Table 4.2 Effort and results of seine netting efforts in P Lake on August 11, 2005

Haul #	Date	Length of haul (m)	Number of fish captured
SH 1	Aug 11	15	0
SH 2	Aug 11	15	0
SH 3	Aug 11	20	0
SH 4	Aug 11	10	0
SH 5	Aug 11	10	0
SH 6	Aug 11	10	0
SH 7	Aug 11	20	0
Total			**0 fish**

4.2.1.3 Snorkel survey

Table 4.3 presents the results of the snorkel surveys conducted in P Lake. An estimated area of 1380 m^2, and the complete diversity of habitat types, were examined during the snorkel surveys. This area represented approximately 10.6% of the total area of the lake. It should also be noted that although the snorkel survey concentrated on 1.5 m width on either side of the snorkeller, visibility often extended beyond this width (e.g., 3+ m). No fish were observed during any of the transects completed.

Table 4.3 Effort and results of snorkel surveys in P Lake during August 11, 2005.

Transect #	Date	Length of transect (m)	Width of transect (m)	Area sampled by individual transects (m^2)	Number of fish observed
T1	Aug 11	50	3.0	150	0
T2	Aug 11	70	3.0	210	0
T3	Aug 11	75	3.0	225	0
T4	Aug 11	85	3.0	255	0
T5	Aug 11	60	3.0	180	0
T6	Aug 11	120	3.0	360	0
			Total	**1380**	**0**

4.2.1.4 Visual observations

Although efforts were made to observe and record any fish that may have been made during visual bank observations, no fish were observed during the bank surveys. No attempt was made to document the level of effort expended during the bank surveys.

4.2.2 Habitat and Water Quality Survey of P Lake and Outlet

4.2.2.1 Habitat

P Lake

Substrates in P Lake were composed primarily of boulders and fractured rock (**Figure G**). Any cover which could be utilized by fish would have been provided primarily by depth, large boulders and fractured rock. Substrates at maximum depths were primarily composed of sands and fines. The north shoreline substrate was predominately fractured rock, some greater than 1-2 m diameter, (**Photo 1**), while the south shoreline substrate was a mixture of gravel with areas of sand and fines (**Photo 2**). All other shorelines consisted of rock.

No aquatic vegetation was observed in P Lake and only small amounts of algae were observed on the substrate. Caddisfly (trichopterids) and freshwater crustaceans (amphipods) were also observed in P Lake.

Outlet and Inlet of P Lake

The outlet of P Lake is characterized a small channel that drains into a small wetland area. From the wetland area, it drains through a small channel and over a large cliff forming a waterfall (**Photo 3**). Below the waterfall the outlet stream was intermittent as flow was subsurface and at times lacked a defined channel. Intermittent surface flow also continued downstream and is apparent where the channel passes through grassy/mossy area. Mean width and depth of the reaches where a defined channel occurred were approximately 0.1, and 0.05 m, respectively) (**Photo 4**). Below the grassy/mossy area, the stream again goes sub-surface and lacks a defined channel once the outlet reaches boulder cobble shoreline (**Photo 5**). **Figure G** provides a map of the shoreline and distribution of substrates in P Lake and the outlet into Telik Inlet.
The outlet stream does not provide fish access to P Lake from the ocean due to the large cliff and lack of a defined channel at several locations. Limited summer flows and conditions also suggest that the outlet stream would freeze to the bottom in winter.

The inlet stream into P Lake is best described as having a no or limited definition channel, and if fish were present in the lake, it is highly unlikely the inlet area would provide fish habitat. It would also be expected to freeze to the bottom in winter.

The mean depth and wetted widths for both the inlet and outlet streams where defined channels existed were both approximately 0.1 m, and 0.2 m, respectively.

Marine Environment
Substrates in the small bay in Telik Inlet where the P Lake outlet stream eventually drains were dominated by very clean cobble and boulders. Some algae was observed, but in limited quantities. Benthos were not sampled due to high tide conditions, time limitations, and the coarseness of the substrate.

4.2.2.2 Water quality
Water quality measurements collected from P Lake with the hand-held YSI units are provided in **Table 4.4**. Some of the above readings, however, were unexpected (e.g., pH = 10.1 exceeds the CREM guidelines for aquatic life of 9.0), so the units were re-calibrated after being returned to Dillon's Yellowknife office. The results of the recalibrations suggested that the sensor units had been damaged in transport to Cape Dorset, and therefore, equipment malfunction is suspected and the above results collected during the present field trip should not be considered reliable.

