REPORT

District of Peachland

Water Treatment Plant and Reservoir Siting Options

0655.0147.01 / April 14, 2010

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1.0 INTRODUCTION

1.1 Subject

The subject of this report is an assessment of several sites for the future construction of a water treatment plant and treated water reservoir for the District of Peachland.

The water source is Peachland Creek (referred to as Water System No. 3) and the sites investigated are in the vicinity of the existing intake on Peachland Creek. The investigation is part of a long-term water system upgrade program outlined in the District of Peachland *Water Master Plan* dated April, 2007.

1.2 Purpose

The purpose of this assessment is not to provide the design for the water treatment facility, but to compare a variety of potential sites and select a preferred site for future construction. This will enable the District to secure the site and implement any preparatory work such as road access, extension of electric power and telecommunications network. The reservoir is anticipated to be constructed in 2011 in order to address the fire storage concerns; selection of a site in 2010 will facilitate the planning and preliminary engineering process for the reservoir.

1.3 Scope

The site selection exercise at this stage is without benefit of water treatability studies or water treatment process selection.

It has been assumed that filtration will ultimately be required in accordance with Interior Health (IH) policies and water quality targets. The type of filtration has not been addressed or selected, but it has been assumed that the parameters of concern will be similar to other upland sources in the Okanagan Valley (e.g. West Kelowna, Summerland). These parameters include Turbidity, Colour, TOC (Total Organic Carbon) and Algae Concentrations. Micro-organism concerns (Viruses, Bacteria, and Protozoa) will also need to be addressed in accordance with IH targets.

The finished water storage requirement is taken from the *Water Master Plan*. The preferred option was adopted as Scenario 3: Peachland Creek Gravity Supply.

