Village of Wawanesa – Lagoon Functional Design Report (Site #2)

Final Report for EAP

June 2014

PROVINCE OF MANITOBA
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2014-06-23
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No. 1789 Date: 2014-06-24

Village of Wawanesa – Lagoon Functional Design
Final Report for EAP (Site #2)

Manitoba Water Services Board

13-8212-7000

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Submitted by
Dillon Consulting Limited

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1 INTRODUCTION

The Village of Wawanesa (Wawanesa) is located approximately 40 km southeast of Brandon, Manitoba and is surrounded on three sides by the Souris River. Circa 1975, a lagoon was constructed to treat Wawanesa's wastewater. It was located northwest of the village, adjacent to the river. The river bank collapsed, breaching the lagoon and collapsing the forcemain leading to the lagoon. As part of an insurance settlement, a Rotating Biological Contactor (RBC) mechanical treatment plant was constructed in the town, circa 1976.

In 2011, Dillon Consulting Limited (Dillon) was contracted by Manitoba Water Services Board (MWSB) to assess Wawanesa's water and wastewater treatment systems. The resulting Water and Wastewater Treatment Assessment Final Report prepared by Dillon indicated that the wastewater treatment plant (WWTP) was in need of many repairs and upgrades. Although the volume and strength of wastewater being treated was within the system's theoretical treatment capacity, effluent tests showed that the WWTP was not treating the wastewater sufficiently to meet the standards required by its environmental licence. Dillon estimated the probable cost of improvements to be approximately $1.0M. The water treatment system also required improvements, which were considered a higher priority. Hence, no substantive upgrades were undertaken to the WWTP.

In March 2013, the WWTP experienced a mechanical failure: the RBC shaft had sheared off. Dillon was retained to provide advisory services and was able to implement a temporary treatment system. According to the Wawanesa WWTP Upgrade Options report completed by Dillon in May 2013, the point at which the shaft had sheared made repair (re-welding) challenging. Taking into consideration both the required shaft repair and the $1.0M in other improvements needed to render the WWTP functional, Wawanesa decided to replace the WWTP with a lagoon system. In February 2014, Dillon completed the functional design for a new lagoon system to be located southwest of the Village in the Rural Municipality (RM) of Oakland. However, the RM of Oakland did not support the chosen location, and in April 2014 Dillon was contracted to revise the functional design and environmental licence application for a new site located southeast of the Village, as shown in **Figure 1** below.

Figure 1: Wawanesa Aerial Image (Google Earth, 2014)

2 EXISTING SYSTEMS

All of the homes in Wawanesa are connected to the central gravity wastewater collection system, which flows to a single lift station. The lift station pumps wastewater to the mechanical treatment plant.

2.1 Collection System

Dillon did not undertake an assessment of Wawanesa's wastewater collection system. Any maintenance or upgrades required to the existing sewers are considered outside the scope of this design.