Table 4.4 Water quality measurements collected from P Lake on August 11, 2005

Parameter measured	Measurement and units
Dissolved oxygen	3.0 mg/l
pH	10.1
Conductivity	42 microsiemens
Salinity	0.0 ppt
Temperature	10.2°C

Other water samples were collected, but were not analyzed because unexpected delays in transit were encountered which exceeded the amount of time which would provide reliable analysis for some parameters (e.g., fecal coliforms).

4.3 Conclusions of Fisheries Investigations

Based on the results of the 2005 fisheries investigations, the absence of historical reports documenting fish presence in P Lake, and the presence of impassible barriers that prevent fish movement between P Lake and other fish-bearing waters, it can be concluded that P Lake is barren of fish. The intermittent flow conditions and waterfalls over the cliff indicate that fish passage into P Lake from the marine environment is impossible.

Given that the P Lake system is barren of fish, there is no reason to suggest that converting P Lake into an output lagoon is likely to cause a Harmful Alteration, Disruption or Destruction of fish habitat (HADD). If a HADD is unlikely, a Federal Fisheries Act Authorization for a HADD will not be required.

5 LAGOON CONSTRUCTION

The P Lake area provides sufficient land to develop an annual retention lagoon, and also use P Lake as a secondary short retention lagoon. The wetlands down gradient of P Lake, which are indigenous to the area, will provide some nutrient and total suspended solids (TSS) removal as well.

As illustrated by aerial photographs and noted topographic maps, the area east of P Lake is situated in a geographic low point, making the area ideal for construction of a bermed lagoon (**Figures E and H**).

The level of treatment obtained from each stage of the system is discussed in **Section 6**.

5.1 Lagoon Configuration

The annual retention lagoon will be constructed using the natural topography of the area. Plug flow conditions will prevail and are important during the time of annual discharge in order to prevent short circuiting, or raw sewage by-passing treatment and directly discharging to P Lake. The lagoon will be constructed to facilitate anaerobic sewage treatment using the following characteristics:

- 3.5 m liquid operating depth
- 0.5 m of allowance on the lagoon bottom for sludge accumulation

The size of the lagoon was determined to accommodate the above design parameters and the predicted volume of sewage generated in 2026 (96 100 m^3). The parameters of the lagoon are illustrated in **Drawing 101**.

The development of the earth work quantities was completed based on a site specific survey and using three dimensional modeling of the landforms and final design.

5.2 Berm Construction

A site specific geotechnical study was completed, which is submitted in **Appendix G.** Based on the geotechnical report, the lagoon will have;

- 1 m freeboard
- 3.5 meter operating depth
- 0.5 meter dead area (sludge retention) at the base of the lagoon.
- Total berm height will be 5 meters.

The berm construction will include;

- 4 m wide at top to facilitate construction;
- Inside to have a 2.5:1 (H:V) slope;
- Outside to be 2.5:1 (H:V) slope;
- The berms to have riprap protection over the inside slope in areas subject to ice and wave action.
- Where fine grained material is used in the berm construction, the material will be protected against wind erosion with a layer of 50 mm minus gravel;
- Emergency spillways are to be provided to prevent overtopping of the berm. The spill way to be protected from erosion with riprap or a half culvert pipe; and
- An impermeable liner placed vertically on the berm and anchored 2.0 m into the sub-base.
- The berm will be constructed with a Bentomat® liner placed near vertical on the inside of the berm. The liner will be keyed into the base to a depth of 2.0 meters. Bentomat is a high performance environmental liner manufactured with durable high-strength geotextiles and a uniform layer of low-permeability sodium bentonite. Bentomat has a long lasting resistance to physical or chemical break-down in harsh environments. The bentonite's high swelling capacity and low permeability provide an effective hydraulic seal. Because of their unique self-healing sodium bentonite base, liners resist cracking that typically occurs in compacted clay liners subjected to repeated freeze/thaw conditions.
- The liner will be installed vertically in the berm only. No liner will be used in the lagoon base. The field investigation, followed by the thermal analysis complete by AMEC indicates that the lagoon base is competent. Freeze back will occur into the berm, and the base of the lagoon will remain impermeable due to permafrost conditions. Therefore AMEC does not recommend the use of a liner on the base of the lagoon.

The types of material to be used in the berm construction are described in section 4.6.

5.3 Truck Pad and Turn-Around

The truck turning access pad will need to be constructed at the top of the lagoon berm to allow for gravity discharge from the truck to the lagoon. The siting of the pad must provide for a cost effective construction, and also provide for a safe operation for the truck drivers in all climatic conditions. The truck pad will have the following elements;

- A minimum turning radius of 15 meters,
- Bollards along the edge of the lagoon to provide for driver safety,
- Stop logs at the discharge location to give the truck driver a physical point to stop the truck,
- Delineators along the edge of the truck pad to indicate the edge of the embankment in winter conditions,
- The side slopes of the truck pad will be protected against erosion with a layer of granular material. The erosion protection will have a minimum gradation of a 50mm minus material. Courser material maybe if economically available.