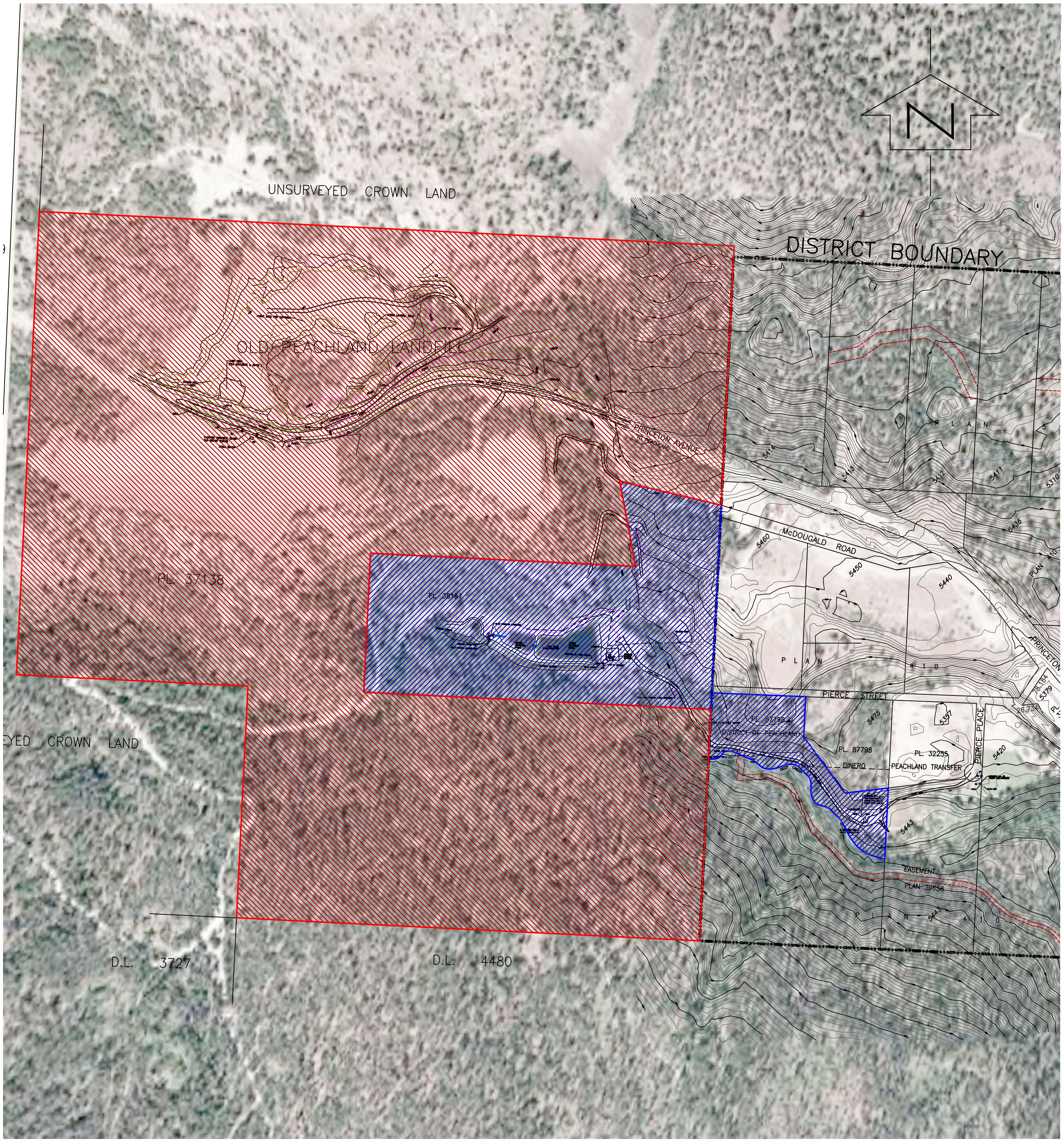
1.4 Methodology

The methodology adopted for the site comparison includes some fundamental guidelines:

a. The site should preferably be on land currently owned or leased by the District of Peachland. Privately-owned sites may be considered if there are no suitable Peachland-owned sites.
b. The elevation of the site for the finished water storage reservoir should be at the current elevation of the operating grade line for Water System No. 3 (579.80 m ASL) or higher, in order to fulfill the pressure requirements of Scenario 3.
c. The site should lend itself to phasing flexibility in terms of reservoir and water treatment plant construction.

With the foregoing basic guidelines, a series of comparison criteria were formulated. These include access, energy consumption, visibility, and ease of integration to the existing system.

Figure 1.1 shows the general area around the existing intake and the lands owned or leased by the District of Peachland.



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OWNED AND LEASED CROWN LAND

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2.0 WATER TREATMENT PLANT

2.1 Type of Treatment

The water supply must conform to the BC Drinking Water Protection Act Regulation. Interpretation and enforcement of the Regulation is undertaken by Interior Health. Interior Health provides target water quality objectives for this purpose. These are known as the 4-3-2-1-0 objectives. Briefly stated, these require:

- 4: 4-log (99.99%) Virus inactivation.
- 3: 3-log (99.9%) Giardia and Cryptosporidium inactivation.
- 2: a minimum of 2 barriers to prevent micro-organism breakthrough.
- 1: a maximum turbidity of 1.0 NTU
- 0: Zero Coliform bacteria

While not all water systems meet these objectives, Interior Health recommends that water system planning address these targets. The targets are generally achieved by filtration and disinfection. If source water turbidity is consistently below 1.0 NTU, it may be sometimes possible to defer filtration. However, the provision to allow deferral only postpones the filtration requirement. The site selected must have the ability to accommodate a water filtration plant.

Peachland Creek source water is generally of good quality, but Turbidity is known to be higher than 1.0 NTU during spring freshet. Colour is also present through the spring and summer months. Therefore, while short-term deferral may be possible, it is prudent to plan for filtration in the site selection exercise.

The historical Turbidity data can be summarized as follows:

Month	2004		2005		2006		2007		2008		2009	
	AVG	>1NTU	AVG	>1NTU	AVG	>1NTU	AVG	>1NTU	AVG	>1NTU	AVG	>1NTU
Jan	.17	-	.29	-	.32	-	.34	-	.36	-	.34	-
Feb	.15	-	.27	-	.25	-	.38	-	.32	-	.28	-
Mar	.33	-	.26	-	.27	-	.45	-	.38	-	.45	-
Apr	.86	9	.39	8	.74	3	.89	10	.52	-	.45	-
May	1.16	16	1.14	1	2.33	1	1.14	1	1.65	24	.65	3
Jun	.86	1	.83	5	1.37	1	.82	2	.84	1	.68	1
Jul	.50	-	.54	-	.63	2	.68	-	.52	-	.60	-
Aug	.47	-	.48	-	.46	-	.58	-	.56	-	.51	-
Sep	.75	6	.44	-	.48	-	.58	-	.52	-	.51	-
Oct	.42	-	.31	-	.35	-	.42	-	.47	-	.40	-
Nov	.30	-	.30	-	.33	-	.40	-	.43	-	.32	-
Dec	.29	-	.33	-	.36	-	.44	-	.45	-	.28	-

The monthly averages are typically below 1.0 NTU except for the months of May and June. The number of exceedances (turbidity greater than 1.0 NTU) range from 1 to 24. They occur mainly in the months of April and May.

Two basic forms of filtration are generally used in municipal water treatment plants. These are known as:

a. Conventional rapid sand filtration
b. Membrane filtration

Other forms of filtration such as "Slow Sand Filtration" and "Diatomaceous Earth Filtration" are also used, but typically limited to very small systems.

a. <u>Conventional Rapid Sand Filtration</u>
   There are several variations on this type of filtration, but the most common is a multi-media granular filter bed consisting of a top layer of crushed anthracite coal, a middle layer of sand, and a bottom layer of finely crushed garnet.