2.2 Lift Station

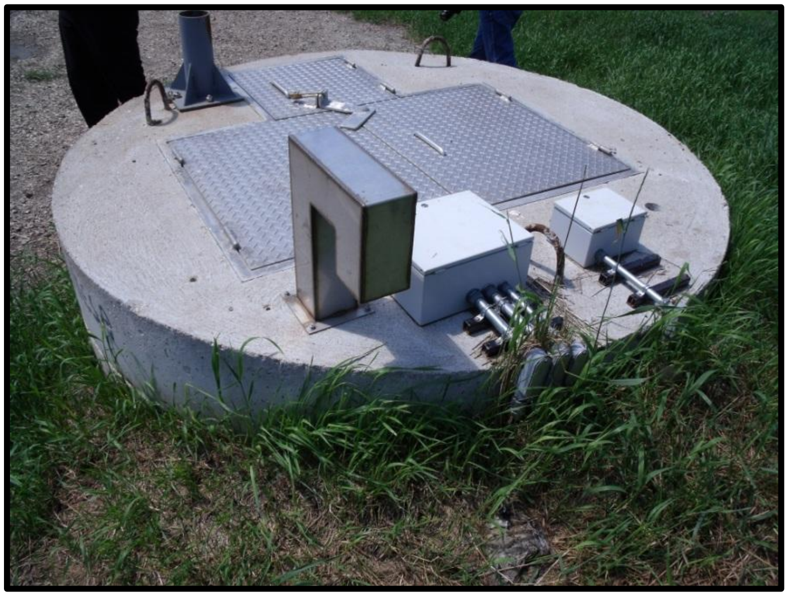


Figure 2: Wawanesa Lift Station Exterior

The existing lift station was constructed by Contec Projects Limited (Contec) in 2007 to replace the sewage pumping station that dated back to Wawanesa's first lagoon (pre-1975). According to information provided by Contec, the lift station consists of a cylindrical concrete barrel wet well with two submersible pumps operating at approximately 3 hp. The lift station is 6.5 m deep and has a 1.8 m inner diameter. Wastewater from the village's sewer system is piped through two lines to a manhole adjacent to the old pumping station. It then flows by gravity through approximately 3 m of 250 mm SDR-35 PVC pipe to the new lift station. There is a second influent line into the lift station: a 100 mm sanitary sewer line from the washroom in the nearby WWTP. Currently the lift station pumps the wastewater out to the WWTP through approximately 21 m of 100 mm diameter SDR 26 Series 160 PVC pressure pipe. Dillon staff who viewed the lift station in 2011 report that the concrete barrel of the lift station appears to be in good condition.

The lift station draws its power from the old pumping station, where the Centex PLC control panel is located. According to the 2011 assessment, service at the WWTP is single-phase, 120/240V, 200A.

2.3 Wastewater Treatment Plant

After the failure of the sole RBC treatment train at the Wawanesa WWTP in March 2013, Dillon staff were able to put in place a temporary aeration and sludge return system. This system provides partial treatment, reducing biological oxygen demand (BOD) prior to discharging the wastewater into the Souris River. This temporary system is not a long-term solution and does not produce wastewater of a high enough quality to meet Wawanesa's environmental licence conditions.

As discussed previously, Wawanesa and MWSB have opted to replace the treatment plant with a new lagoon. Manitoba Conservation and Water Stewardship (CWS) is aware of the situation and has indicated they will expedite the review of Wawanesa's new lagoon licence application, where possible.

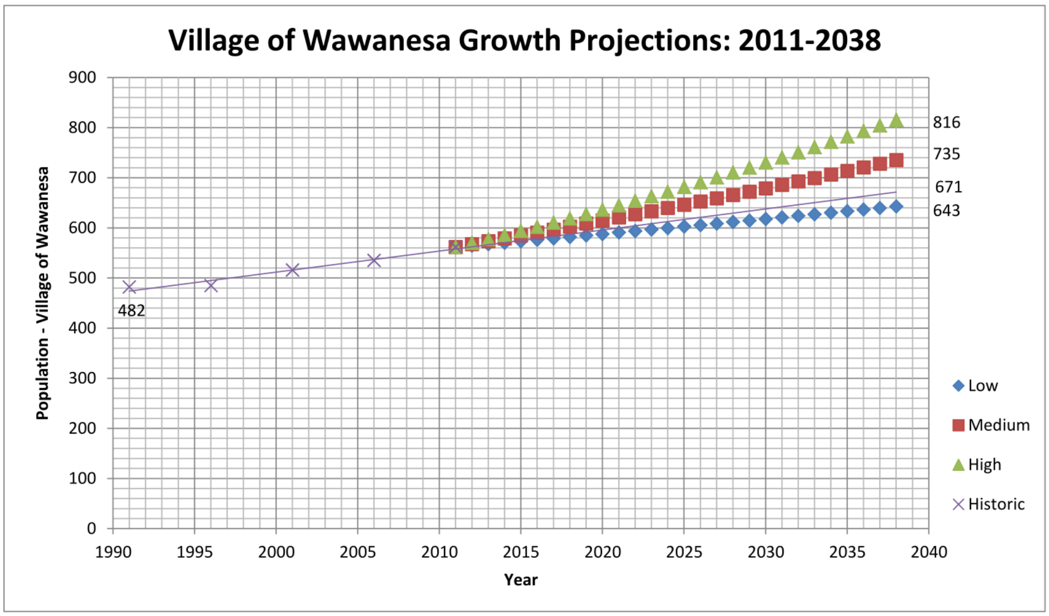
3 WASTEWATER GENERATION

To determine the size of Wawanesa's new lagoon and the extent of upgrades necessary to existing infrastructure, Wawanesa's current and future wastewater generation (flow and strength) was estimated.