The truck turn around pad requires a radius of 15 m if a standard sewage truck is to be used. Should the owner feel that the use of larger trucks (B-Trains) are to be used, then a larger turning radius is required. The side slopes of the turn around pad are to be 3:1 (H:V).

5.4 Sewage Discharge Flume

Sewage will be disposed from sewage trucks into the lagoon though a free fall discharge pipe. With respect to trucked sewage, free fall discharges are simple in design and operation and easy to access and maintain. The discharge will be constructed with the following specifications:

- 2- 800mm nestable half culverts
- 30% slope
- CSP supported by PTW embedded into the embankment
- Rip rap will be placed on the interior slope of the lagoon berm along the length of the discharge flume, and to the base of the lagoon to prevent erosion.
- Bollards will be in place to prevent the sewage truck from damaging the flume
- Wheel stops will be placed in front of flume.

Drawing 101 shows a layout of the truck pad and discharge flume area. Details of the design elements of the truck pad and discharge flume are shown in **Drawing 109.**

5.5 Lagoon Discharge Structure

The effluent from the lagoon will be discharged to Lake P through discharge system. Several options are possible. Methods used at other locations in Nunavut include;

- A siphon (used in Qikigtarjuaq)
- A gas or diesel fired pump
- A gravity discharge and valve

The goals and objectives of the project are to provide a system with low operating costs and to simplify the operational elements. For these reasons the use of a pumped system is not recommended.

The use of a siphon works well once the lagoon is operating at design capacity. When it is operating in the early years the siphon system has proven problematic. Often the siphon requires priming with a vacuum pump at each discharge event. For these reasons the use of a siphon system is not recommended

The recommended system is the use of a gravity discharge line with a valve located in a manhole within the berm. This is a common system installed throughout Nunavut, most recently in Pond Inlet. Design components of the system include;

- An intake structure to raise the inlet of the pipe above the sludge deposition level. This is a 90 degree bend located at the inlet of the pipe.
- A steel pipe through the lagoon berm
- An access vault located within the berm. The manhole is located in the berm crest to facilitate access to the top of the manhole.
- A valve installed in the Manhole on the discharge pipe.
- A pipe out fall and discharge channel. The outfall area has riprap protection to prevent erosion and to dissipate the energy of the discharge.

Drawings 111 and 112 illustrate the plan and section of the proposed effluent discharge system.

5.6 Berm and Road Material

5.6.1 Berm

The design intent for the lagoon is an annual retention cell. The berms are to be constructed to provide low permeability to the sewage. The geotechnical investigation indicted that there is sufficient quantity of material that is suitable for use in the berm construction. Based on the geotechnical report (appended) the following will be used for the construction of the berm walls;

- Construction of the berm walls using a berm core of fine grained (low permeability) material. . The outer faces of the berms would have coarser grained material to prevent erosion on the inside of the walls to protect the berms from wave and ice erosion.
- The berm wall will have a near vertical liner to increase the berms impermeability. The liner will extend from 2.0 m below the base of the lagoon and extend to the top of the berm.

5.6.2 Road

The cost estimate for the road alignment and profile are based on the information provided by a site survey completed by sub Arctic Surveyors in August 2005. In order to access the lagoon location a steep rock face must be traversed. Access would be partially provided along an existing road but a new road 950 m in length and with an elevation change of approximately 56 m would need to be constructed. The plan and profile are shown in **Drawings 102-108**. The profile consists of interpreted data from existing contour mapping, survey data and digital mapping.

The road will be developed based on the following standards;

- The road width will be 8.0 meters. This will allow two trucks to pass safely.
- The maximum grade will be 8%. Where possible the road grade will be maintained below 6%.
- Guard rails will be provided on curves and areas of steep embankment.
- Curves will be a minimum of 25 meters to provide for road safety is adverse climatic conditions.
- Road side delineators will be install to assist in snow clearing
- Side slope of the road will be governed by the stability of the granular material used for the road construction. Geotechnical recommendation will be used to determine the minimum side slope. For safety reasons, a minimum slope of 2:1 will also govern.