   This type of filtration is often preceded by a clarifier if source water turbidity is high. The clarifier removes heavier particles and thereby prevents overloading of the filter. Clarifiers can come as gravity settling tanks or as upflow dissolved air flotation (DAF) tanks. DAF is somewhat more power consumptive, but has better ability to remove lighter suspended particles such as algae.

If source water turbidity is relatively low (less than 5 NTU), the clarifier may be avoided, in which case the process is referred to as "Direct Filtration" In either case, a coagulant must be added, in the form of Iron or Aluminum salts to assist in the clarification or filtration process. The addition of coagulant results in the agglomeration of tiny particles to form larger "flocs" which are more easily settled out of suspension and more easily trapped in the filter media.

The US EPA (Environmental Protection Agency) provides log removal credits for micro-organisms based on the type of filtration used. These are based on Giardia and Cryptosporidium removal capabilities as follows:

Conventional Filtration:	2.5 log credit
Direct Filtration:	2.0 log credit

Giardia Lamblia and Cryptosporidium Parvum are two of the more common protozoa found in the Okanagan basin. Ingesting these micro-organisms can result in Giardiasis (beaver fever), or Cryptosporidiosis.

It is evident that this form of filtration alone will not achieve the Interior Health 3 log target for Giardia and Cryptosporidium. The remaining credit can be achieved by disinfection with UV light irradiation. Secondary disinfection (to maintain water quality in the piped network) is provided by chlorination.

The use of conventional filtration technologies will require provision of UV irradiation and chlorination to meet the Interior Health targets.

b. Membrane Filtration

Membrane filtration utilizes either hollow fibre or spiral wound membranes with pore sizes smaller than the targeted micro-organisms; typically 1 micro-metre (micron). With such small pore sizes, micro-organisms are trapped on the upstream side of the membrane and disposed of in the backwash.

Membrane filters typically require fine screening to remove larger grit or silt particles and protect the membranes from damage or excessive plugging. Coagulants are required if Colour removal is desired. Backwash is carried out automatically.

Since the membrane pore size is smaller than the targeted protozoa (Giardia and Cryptosporidium), the 3 log removal standard is easily achieved with membrane filtration. UV disinfection should not be required. Chlorination, however, is necessary for Virus and Bacteria inactivation, and to maintain the required free chlorine residual in the distribution network.

2.2 Plant Residuals

Water filtration plants produce several wastewater streams. These include:

- Clarifier sediment or DAF float (sludge)
- Filter backwash water
- Filter rinse water

Filtration plants also require sanitary facilities for the operator(s), and a laboratory. These facilities produce wastewater which must be dealt with. Wash down water for routine cleaning also requires disposal.

The sites under consideration are remote from the community sanitary sewer system. Extension of sanitary sewers to the upper Princeton Avenue area is not anticipated in the near or medium term. Therefore, wastewater streams will have to be processed on-site.

Plant residuals (backwash) are sometimes allowed to return to the creek downstream of the intake. However, if coagulant chemicals are used, they must be removed prior to discharge. In the case of Peachland Creek, which is a fish-bearing stream, plant wastewater must be treated to meet the DFO water quality requirements for fish. Another potential form of disposal utilizes ground infiltration. In that case, the soil permeability and depth to the groundwater would need to be established.

Most water filtration plants return the process residual water back through the plant in order to reduce wastage. Peachland Creek has a limited watershed yield and expected to further decrease in the long term due to climate change. Therefore, recycle of plant residual water is an option which should be considered.

The US EPA has a long-standing Filter Backwash Recycle Rule (FBBR) which requires that filter backwash water be returned to the front end of the plant. Additional precautions and barriers for recycled water include:

- Sedimentation
- Filtration
- Disinfection

Sedimentation requires dewatering and disposal of the sediment (sludge). Any secondary settling process should not recycle its liquid fraction. For the purpose of site comparisons, the following residual waste stream has been assumed:

a. Clarifier Sediment or DAF float:
   - to settling pond
   - liquid fraction disinfected by UV and returned to plant
   - settled sludge dewatered mechanically
   - liquid fraction to ground infiltration
   - dewatered solids to landfill

b. Filter Backwash:
   - to settling pond
   - liquid fraction disinfected by UV and returned to plant
   - settled sludge dewatered mechanically
   - liquid fraction to ground infiltration
   - dewatered solids to landfill

c. Filter Rinse:
   - return directly to head end of plant

d. Membrane Filter Backwash:
   - to settling pond
   - liquid fraction disinfected by UV and returned to plant
   - settled sludge dewatered mechanically
   - liquid fraction to ground infiltration
   - dewatered solids to landfill

e. Sanitary Sewage:
   - to septic tank and leaching field

f. Laboratory Wastewater:
   - to holding tank

g. Wash down Water:
   - to sedimentation tank and leaching field

h. Spill Containment:
   – to holding tank

i. Storm and Roof Drains:
   – to surface drainage system

2.3 Finished Water Reservoir

The *Water Master Plan* proposes an ultimate size of 7,500 m^3 for finished water storage at the upper Princeton area. This includes provision for filter backwash water, fire storage and peak demand storage.