3.1 Population

According to the most recent Statistics Canada Census (2011), the population of Wawanesa is 562 people. Statistic Canada's population data from 1991 through 2011 was reviewed to generate an estimate for potential future growth rate in Wawanesa of 0.77%. Alternately, Manitoba Bureau of Statistics population and demographics projections, generated in 2010, estimate a 20-year average annual growth rate of 1.39% for the Southwest Region, of which Wawanesa is a part.

The Village of Wawanesa and MWSB were presented with four population projections at the project initiation meeting on September 24, 2013 (**Figure 3**, below). After consideration for both historical growth patterns and future possibilities, the medium growth scenario of 1% per year was selected. Hence, at the 25-year design horizon (2038), the population of Wawanesa is projected to be 735 people.



Scenario	Rate	Source
Low	0.50%	
Medium	1.00%	
High	1.39%	*Manitoba Bureau of Statistics (2010) - Population and Demographics Projections 2008-2031*
Historic	0.77%	*Statistics Canada Census Population Counts (2012) - 1991-2011 Trend*

Figure 3: Wawanesa Population Projections (2011-2038)

Consideration was also given to the neighbouring rural populations. At the Village of Wawanesa Council meeting on December 9, 2013, by Resolution No. 155/13 the council determined to include an allowance for rural contributions in the design of the new lagoon. That allowance was set as follows:

- 1/3 of the residences in the RM of Oakland will truck septage to the lagoon;
- 1/10 of the residences in the RM of South Cypress will truck septage to the lagoon;
- 1/10 of the residences in the RM of Riverside will truck septage to the lagoon; and,
- Any population growth in the above RMs would be accommodated by other wastewater systems.

The number of residences in each of these RMs as stated in the 2011 Statistics Canada Census is summarized in **Table 1**, below.

Table 1: 2011 Census Data for Neighbouring Rural Municipalities

Rural Municipality	Population	Dwellings Occupied by Usual Residents	Dwellings To be Serviced by the Wawanesa Lagoon
Oakland	1,056	371	124
South Cypress	838	210	21
Riverside	780	271	27
		Total:	**172**

Hence, the Wawanesa lagoon is designed to accept trucked septage from 172 rural dwellings. According to the 2011 Statistics Canada Census, the average household size for these three RMs is three persons per residence. Therefore, the Wawanesa lagoon can accept trucked septage from 516 people.

3.2 Per Capita Sewage Generation

In the absence of actual metered wastewater flow data, it is typical to estimate wastewater flows based on the per capita water consumption rate. According to the 2011 Wawanesa Public Water System Water Assessment report by Genivar, the average water demand from 2007-2009 was 179 m^3/d. This is consistent with the 2010 and 2011 water supply pumping records provided to Dillon. On average, between 2007 and 2011, Wawanesa drew 327 L/p/d from its groundwater wells. As agreed upon at the project initiation meeting, wastewater generation has been conservatively estimated at 80% of water supply, or **262 L/p/d**. The strength of the wastewater is taken to be **0.085 kgBOD$_5$/p/d** (Metcalf and Eddy, 2002). Hence, in 2038 Wawanesa will generate **62 kgBOD$_5$/d** and **193 m^3/d** of wastewater.

Table 2: Wawanesa Projected Wastewater Loadings (2013-2038)

Calendar Year	Design Year	Total Population	Average Hydraulic Loading m^3/d	Average Organic Loading kgBOD$_5$/d
2013	0	573	150	49
2018	5	603	158	51
2028	15	666	174	57
2038	25	735	193	62

It is worth noting that this wastewater generation, while based on residential population data, is assumed to include all commercial and institutional contributions as well. This is a reasonable assumption based on the conservativeness of the per capita wastewater generation rate selected and the current absence of wet industries in Wawanesa.

