally occurring radionuclides occur from the leaching of rocks and from depositions from the atmosphere. It is expected that compliance with radiological limits will be achieved for the proposed Porters water supply given the remote location of the site and the geology of the area.

Protozoal Compliance: In accordance with the DWS, a protozoa barrier must be provided in the treatment system. This is to protect the water supply from protozoa such as G*iardia* and *Cryptosporidium*. The main function of any treatment will be to provide this protozoal barrier.

5.9.2 SDC's Sustainability Principles

The Drinking Water Standards for New Zealand 2005 (Revised 2008) (or DWS) outline five compliance requirements that must be met to comply with the standard. They are:

5.10 Health (Drinking Water) Amendment Act 2007

5.10.1 Introduction

Previously there was no legislative requirement to comply with the DWS; however in October 2007 the Health (Drinking Water) Amendment Act was passed which made future compliance with the DWS mandatory.

Below are some of the methods available to achieve compliance with the DWS where the water supplies do not meet the standard.

5.10.2 Ultraviolet Disinfection

Ultraviolet (UV) disinfection has been used overseas and in New Zealand as an alternative to chlorination. Until recently, however, it was only considered suitable for bacteriological and viral disinfection. More recently, studies have shown its effectiveness in inactivating the protozoa *Cryptosporidium* and *Giardia* assuming water turbidities are kept below 1 NTU. UV disinfection is now widely accepted as being effective.

UV disinfection does not leave a protective disinfection residual in the reticulation system, and therefore, residual chlorination is still encouraged to provide this additional protection.

UV disinfection alone is a suitable method of providing a protozoal barrier if the turbidity is kept below 1 NTU. If the source water quality is occasionally above 1 NTU then some form of filtration will need to be used in conjunction with the UV disinfection.

It is recommended that regular turbidity tests are undertaken from the proposed "lower point" take to determine if the water meets the turbidity criteria for UV disinfection. The "upper take" point is not expected to have turbidity issues as groundwater will be abstracted.

UV disinfection without any other form of filtration would be a suitable method of providing treatment for Porters. This is especially true for the upper take point.

5.10.3 Depth Filtration and UV Disinfection

This option consists of a media pressure filter followed by UV Disinfection. The purpose of the filtration stage is to reduce the turbidity of the water down to an acceptable standard so that the UV disinfection can deactivate the protozoa. The most suitable media in this case would be to use the proprietary media called Kinetico Macrolite. This type of media is a more effective filtration media than sand and is likely to effectively filter the water without additional coagulant chemicals.

The filters need occasional (e.g. daily) backwashing. This is achieved by forcing water into the filter from the bottom to stir up the media and carry away the dirt. Sometimes air is pumped through the bed to assist with backwashing. This cleaning process can be instigated automatically and only takes a few minutes and the filter can be used again within a short time. Waste from the backwashing process would be discharged to ground or taken off site. The backwash will potentially consist primarily of suspended solids and discharging these to ground will have no effect on the ecology or water quality.

5.10.4 Membrane Filtration

Membrane filtration consists of very fine filtration down to 0.1 - 0.01 microns. This type of treatment provides a barrier to the protozoa as the protozoa are too large to pass through the filter. The filter itself consists of bundles of hollow strands, similar in size to spaghetti, in which the water flows from the inside out or the outside in, depending on the manufacturer. The fibres are cleaned periodically by reversing the flows with filtered water. It is unlikely that any coagulant chemicals would be required for this treatment system however this would need to be confirmed through on-site trials. A large building would be required for the installation of the membrane plant.

With adequate monitoring, membrane filtration systems can meet the current and likely future requirements of the New Zealand Drinking Water Standards.

5.10.5 Chlorination

As stated above the use of chlorine as part of the treatment system is not essential. It is encouraged by the Ministry of Health and provides a further barrier against contamination. In addition, if there is no chlorination additional monitoring for *E.coli* would be required.

5.10.6 Proposed Water Treatment

If the “upper take” point is the exclusive source of potable water, then a UV treatment and/or chlorine would be adequate as the water is already “clean” and would meet the DWS and the Act.

Porters is also applying for a second take point as a fall back measure in case the preferred location will not yield the desired flows. If this second take point has to be used, the water will require more treatment than the “upper take” point. For this second take point it is recommended that the use of a combination of deep filtration and UV disinfection, described in Section 5.10.3 above, is the best option for water treatment. Bench tests at the “lower take” point are recommended to be undertaken prior to the detailed design stage to confirm the need for coagulants. Based on the water sampling results that have been carried out to date, it is unlikely coagulants will be necessary.

It was recommended earlier that further flow measurements be undertaken between now and when the detailed design commence. At that stage the possible yield from the "upper take" point may be known and if it is enough to meet the water demands, then the final decision on the treatment water treatment system, between the options discussed above, will be made then.

6.0 INFRASTRUCTURE MANAGEMENT OPTIONS

6.1 Introduction

Porter Ski Area Limited has considered a number of options for operating and managing the proposed infrastructure. The main options are:

- Vesting the infrastructure with Selwyn District Council;
- Porter Ski Area Limited operating and managing the systems; or,
- Engaging a private operator to operate and manage the infrastructure.

6.2 Vesting in Selwyn District Council

A number of meetings have been held with the Selwyn District Council. The most recent meeting was held with the Council's asset management team on the 6th of July 2010.

The issue of ownership and management of the infrastructure was discussed. While no formal decision was reached, the council advised that they could take ownership of the infrastructure provided it met all council standards in terms of design, whole life costs analysis and performance after construction.

Council staff also pointed out that based on their experience, the size of the development, occupancy, distance from other service centres the rating bill would be on the high side and therefore they thought that long-term it may be more cost effective for the owners to operate and maintain the infrastructure without vesting to SDC.

At the end of the meeting it was agreed to leave the vesting option open at this stage.

6.3 Operation and Management by Porter Ski Area Limited

Under this option, Porter Ski Area Limited would operate and manage the wastewater, water supply and stormwater infrastructure.

In the short to medium term, the developer will manage and operate the infrastructure to ensure that it operates as specified by manufactures and that it complies with any consent conditions. As the project uptake increases, the developer may then decide to pursue one of the two other options (Sections 6.2 and 6.4) for the long term operation and maintenance of the infrastructure.

6.4 Private Operator

A private operator could be engaged to operate and manage the infrastructure. This is becoming more common in New Zealand. Responsible Management Entities (RMEs) such as Utilities Management (NZ) Ltd are contracted by either Porter Ski Area Limited or the body corporate to operate, maintain, and manage the infrastructure on a day to day basis. RMEs effectively take on the role of the council with the primary difference being that they do not own the infrastructure and are under contract for a specific period.

Therefore the RME model could be used (i) in the short to medium term prior to handing over to Council for long term operation and maintenance; or (ii) throughout the life of the infrastructure if it is not vested with the Council.

7.0 CONCLUSIONS

The preceding sections have described in detail the proposed concepts for wastewater, stormwater and water supply for the expanded Porters Ski Area.

This report has detailed the concepts based on the proposed Masterplan. The details will be finalised at the detailed design stage. The proposed infrastructure is technically feasible and can be designed, installed and operated to meet the high environmental standards envisaged by Porters with cost effective operation and maintenance regimes.

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