The backwash water component should allow for two backwash volumes. An approximate volume requirement can assume a backwash flow rate of 1700m^3/hr. A typical backwash cycle may last 20 to 30 minutes, so the volume required for two cycles is 1700 x 0.5 x 2 = 1700 m^3.

The above volume could be accommodated below the filtration plant or in a separate reservoir. Provision of backwash water directly below the filters simplifies the backwash pump system and it is assumed that a minimum storage of 1700 m^3 can be accommodated below the plant.

2.4 Plant Sizing Criteria

The water treatment plant capacity for the long-term MDD (Maximum Daily Demand) is taken as 500 L/s (from Figure 2.0 in the 2007 *Water Master Plan*). The recommended Scenario 3 provides this from Peachland Creek while the facility will be constructed in stages, the site should provide sufficient space for the long-term. The MDD rate of 500 L/s equates to 1800 m^3/hr.

Typical loading rates for the various unit processes range as follows:

- Conventional Gravity Clarifiers: 10 to 12 m/hr
- High Rate Ballasted Clarifiers: 40 to 50 m/hr
- High Rate DAF: 30 to 40 m/hr
- Conventional Multi-Media Filters: 12 to 14 m/hr
- Filter Backwash Rate: 40 m/hr
- Membrane Filters: 4 m^3/hr/tube
- Rapid Mixing: 0.5 to 1 minute
- Flocculation: 5 to 8 minutes

Table 2.1 provides a brief overview of the floor space requirements for three water treatment plant configurations. The first column is a conventional gravity clarifier, multi-media filtration plant; the second column is similar but utilizes either a high rate ballasted clarifier, or a high rate DAF. The third column utilizes membrane filtration preceded by fine screening and flocculation.

Table 2.1 – Water Treatment Plant Component Sizing

Component	Areas for Plant Capacity of 1800 m³/hr			
	Loading Rate m/hr	Conventional m²	High Rate m²	Membrane m²
Conventional Clarifier	10	180	-	-
High Rate Clarifier	40	-	45	-
High Rate DAF	40	-	Same	-
Fine Screening	-	-	-	50
Conventional Filter	12	150	150	-
Membrane Filter	-	-	-	200
Rapid Mixing	1 minute	20	20	20
Flocculation	5 minutes	80	80	80
Chemical Storage	-	90	90	90
Chlorination	-	100	100	100
UV	-	60	60	-
Office/Lab/Control Room	-	160	160	160
Electrical Room	-	60	60	60
Sanitary Facilities	-	30	30	30
Workshop	-	40	40	40
Total Floor Space		**970**	**835**	**830**

Table 2.1 indicates a floor space requirement of 800 to 1000 m² depending on the selected process. This is typically divided on two floors because of the depth of clarifiers and filter tanks.

The "operating" floor level should accommodate the mixing, flocculation, clarifier and filter area, as well as the office and control room and sanitary facilities. Assuming either a high-rate or membrane process, this adds up to 500 to 600 m² for the main operating floor; other components can be located on the lower floor.

For the purpose of the site selection exercise, a footprint of 600 m² will be used. If a clearwell is located below this building, with a 3 m water depth, the volume is approximately 1800 m³. This coincides with the required backwash volume.

2.5 Residuals Processing

Using the assumption that residuals will be recycled, the provision of settling ponds, leaching fields and a dewatering building should be accommodated by the site. The following are approximate footprints for these components:

• Dewatering Building:		100 m^2
• Settling Ponds:	2 @ 1,500 m^2 =	3,000 m^2
• Leaching Fields:	2 @ 200 m^2 =	400 m^2
Total		**3,500 m^2**

2.6 Finished Water Reservoir

The *Water Master Plan* recommends a storage volume of 7,500 m^3, including clearwell backwash storage. If approximately 1,800 m^3 is available below the plant, an additional 5,700 m^3 will be required for fire and emergency storage. Assuming a practical water depth of 5 m, the reservoir footprint is 1,140 m^2. Allowing for access and parking, the area requirement is approximately 1,400 m^2.

2.7 Total Area Requirement

Using an allowance for roads, parking, landscaping and storage of: 2,500 m^2, the approximate land requirement if the treatment plant, residuals processing and finished water reservoir are all on the same site is:

• Water Treatment Plant:	600 m^2
• Residuals Processing:	3,500 m^2
• Reservoir:	1,400 m^2
• Roads, Parking Landscaping:	2,500 m^2
Total	**8,000 m^2 (0.8 hectare)**

A site of approximately 1 Hectare (2.5 acres) is required.

2.8 Disposition of Plant Solids

Table 2.2 presents an approximation of solids production based on a waste stream consisting of clarifier sludge and filter backwash water. The filter rinse-to-waste cycle is typically returned directly to the head end of the plant and does not enter the waste processing stream.