3.3 Rural Septage Contributions

Most rural residences employ private on-site wastewater disposal systems, such as septic fields or sewage ejectors. In a typical on-site system configuration, wastewater flows to a one or two-chamber septic tank. Solids, also called sludge or septage, settle to the bottom of the tank. Scum floats to the top of the tank. The clarified wastewater in the middle of the tank is either pumped or flows by gravity to a disposal field on the homeowner's property. Over time, the solids build up in the bottom of the tank, decreasing the available storage space in the tank and stressing the disposal field. The septic tanks require regular removal of the settled solids to keep the on-site systems functioning adequately.

To estimate the quantity and strength of septage contributions from the 172 residences in the RMs of Oakland, South Cypress, and Riverside, the following assumptions were made:

- Each rural residence pumps out its septic tank once every 16 months to remove accumulated sludge from the bottom of the tank (Wawanesa Project Initiation Meeting);
- Sludge is generated at a rate of 60 L/cap/yr (Province of Manitoba, *Homeowner's Manual for Onsite Wastewater Management Systems*);
- Sludge has a strength of 7 $kgBOD_5/m^3$ (*Ten States Recommended Standards for Wastewater Facilities*);
- When a septic tank pump-out occurs, both the accumulated sludge and the liquid sewage being stored in the tank above the sludge layer are removed and hauled to the lagoon;
- On average, a total volume of 3,000 L of combined sewage and sludge is pumped from each tank; and,
- The sewage portion of the pump-out has an average strength of 0.38 $kgBOD_5/m^3$ (Metcalf and Eddy, *Wastewater Engineering Treatment and Reuse 4E*).

This leads to an estimate for rural wastewater contributions to the lagoon of **2.0 $kgBOD_5$/residence/yr** and **2.25 m^3/residence/yr**. Septage is only permitted to be trucked to a lagoon between June 1 and October 15 each year. Typically most trips (assumed 70%) are made in the 45-day period from September 1 to October 15. This does not have a substantial hydraulic impact on the lagoon, due to the low volumes involved (a total annual volume of **390 m^3**). However, due to the high strength of trucked septage, the clustering of truck trips does organically impact the lagoon. To account for this effect, the biological oxygen demand from rural septage is taken to be the daily average during the 45-day "fall peak" period, not the annual average. In Wawanesa's case, that means the wastewater contribution from rural dwellings is estimated to be **6 $kgBOD_5/d$**.

4 DESIGN PARAMETERS

Design parameters are determined by combining the projected wastewater generation rates (**Section 3**) with anticipated regulatory requirements and Wawanesa's desired outcomes.

4.1 Lagoon

The proposed Wawanesa lagoon will accept piped wastewater from the Village and truck-hauled septage from 172 rural residences. Combining the 25-year (2038) wastewater generation estimates from **Section 3**, the lagoon loading design parameters are **194 m^3/d** and **68 kgBOD$_5$/d**[1]. Based on a pre-consultation meeting with CWS on October 10, 2013, Dillon does not anticipate any out of the ordinary regulatory requirements for the Wawanesa lagoon. Hence, the lagoon will need to achieve the following standard Manitoba CWS design criteria:

- Store wastewater for a minimum of 230 days (between November 1 and June 15 each year);
- Treat wastewater using a maximum organic loading rate of 56 kgBOD$_5$/hectare of primary cell surface area, measured at mid-liquid depth; and,
- Implement a demonstrated nutrient reduction strategy to lower phosphorus levels in the lagoon effluent.

Therefore, the required hydraulic storage capacity of the Wawanesa lagoon is approximately **44,500 m^3** (194 x 230). The required surface area of the primary cell is approximately 1.21 ha (68 / 56) or **12,100 m^2**.