The access road to the lagoon will be constructed to the following standards:

- Transportation Association of Canada (TAC): Geometric Design Guide for Canadian Roads, Section 3.1.6: Traffic Barriers (1999)
- Uniform Traffic Control Devices for Canada, Third Edition. Part C, Division 3: Hazard and Delineation Markings (1994)
- AASHTO, Roadside Design Guide, 1989

5.6.3 Granular Supply

Cape Dorset has approximately 30,000 m^3 of available granular material. A blast and crush operation will likely be required as part of the road and lagoon construction. We have had several discussions with the community representatives, and with the local contractor Fred Schell. Based on these conversations it is understood that;

- The Hamlet owns a crusher
- The contractor owns an air track drill rig
- The contractor has previously been given a quarry permit for sufficient resources to complete the proposed works. The permit had a 1 year expiry date, and has since expired.
- There are other contractors (Canadrill) who would be willing to provide services to develop a granular resource for this project.

Based on the above discussions, the development of the granular resources to complete the proposed works is possible and there is little risk to the owner associated with the development of the resource. The costs estimates to develop the resource and supply and place the granular material are based on \$100/m^3 for rock excavation and \$40/m^3 for fill. Since most of the material will be obtained from a blast and crush operation, there is little cost savings when using on site cut (blast) material for fill material.

We have used a 30% expansion factor for on site cut when applied to fill.

The cost estimates for granular supply are summarized below.

Table 5.1: Cost Estimates

Item	Quantity (m^3)	Unit Cost	Total Cost
Access Road Cut	6,200	$100	$620,000
Access Road Fill (from cut)	8,100	$20	$16,200
Access Road Fill (borrow)	21,700	$40	$868,000
Lagoon Berms Cut	0	$100	0
Lagoon Fill (from cut)	0	$20	0
Lagoon Fill (borrow)	25,200	$40	$1,000,000
Total			**$2,500,000**

All estimated costs have an associated accuracy of ± 20.

As part of the geotechnical study, five granular sources were identified. All of these sources are found within the municipal boundary for the community. Granular resources developed within the municipal boundary, and that are used for municipal purposes are exempt from the Nunavut Environmental Impact Review process.

The identified granular sources contain high levels of fines. All sources will require processing (screening) to obtain granular material for the construction of the lagoon berms.

5.6.4 Culvert Design

As illustrated in **Drawings 102-104,** the construction of the road from the Hamlet to the proposed lagoon site will require the construction of several culverts. To determine the design parameters for the construction of the culverts, the peak water flow through the culvert was determined.

5.6.4.1 Method of Flood Estimation based on Rainfall Data

The rational method was used to estimate the peak flow for the culvert design. This method generally produces reliable results for small watersheds where the input parameters can be properly quantified. The equation used is:

$$Q = \frac{RIA}{360} \qquad [1]$$

Where,

Q = the design flow in m^3/s

R = a co-efficient that estimates the fraction of rainfall that runs off (the rest is assumed to sink into the ground or get stored in depressions and not appear at the point of concentration (POC) until after the peak flow has passed)

I = the average rainfall intensity (derived from a regional IDF curve for a 10 year return period) during the time of concentration (T_C) (**Appendix H**)

T_c = the time that it takes water to flow from the highest point in the watershed to the point of interest. T_c is also knows as the "response time" of the basin (**Appendix H**)

A = watershed area in hectares (Determined by Catchment Area Drawing in **Appendix H**).

For the Cape Dorset region, a set of Intensity-Duration Frequency (IDF) curves was available from Environment Canada (**Appendix H**). The data used to create the IDF curves was collected at the Cape Dorset Airport.

Practical difficulties are inherent when estimating design floods based on rainfall data. It is difficult to estimate T_c, the time of concentration, R, the fraction of the rainfall that runs off (rather than sinking into the ground) and I, the "design" rainfall intensity. For this reason, a factor of safety of 10% has been added to the design flow in addition to the following values to account for inaccuracies in the inputs.

Area inaccuracy 5% or 0.05 = 105% of calculated value
R inaccuracy 20% or 0.20 = 120% of estimated value
I inaccuracy 5% or 0.05 = 105% of value taken from IDF curve

With a design peak flow (Q_d) based on an estimated 10 year return peak flow (Q_{10}) established, the Manning's formula (**Equation 2**) was used to size the culvert diameter based on a velocity of water flow in the pipe of approximately 1.5 m/s:

$$Q = \frac{A\left(kr^{\frac{2}{3}}s^{\frac{1}{2}}\right)}{n} \qquad [2]$$

Where,

Q = Flow in m^3/s or ft^3/sec
k = units factor (1.0 metric) or (1.486 Imperial)
r = the hydraulic radius of the channel section
s = slope of channel in m/m or ft/ft
n = Mannings coefficient (equal to 0.013 for typical culvert material)

As show in **Appendix F**, the peak flow through the culvert was conservatively calculated to be **1.72 m³/s (60.7 ft³/sec)**. Using a design flow of **1.72 m³/s** and a slope of 0.18, a typical 1,200 mm (48") circular steel culvert will give a velocity of approximately 1.5 m/s.