Settled sludge in the clarifier is removed on a regular basis and filter backwash cycles can be assumed as once per day. The combined wastewater stream has a typical suspended solids concentration of 250 – 300 mg/L (0.035 – 0.030%).

The settling pond results in a settled sludge ranging from 1.5% to 2% solids concentration. The thickener can achieve a solids concentration of approximately 4%. Dewatering of the thickened sludge can be undertaken mechanically or with the use of drying beds if suitable land is available. In either case, dewatered sludge has approximately 20% solids content. **Figure 2.1** presents a schematic diagram of the plant wastewater processing stream.

The plant operates at maximum capacity during the high consumption days of summer. This is also when the maximum amount of wastewater is generated. The plant throughput during the non-irrigating season is considerably less, and wastewater production is also reduced. **Table 2.2** presents the calculated sludge production over a variety of throughput rates.

Table 2.2 – Daily Sludge Production Projection

Plant Output m³/d	Wastewater 0.025% m³/d	Settled Sludge 1.5% m³/d	Thickened Sludge 4% m³/d	Dewatered Sludge 20% m³/d
10,000	1,540	26	10	2
20,000	2,100	35	13	3
30,000	3,240	54	20	4
40,000	3,670	61	23	5
50,000	4,080	68	26	6
60,000	4,530	78	33	7

The monthly consumption pattern is based on the trend plotted in the *Water Master Plan*. The records used were from 1999 to 2002, these recorded total monthly consumption for each month on a system-wide demand basis.

The projected consumption for the year 2040 is approximately 2 times the existing consumption rates. The projected average daily rates on a monthly basis, for the year 2040 are shown on **Table 2.3**.

Table 2.3 – Monthly Consumption Variation (2040)

Month	Average Demand (m^3/d)
January	6,000
February	6,000
March	6,000
April	12,000
May	20,000
June	34,000
July	40,000
August	36,000
September	24,000
October	12,000
November	6,000
December	6,000

In order to establish the maximum future quantities, the year 2040 demands have been used. Quantities under present consumption rates would be roughly 50% of the future projections.

Table 2.4 shows the calculated daily quantities of wastewater and sludge on a monthly basis for the year 2040.

Table 2.4 – Monthly Wastewater and Sludge Quantities

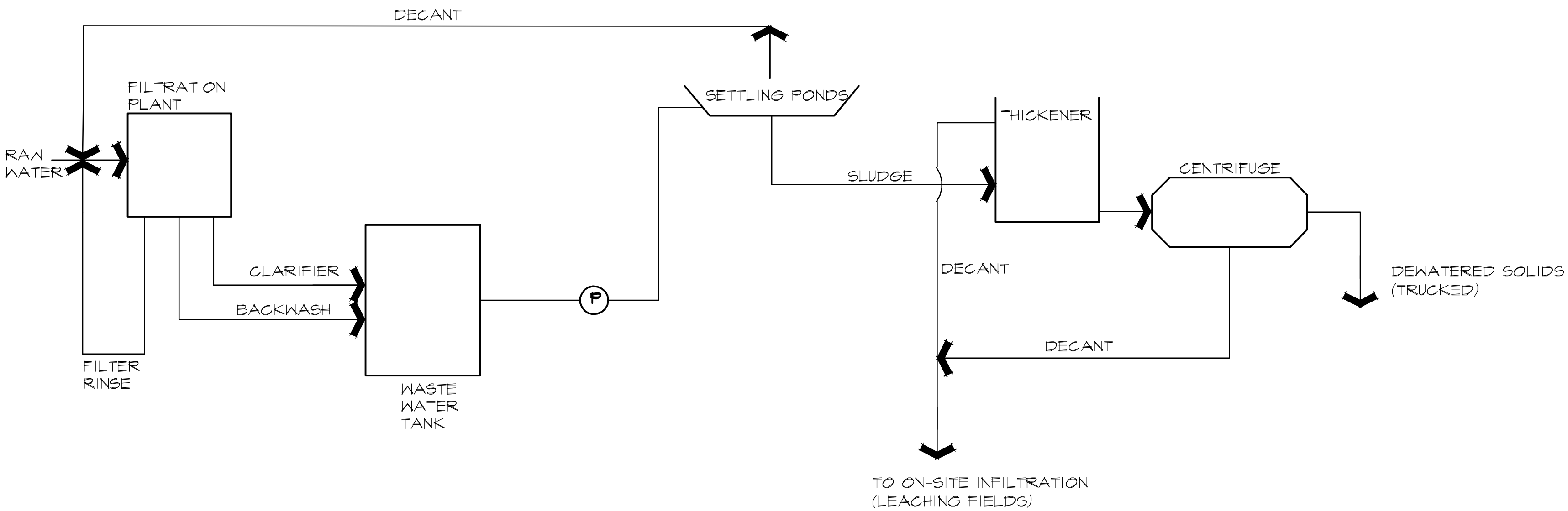
Month	Average Daily Plant Output	Wastewater 0.025% m^3/d	Settled Sludge 1.5% m^3/d	Thickened Sludge 4% m^3/d	Dewatered Sludge 20% m^3/d
January	6,000	1,400	23	8.6	1.7
February	6,000	1,400	23	8.6	1.7
March	6,000	1,400	23	8.6	1.7
April	12,000	1,800	30	11.3	2.3
May	20,000	2,100	35	13.1	2.6
June	34,000	3,400	57	21.4	4.3
July	40,000	3,670	61	22.9	4.6
August	36,000	3,500	58	21.8	4.4
September	24,000	2,800	47	17.6	3.5
October	12,000	1,800	30	11.3	2.3
November	6,000	1,400	23	8.6	1.7
December	6,000	1,400	23	8.6	1.7