Strategies for reducing phosphorus in the effluent were outlined and compared separately by Dillon in the *Wawanesa Lagoon Nutrient Reduction Strategy Options – Final Letter Report*. Although trickle discharge was Wawanesa's preferred strategy, after discussing the implications further with the regulators, Dillon understands that to employ trickle discharge as the sole strategy for nutrient reduction the following two conditions must be met:

1. Discharge from the lagoon should occur only when vegetation is growing and hence able to take up nutrients from the soil. As the average "first fall frost" for the region is in September, this implies a minimum of 260-day winter storage would be required; and,
2. The lagoon system must have sufficient capacity to store influent during the entire secondary cell isolation, quality testing, and trickle discharge period. As the primary and secondary cells are hydraulically connected prior to the isolation start period, system storage capacity cannot simply be calculated using the standard method (described above). Equalization of lagoon levels must be considered and used to determine "effective" storage capacity.

The above two conditions have significant impacts on the sizing of the lagoon. Dillon calculated that selection of a four-week trickle discharge strategy would result in a lagoon footprint increase of approximately 30%. Given the cost implications of this sizing increase, Wawanesa has chosen instead to select chemical addition (alum) as the method of reducing phosphorus levels in the lagoon effluent when necessary.

[1] When a lagoon accepts truck-hauled wastewater, the parameters typically selected for the lagoon design are average <u>annual</u> hydraulic load and average <u>45-day peak</u> organic load.

4.2 Forcemain and Lift Station

A new forcemain is required to convey wastewater from the existing lift station to the proposed lagoon. To convert the average daily design flow estimated in **Section 3** into a peak hourly flow (the design parameter used for wastewater collection systems), a conservative Harmon peaking factor of four was assumed based on Wawanesa's population. The design peak hourly flow rate is hence **9 L/s**. The forcemain is also sized to provide a minimum velocity of **0.9 – 1.4 m/s** through the pipe (scouring velocity) to prevent solids build-up in the pipe.

The lift station pumps are selected to provide enough energy (called "head") to overcome the elevation change between the existing lift station and the proposed lagoon. As further detailed in **Section 5.2**, Dillon conducted two preliminary site surveys in November 2013 and April 2014. Ground elevation at the lift station was measured, and a topographic survey of the proposed lagoon site surface was conducted. The elevation of the bottom of the lift station is calculated by subtracting the depth of the lift station from the ground measurement. The topographic survey is used to set the elevation of the lagoon floor, which then gives the elevation of the maximum liquid level in the lagoon (1.5 m above the floor). The pumps are sized to overcome a maximum static head equal to the difference between the maximum liquid level in the lagoon and the bottom of the lift station. This calculation is conducted in **Section 5.2**.

5 SITE INVESTIGATIONS

The proposed lagoon location is the SE ¼ 25-7-17 W. The property is currently agricultural land; the owner has agreed to sell a portion of the land to Wawanesa to use for the development of their municipal lagoon. Wawanesa is preparing a formal offer to purchase the proposed lagoon site as shown on Plan M-1 in **Appendix A**.

To assist in determining the siting for the lagoon on this property, Dillon conducted a soil sampling program and a topographic survey of the proposed lagoon site in April 2014.

5.1 Soils Investigation

On April 22, 2014, Dillon staff and Paddock Drilling Ltd, drilled eight boreholes to depths of between 4.6 to 6.1 m at the locations shown on sheet M-1 in **Appendix A**. Soils were visually characterized and water levels in the boreholes at the end of the day were noted. All boreholes were backfilled with bentonite. Grab samples and Shelby tube samples were taken from selected boreholes at a range of depths. Shelby tube samples were submitted for laboratory analysis to determine hydraulic conductivity of the soils. As hydraulic conductivity may be correlated to particle size and plasticity, grab samples were submitted for particle size analysis, water content, and Atterberg limits (plasticity) to assist in characterizing and identifying soils that may be suitable for use in the construction of a soil liner. Detailed borehole logs are attached in **Appendix B**, and the laboratory results are included as **Appendix C.**

Figure 4: Borehole TH3 Hydraulic Conductivity Sample (approximately 2.0 m below ground level)

The soils are relatively consistent at the proposed site. Surficial (top) soil ranges in thickness from 0.13 m to 0.23 m. Below the surficial soils is a medium plasticity clay formation with variable silt content. In the two western-most boreholes (TH7 and TH8), this clay was the sole formation observed beneath the topsoil to a maximum depth of 4.8 m below the surface. At the other six testholes, the clay layer was underlain with a saturated (water-bearing) silty sand "lens" of between 0.5 m and 1.5 m in thickness[2]. In general, this layer was followed by another medium plasticity clay formation similar to the upper clay layer. The exception was at TH5, where the sand was underlain by silt to the bottom of the borehole.