5.7 P Lake Dimensions

P Lake will serve as a short retention lagoon. As per **Section 3** and **Drawing B** the volume of the Lake is approximately 11 667 m^3. The P Lake lagoon will offer treatment during a 2 week period in the fall, as the primary lagoon is discharged.

Table 5.2 shows the sewage retention time in P Lake for each volume of sewage produced by the community over the 20 year lifespan of the system.

Table 5.2: Sewage Retention Time in P Lake

Year	Annual Sewage Volume (m^3)	Discharge Time (days)	Retention time (days)
2006	59829	14	2.7
2007	61448	14	2.7
2008	63026	14	2.6
2009	64671	14	2.5
2010	66330	14	2.5
2011	68283	14	2.4
2012	70198	14	2.3
2013	71902	14	2.3
2014	73735	14	2.2
2015	75467	14	2.2
2016	77213	14	2.1
2017	79208	14	2.1
2018	81042	14	2.0
2019	83190	14	2.0
2020	85358	14	1.9
2021	86530	14	1.9
2022	88405	14	1.9
2023	90294	14	1.8
2024	92197	14	1.8
2025	94115	14	1.7
2026	96047	14	1.7

6 TREATMENT QUALITY

The lagoon treatment system will be designed to meet the following effluent criteria at the discharge point to Telik Inlet:

- 45 mg/L BOD_5
- 45 mg/L SS
- 10^4 Fecal Coliform / 100mL

Dillon has taken several measures to predict treatment quality and ensure that the lagoon treatment systems effluent discharged to Telik Inlet will meet the above criteria.

6.1 Annual Lagoon Kinetics

The level of treatment achieved by a lagoon system can be predicted using the following kinetic formula[1]:

$$\frac{C_e}{C_i} = e^{-Kt} \qquad [2]$$

Where,
C_e = Concentration of substrate (BOD_5) in lagoon effluent (mg/L)
C_i = Concentration of substrate (BOD_5) in lagoon influent (mg/L)
t = Residence time of sewage in lagoon (days)
K = kinetic rate constant for (days^{-1})

The kinetic rate constant, K, varies according to temperature:

$$K = K_{20}\theta^{T-\theta} \qquad [3]$$

Where,
K = BOD_5 kinetic rate constant (days^{-1})
K_{20} = BOD_5 kinetic rate constant (days^{-1}) for 20^0C
θ = temperature coefficient
T = temperature of lagoon contents in the critical or coldest winter months in degrees Celsius (^{0}C)

A typical value for θ is 1.06[2]. Although typical values for K_{20} range from 0.25 – 0.50 days^{-1}, a significantly lower value for K (0.10 days^{-1}) was assumed in this case, to be conservative and to account for the cold climate conditions. Using these assumed values the effluent quality from the constructed

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[1] Environment Canada Report EPS 3 NR 1. (1987) Cold Climate Sewage Lagoons. *Proceedings of the June 1985 Workshop, Winnipeg, Manitoba.* Appendix D-3.

[2] Metcalf and Eddy, Inc. (1991). *Wastewater Engineering: Treatment, Disposal and Reuse, 3rd Edition.* Toronto : McGraw-Hill Inc.

primary lagoon was predicted for a variety of conservative temperatures and retention times (Error! Reference source not found.). Although the lagoon will hold sewage for a year's time, the effective treatment time used in Error! Reference source not found. only accounts for the length of time sewage is completely thawed for treatment during the summer months. Since freeze-up can vary and occur anytime from September – November, a range of 70-90 days of treatment were analyzed. Winter treatment was assumed to be negligible in **Table 6.1**.

Table 6.1: Prediction of Effluent BOD using Lagoon Kinetics (Annual Retention Lagoon)

t (days)	K_{20} (days^{-1})	θ	T (°C)	K (days^{-1})	C_e/C_i	C_i (mg/L)	C_e (mg/L)
90	0.1	1.06	3	0.037	0.0354	625	22
90	0.1	1.06	4	0.039	0.0289	625	18
90	0.1	1.06	5	0.042	0.0234	625	15
90	0.1	1.06	6	0.044	0.0187	625	12
90	0.1	1.06	7	0.047	0.0147	625	9
80	0.1	1.06	3	0.037	0.0513	625	32
80	0.1	1.06	4	0.039	0.0429	625	27
80	0.1	1.06	5	0.042	0.0355	625	22
80	0.1	1.06	6	0.044	0.0291	625	18
80	0.1	1.06	7	0.047	0.0235	625	15
70	0.1	1.06	3	0.037	0.0743	625	46
70	0.1	1.06	4	0.039	0.0636	625	40
70	0.1	1.06	5	0.042	0.0539	625	34
70	0.1	1.06	6	0.044	0.0452	625	28
70	0.1	1.06	7	0.047	0.0376	625	23

Based on the above data, the BOD_5 of the effluent discharged from the primary lagoon will range from 9 mg/L (90 day treatment period, 7°C) to 46 mg/L (70 day treatment period, 3°C). The short detention lagoon (P Lake), wetlands area and outfall to Telik Inlet will reduce this value even further to meet the effluent discharge criteria.