The total annual quantity of dewatered sludge is in the order of 1,000 m^3. This material is largely inorganic (silt and fine sand) and must be disposed of. Discussions with the District of Peachland indicate potential disposal sites to consider include:

- The landfill site north of Princeton Avenue
- The abandoned gravel pit west of McDougall Road

Investigation for the potential use of these sites will require field investigations to determine the groundwater regime, the types of subsurface soils, and the direction of groundwater flow.

DISTRICT OF PEACHLAND



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3.0 AVAILABLE SITES

3.1 Initial Search Results

The guidelines used for the initial search of sites include:

- The overall size requirements of 0.8 to 1.0 hectare.
- Land should be owned or leased by the District of Peachland. Crown land which might be available for lease can also be considered. Private land is the least desirable, but can be considered if all other options are exhausted.
- Land should be in the vicinity of the existing intake in the upper Princeton area.
- The elevation of the site should be at least as high as the current intake elevation, or higher, to achieve the required grade for a gravity supply.

The initial search revealed three sites that meet the above criteria:

Site 1 – Lot B – Plan KA 87798 DL 2538, at the west end of Pierce Street. Owned by the District of Peachland (see **Figure 3.1**). This parcel is currently zoned as "Park" in the District's zoning by-law.

Site 2 – Lot I – Plan 38197, owned by the District of Peachland – currently used for storage. (see **Figure 3.1**).

Site 3 – Lot I – Plan 38197, owned by the District of Peachland at the west end of McDougald Road. (see **Figure 3.1**).

3.2 Other Potential Sites

Other potential sites could be considered, although they do not meet all the criteria. These include:

a. <u>The Existing Chlorination Station Site.</u>
   This site will not accommodate the finished water reservoir, nor is it large enough for settling ponds for plant residuals. Nevertheless, a small footprint membrane plant could be located on top of the existing chlorination tank. The tank is 20 m x 20 m, so at 400 m^2 it could accommodate the membrane filter modules. Other components could be located in a building extension to the north. Treatment of backwash water would be achieved mechanically. The existing chlorination tank holds 1,700 m^3 - sufficient for backwash purposes. Finished water storage would be in a separate location.

b. Site on Princeton Avenue
There is a site on Princeton Avenue immediately west of the District boundary. The site is on Crown Land leased to the District of Peachland. The site may be accessed from either McDougald Road or Princeton Avenue. This site is high enough to supply the Law Street reservoir zone by gravity (elevation 648.0 m). The zone (PZ 3-4) is, however, relatively small.

Other privately-owned parcels in the area may be available, but these have not been assessed at this time since the cost and availability of the land must be factored in.

3.3 Site Comparisons

The initial discussion and comparison of sites deals with Sites 1, 2 and 3. **Table 3.1** presents the physical characteristics of each site, and provides observations on the ease of connection to the existing system.

Table 3.1 – Site Comparisons

Parameter	Site 1	Site 2	Site 3
1. Area	160 x 50 (0.8)	120 x 70 (0.84)	110 x 80 (0.88)
2. Slope	590 to 606	594 to 600	620 to 630
3. Access	Difficult	Existing	Good
4. Power	3 Ø on Pierce Street	3 Ø Extension – 100 m	3 Ø Extension – 100 m
5. Raw Water Supply	300 m	150 m	300 m
6. Finished Water Tie-in	100 m	80 m	300 m
7. Alternate Tie-in to Princeton	500 m	800 m	900 m
8. Pump Lift (TDH)	30 m	20 m	60 m
9. HP @ 500 L/s	250 HP	175 HP	500 HP
10. Distance to Creek	50 m	100 m	250 m
11. Soils	Unknown	Unknown	Unknown
12. Trees	Treed	Mostly Cleared	Treed
13. Visibility	High	Low	Medium
14. Earthwork	High	Low	Medium

3.4 Discussion

1. Area:
   All 3 sites have roughly equivalent available areas of at least 0.8 hectare.

2. Elevation:
   Site 1 is the steepest site with over 16 m change across the site. Site 2 is flatter (6m); site 3 has a 10 m grade change.

3. Access:
   Access to Site 1 is off Pierce Street, but there is a large grade change (10 m) which would require significant earthworks. An alternate access may possible from the dam access road. Site 3 has the easiest access off the end of McDougald Road.

4. Power:
   Site 1 has 3-phase power nearby. Sites 2 and 3 require extensions in the order of 100 m.

5. Raw Water Supply:
   The shortest connection to the raw water intake is at Site 2 (150 m). The other two sites require 300 m of new raw water line.

6. Finished Water:
   The shortest tie-in back to the existing main is from Site 2 (80 m). In the long term, however, this supply main will be too small and the *Water Master Plan* proposes replacement of the main supply line with a larger pipe.

7. Alternate Tie-in:
   This item considers alternate routing for a new supply line in the longer term. The addition of a new large diameter main beside the existing pipe may involve precarious construction, so an alternate route may be preferable. The shortest alternate route is from Site 1.

8. Pump Lift:
   Water must be pumped to each of the sites from the existing intake. The relative static lifts are:

   To Site 1: 578 to 598: 20m
   To Site 2: 578 to 596: 18m
   To Site 3: 578 to 636: 58m

9. HP @ 500 L/s:

   This item provides an indication of the pump horsepower required at the future MDD flow condition. The lowest requirement is with Site 2. The highest is with Site 3.

   Pumping at the MDD rate does not occur every day, so an annual power consumption estimate is not a simple extension. However, it can be approximated with some assumptions:

   - 240 days at 1/3 of the MDD rate : 167 L/s
   - 85 days at ¾ of the MDD rate: 375 L/s
   - 40 days at the MDD rate: 500 L/s

   Using a power rate of $0.07/kW-hr (kilowatt-hour), the relative annual power costs are approximately:

   - Site 1: 822,000 kW-hrs: $57,600 per year
   - Site 2: 578,000 kW-hrs: $40,500 per year
   - Site 3: 1,644,000 kW-hrs: $115,000 per year

10. Distance to Creek:

    This is provided for considerations in the design of ground infiltration systems for the domestic wastewater and the relative subsurface travel time to Peachland Creek.

11. Soils:

    There is evidence of granular deposits in most cut banks in the area. However, on-site investigations have not been done and should be undertaken.

12. Trees:

    Sites 1 and 3 are treed; Site 2 has been previously cleared for the storage area.

13. Visibility:

    The most visible site is Site 1.

14. Earthwork:

    This is a measure of the extent of earthwork required to prepare a site. Site 1 has the largest elevation change and would require the most earthworks. Site 2 is the flattest (6 m elevation change).

3.5 Initial Site Selection Discussion

Discussions with District staff concluded that Site 1 could be eliminated because of its awkward topography and difficult access. Site 2 also has a somewhat circuitous access, but this road must be maintained in any event to retain access to the dam, intake structure and future pumphouse. The gravel road paralleling Deep Creek should be widened and upgraded if Site 2 is selected.

Site 3 has very good access from McDougall Road and minimal improvements would be required. There may also be an advantage in the long term upsizing of the main supply line. The main supply pipe currently runs parallel to Deep Creek. The Water Master Plan identifies the upsizing of the supply main from the intake to Turner Avenue is required by 2020. The Master Plan suggests an additional 800 mm diameter supply main. Construction of such a large diameter main would be more economical on McDougall Road. A potential layout of a filtration plant and reservoir is provided on **Figure 3.3.**

Site 3 is at a much higher elevation, and the hydraulic grade line would be raised from the current 579.8 m ASL (Above Sea Level) to 634 m ASL; an increase of 54 m, or 78 psi. The additional 78 psi may require pressure reducing stations and establishment of an additional pressure zone.

Overall, Site 2 appears to have more positive attributes, chief of which is the lower power costs incurred by pumping to Site 2. The hydraulic grade line from Site 2 is 15 m higher than the current grade line, and represents a nominal pressure increase of 20 psi.

3.6 Phasing Considerations

The Water Master Plan suggests construction of the reservoir in 2011 in order to address the fire storage concerns in a timely matter. The water filtration plant is planned for 2017 construction.

If Site 2 is adopted, the reservoir construction would also require the pumping system to deliver water from the current intake. The existing chlorine contact tank at the intake would continue to be used for that purpose, and chlorinated water would be pumped to the new reservoir. There is a risk, however, that low winter flows could deplete the chlorine residual and "touch-up" chlorination may be required. This could be undertaken with a second "top-up" injection from the existing chlorinator.

The piping configuration has been briefly examined and a suggested layout is provided on **Figure 3.2**.