6.2 P Lake Lagoon Kinetics

The same kinetics used in **Section 5.1** can determine the amount of treatment that the P Lake lagoon will offer during the 14 day annual discharge of the primary lagoon. **Table 6.2** illustrates the effluent BOD_5 values determined for P Lake. These values were determined using equations **[2]** and **[3]** and the following parameters:

- The range of retention times determined in

Table 5.2

- The range of BOD_5 influent values (BOD_5 effluent values from primary lagoon) determined in **Table 6.1**
- $\theta = 1.06$
- $K = 0.1$ days^{-1}

Table 6.2: Prediction of Effluent BOD using Lagoon Kinetics (P Lake Lagoon)

t (days)	K_{20} (days^{-1})	θ	T (°C)	K (days^{-1})	C_e/C_i	C_i (mg/L)	C_e (mg/L)
3.5	0.1	1.06	5	0.042	0.8641	50	43
3.5	0.1	1.06	5	0.042	0.8641	40	35
3.5	0.1	1.06	5	0.042	0.8641	30	26
3.5	0.1	1.06	5	0.042	0.8641	20	17
3.5	0.1	1.06	5	0.042	0.8641	10	9
2.5	0.1	1.06	5	0.042	0.9009	50	45
2.5	0.1	1.06	5	0.042	0.9009	40	36
2.5	0.1	1.06	5	0.042	0.9009	30	27
2.5	0.1	1.06	5	0.042	0.9009	20	18
2.5	0.1	1.06	5	0.042	0.9009	10	9
1.5	0.1	1.06	5	0.042	0.9393	50	47
1.5	0.1	1.06	5	0.042	0.9393	40	38
1.5	0.1	1.06	5	0.042	0.9393	30	28
1.5	0.1	1.06	5	0.042	0.9393	20	19
1.5	0.1	1.06	5	0.042	0.9393	10	9

Although equation **[2]** is useful for a first look at the potential performance of the annual retention lagoon and P Lake lagoon, the equation has not been tested for it's effectiveness in modeling Northern lagoon systems. Heinke *et al*[3] studied the effectiveness of lagoon sewage treatment in the North, and tabulated predicted lagoon treatment for Northern lagoon systems (**Table 6.3**).

Table 6.3: Expected Performance of Lagoon Treatment of Municipal Type Wastewaters for Lagoon Systems

Parameter	Short Detention (% Reduction)	Long Detention (% Reduction)
Summer		
BOD_5	40	80
Suspended Solids	50	80
Fecal Coliform	60	99.9
Winter		
BOD_5	40	50
Suspended Solids	60	50
Fecal Coliform	70	80

[3] Heinke, G.W., Smith, D. W., Finch, G.R. (1991) Guidelines for the planning and design of wastewater lagoon systems in cold climates. *Canadian Journal of Civil Engineering*, 18(4) 556-567.

Based on this data, the primary annual detention lagoon will reduce the influent 625 mg/L BOD_5 to 86 mg/L from both winter and summer treatment. Following this treatment, the secondary short detention lagoon (P Lake) will reduce the influent 86 mg/L to 51 mg/L. The wetland treatment and outfall for Telik Inlet will offer additional treatment to meet the effluent discharge criteria.

6.3 Fecal Coliform Reduction

The reduction of fecal coliforms (FC) can also be predicted using **Table 6.3**. The average generation of FC in domestic sewage is $2x10^9$ FC per person per day[4]. Using this value and the predicted sewage volume generation from **Table 2.1**, the average fecal coliform concentration in the P Lake Lagoon system was determined in **Table 6.4.**