3.7 Action Plan

The following are suggested short-term actions to enable proceeding with the site development:

- Review the site comparisons and make a site selection.
- Prepare terms of reference for a geotechnical and hydrogeological investigation of the site.
- Refine the proposed piping layout and phasing plan.
- Undertake a detailed chlorine decay prediction to determine the requirements for re-chlorinating.
- Undertake a more detailed analysis of the pumping requirement in the conventional filtration scenario and the membrane filtration scenario.
- Begin water quality sampling for temperature, conductivity, pH, TOC, Colour and Turbidity. Treatability work with either bench scale testing or a pilot plant should be planned for 2015.
- Prepare a pre-design report for the construction of the reservoir and pump station in 2011. A logistics and functional plan will be required to ensure that the chlorination system will continue to be effective.

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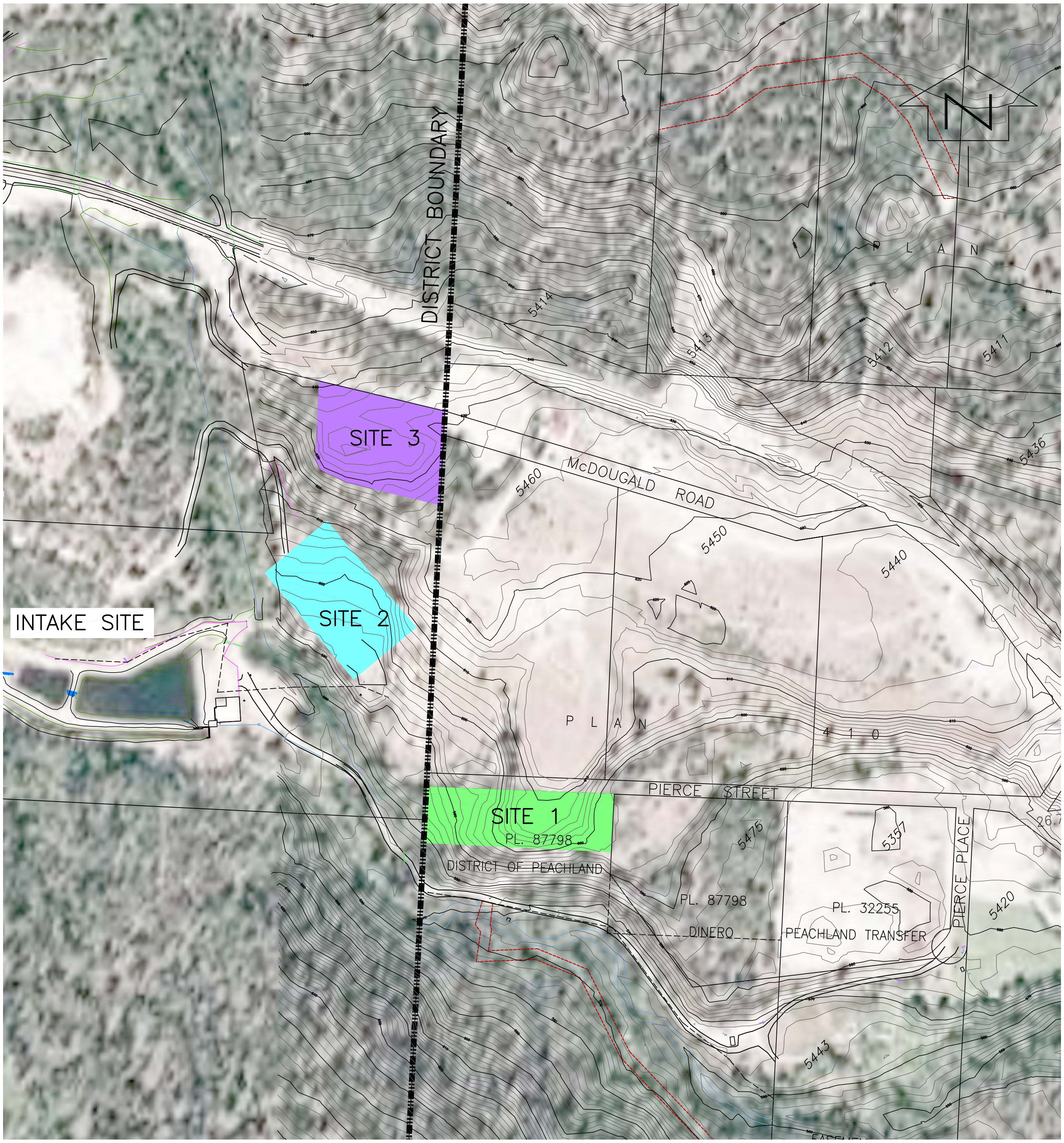

LEGEND

- SITE 1, 160 x 50m
- SITE 2, 120 x 70m
- SITE 3, 110 x 80m

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		AVAILABLE SITES

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CONCEPTUAL LAYOUT AT SITE 2		

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CONCEPTUAL LAYOUT AT SITE 3		