Table 6.4: Reduction of Fecal Coliform from Lagoon Treatment System

Year	Population	Fecal Coliorm		Sewage Volume	Fecal Coliform			
					Raw Influent	Annual Lagoon Effluent (99.9% Reduction)	P Lake Lagoon Effluent (40% Reduction)	P Lake Lagoon Effluent
		(FC/p/d)	(FC/d)	(L/d)	(FC/L)	(FC/L)	(FC/L)	(FC/100mL)
2006	1382	2.0E+09	2.8E+12	1.6E+05	1.7E+07	1.7E+04	3.4E+03	3.4E+02
2007	1412	2.0E+09	2.8E+12	1.7E+05	1.7E+07	1.7E+04	3.4E+03	3.4E+02
2008	1441	2.0E+09	2.9E+12	1.7E+05	1.7E+07	1.7E+04	3.3E+03	3.3E+02
2009	1471	2.0E+09	2.9E+12	1.8E+05	1.7E+07	1.7E+04	3.3E+03	3.3E+02
2010	1501	2.0E+09	3.0E+12	1.8E+05	1.7E+07	1.7E+04	3.3E+03	3.3E+02
2011	1536	2.0E+09	3.1E+12	1.9E+05	1.6E+07	1.6E+04	3.3E+03	3.3E+02
2012	1570	2.0E+09	3.1E+12	1.9E+05	1.6E+07	1.6E+04	3.3E+03	3.3E+02
2013	1600	2.0E+09	3.2E+12	2.0E+05	1.6E+07	1.6E+04	3.2E+03	3.2E+02
2014	1632	2.0E+09	3.3E+12	2.0E+05	1.6E+07	1.6E+04	3.2E+03	3.2E+02
2015	1662	2.0E+09	3.3E+12	2.1E+05	1.6E+07	1.6E+04	3.2E+03	3.2E+02
2016	1692	2.0E+09	3.4E+12	2.1E+05	1.6E+07	1.6E+04	3.2E+03	3.2E+02
2017	1726	2.0E+09	3.5E+12	2.2E+05	1.6E+07	1.6E+04	3.2E+03	3.2E+02
2018	1757	2.0E+09	3.5E+12	2.2E+05	1.6E+07	1.6E+04	3.2E+03	3.2E+02
2019	1793	2.0E+09	3.6E+12	2.3E+05	1.6E+07	1.6E+04	3.1E+03	3.1E+02
2020	1829	2.0E+09	3.7E+12	2.3E+05	1.6E+07	1.6E+04	3.1E+03	3.1E+02
2021	1848	2.0E+09	3.7E+12	2.4E+05	1.6E+07	1.6E+04	3.1E+03	3.1E+02
2022	1879	2.0E+09	3.8E+12	2.4E+05	1.6E+07	1.6E+04	3.1E+03	3.1E+02
2023	1910	2.0E+09	3.8E+12	2.5E+05	1.5E+07	1.5E+04	3.1E+03	3.1E+02
2024	1941	2.0E+09	3.9E+12	2.5E+05	1.5E+07	1.5E+04	3.1E+03	3.1E+02
2025	1971	2.0E+09	3.9E+12	2.6E+05	1.5E+07	1.5E+04	3.1E+03	3.1E+02
2026	2002	2.0E+09	4.0E+12	2.6E+05	1.5E+07	1.5E+04	3.0E+03	3.0E+02

The predicted concentration of FC/100mL of P Lake Lagoon effluent is far beneath the design criteria of 10^4 FC/100mL.

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[4] Metcalf and Eddy, Inc. (1991). *Wastewater Engineering: Treatment, Disposal and Reuse, 3rd Edition.* Toronto : McGraw-Hill Inc.

6.4 Wetland Sewage Treatment - Nutrient Removal

Lagoon treatment systems often have difficultly reducing nutrients (nitrogen and phosphorus) to regulated levels. The effluent from the P Lake lagoon will be discharged to a wetlands area. Northern lagoon systems that run into wetlands areas can generally meet nutrient levels. Wetlands remove nutrients by a variety of natural processes: plant uptake, filtration, sorption, flocculation, sedimentation and biological degradation.

6.5 Suggested Design Criteria

Design criteria have been developed to ensure that conditions in a lagoon treatment system are sufficient for proper sewage treatment. These design criteria take the following details into consideration:

- Sunlight for disinfection of microorganisms
- Wind for sewage aeration
- Odour control
- Sufficient Treatment of BOD_5

For sewage lagoons, the province of Manitoba[5] recommends not exceeding an organic loading of 56 kg/ha/d. Based on equation **[1]** and a BOD_5 of 625 mg/L, the maximum organic loading (in year 20) will be 53 kg/ha/d (**Appendix I**).

6.6 Functional Lagoon Systems in the North

To support the proposed lagoon treatment system design, Dillon will draw design parameters from existing sewage treatment lagoon that are functional and achieving the proposed effluent criteria for Cape Dorset, NU.

6.6.1 Nunavut

Dillon consulted with Mr. Constantine Bodykevich, Water Resources Officer, Indian and Northern Affairs Canada, Nunavut District Office. Based on his annual inspections of existing lagoons systems in Nunavut, Mr. Bodykevich offered the following advice for constructing a functional sewage lagoon system in Cape Dorset:

- Construct an annual retention lagoon as opposed to a seepage lagoon
- Construct lagoon berms with fine grade material that won't allow for the seepage of small particulate matter, i.e. shale, not course gravel
- Construct berms will a 3 m width at the top and a side slope with less than 50% grade

Dillon's preliminary design meets and/or exceeds these recommendations.

[5] Province of Manitoba. (1985) Design Objectives for Standard Sewage Lagoons.

6.6.2 Fort Liard

Dillon has been involved with various stages of the design of the sewage lagoon system in Fort Liard, NWT. Although the system in Fort Liard consists of 3 cells in sequence, the first 2 cells have a retention time of approximately 1 year and are comparable to a single cell with annual retention, proposed for Cape Dorset. A sample taken from the second cell on July 16, 2002 showed the following concentrations:

- BOD_5: 16 mg/L
- FC: 8 CFU/100 mL
- TSS: 41 mg/L

Although the climate in Fort Liard is warmer than in Cape Dorset, this sample was taken mid-way through the treatment season, after approximately 2.5 months of treatment. This is approximately the length of the treatment season in Cape Dorset. The results indicate that an annual retention lagoon is capable of treating sewage to the required guidelines.

6.7 Additional Sewage Treatment

Looking beyond the 20 year design horizon, there are opportunities to expand the lagoon system on the site, or to enhance the treatment system to provide additional future capacity. The proposed lagoon treatment system may be upgraded to enhance treatment and prolong the life of the system using a number of technological means. A long tem monitoring program is recommended to determine what maybe undertaken in the future to extend the 20 year design life of the facility. These are discussed below.

6.7.1 Aeration

Should additional treatment be required by the lagoon treatment system, the annual retention lagoon can be retro-fitted with aerators during the summer months. Aeration enhances the level of treatment in several ways:

- Completely mixes the system
- Increases temperature
- Addition of dissolved oxygen

All of the above factors lead to an increased rate of BOD_5 degradation. Depending on the detention time of the lagoon, the effluent from an aerated lagoon contains about one-third to one-half the value of BOD_5 from that of a non-aerated system.

Aeration is best practiced in an on/off operation. Approximately one-month prior to annual discharged the sewage can be aerated to advance treatment of the system. Two weeks prior to annual discharge, the aerators can be turned off, allowing settling of solids and removal of microorganisms.

6.7.2 Solar Aerators

The client has expressed interest in using a solar aerator in the sewage lagoon, to increase the rate of treatment. However, the effectiveness of this is uncertain. Cooler water temperatures hold more dissolved oxygen, and decrease the metabolic rate of microorganisms. Thus, the rate of BOD_5 degradation may be temperature-limiting, rather than oxygen-limiting.

The rate of BOD degradation in a lagoon system assumes that there is sufficient oxygen in the system. Adding aerators would not substantially increase the treatment rate above what is given by this equation. In the feasibility report, values of k were estimated to be between 0.037 – 0.047 d^{-1}. Using an influent concentration of 625 mg/L and an effluent concentration of 45 mg/L (as required by the Water Board), the treatment time would range between 56 and 71 days. To increase this rate, the water temperature would need to be increased.

6.7.2.1 Feasibility of Aerators in Cape Dorset

SolarBee® aerators have 3-80 watt solar panels, each producing 68 watts of usable output, for a total of 102 usable watts. Low speed, surface aerators typically have an oxygen transfer rate of 0.7 – 1.5 kgO_2/KWh.

The oxygen demand of 7 months of sewage (November – May), with an influent BOD_5 concentration of 625 mg/L, would range between 32719 kg in 2006 to 52525 kg is 2026. This is the total oxygen demand, assuming a ratio of $BOD_u : BOD_5$ of 1.5, where BOD_u is the ultimate oxygen demand. Over a 60 day period, the daily oxygen demand would be 875 kg O_2/d.

According to H2O Logics Inc., a SolarBee® distributor, the oxygen transfer rate for one of their machines is 300 lbs O_2/acre/day (or 336 kg O_2/ha/d). Also according to H2O Logic, the natural reaeration rate of lakes is 50 lbs O_2/acre/day (or 56 kg O_2/ha/d). With a lagoon size of 2.4 ha (185 m x 132 m), the natural surface reaeration is 134 kg O_2/d, considerably less than the 875 kg O_2/d predicted above.

6.7.3 Treatment Beyond 20 years

The proposed 2-celled lagoon treatment system is designed for a 20 year use; however, to maximize the utilization of the system, the treatment area may be expanded to accommodate sewage beyond 2026. **Table 6.5** lists the predicted yearly volume of sewage for years 20-40 of the systems use.