Water was observed in three boreholes approximately two hours after they were initially drilled. Water was observed at depths of 2.6 m below ground level (TH1), 3.2 m below ground level (TH6) and 3.6 m below ground level (TH2).

[2] Note that in one borehole (TH6), the silty sand layer was detected near the bottom of the borehole and as such its thickness was not determined.

5.1.1 Results and Conclusions

Three grab samples and two Shelby tube samples from approximately 1.2 to 2.3 m below ground level were submitted to Stantec's geotechnical laboratory (previously National Testing Laboratories Limited) for analysis. A fourth grab sample taken from approximately 4.0 m below ground level at TH5 was also submitted for analysis. The laboratory results confirmed the field observations that the sample had significantly higher silt content than the shallower soils observed at other boreholes. These results indicate that these deeper soils would not be suitable for use in the construction of a soil liner without an admixture (e.g., bentonite).

The laboratory results classified all the upper clay layer samples as low to medium plasticity, with between 30 – 50% clay content. Hydraulic conductivity tests were conducted on the Shelby tube samples; one of which was compacted (remoulded) prior to testing. The undisturbed Shelby tube sample had the lowest plasticity (plasticity index 17), and its hydraulic conductivity was found to be 2.1 x 10^{-7} cm/s. This exceeds Manitoba Conservation's maximum hydraulic conductivity for a natural (in situ) soil liner (10^{-7} cm/s). The second Shelby tube sample was compacted prior to testing, and its hydraulic conductivity was found to be 9.1 x 10^{-9} cm/s, which is better than required by the regulators. Hence, Dillon concludes that the soil at the proposed lagoon site is not suitable for constructing an in situ soil liner but is suitable for a compacted soil liner, otherwise known as a constructed liner.

Dillon recommends that the base (floor) of the lagoon be set at a minimum elevation of 377.8 m above sea level (see **Table 3**) to ensure the 1 m thick liner beneath the lagoon floor is placed above the highest point at which water or saturated sand was observed during the soils investigation. Current best practice is to construct the base of the lagoon at least 1 m higher than the local groundwater table. Although the water observed in the boreholes during the soils investigation likely came from a localized sand lens, maintaining a 1 m buffer between the lagoon and the possible water table is important. Additionally, constructing/excavating a lagoon in saturated soils is challenging and would likely result in higher construction costs.

5.1.2 Alternatives

Where suitable soils exist or are available, a typical alternative to an in situ soil liner is a constructed soil liner. A constructed soil liner consists of excavation and manual compaction of the existing soils on-site by an experienced contractor. The manual compaction (with a sheep's-foot roller) increases the density and decreases the hydraulic conductivity of the soil. If the soil quality varies on site and/or the contractor's compactive effort does not achieve the required hydraulic conductivity continuously over all surfaces of the lagoon, the soil liner may not pass CWS compliance testing. However, as discussed previously the laboratory results indicate the soils are suitable for use in the construction of a constructed soil liner and achieving the regulatory standards is likely. Constructed liners are substantively less costly than the alternatives.

Three alternatives to a constructed liner were explored by Dillon:

- a synthetic liner;
- a bentonite mix liner; and,
- a bentonite mat liner.

Synthetic liners (HDPE) are vulnerable to tearing and require the installation of a gas relief system to alleviate (vent) biological gas pressure from below the liner. The preliminary estimate of capital costs for a synthetic liner and its associated works for a lagoon this size is $1M.

The second alternative is a bentonite mix liner. Bentonite is a type of industrially used absorbent clay that expands when wet, bonding with existing soils and becoming a groundwater barrier as it dries. It is the material that is used by drillers to seal and decommission boreholes or groundwater wells and has been used by Dillon in the past to repair soil lagoon liners. Bentonite can be applied as a slurry or in powder/pellet form. To use a bentonite mix liner at Wawanesa, existing clay would need to be excavated, mixed with bentonite, re-placed, and compacted to form the lagoon liner; a more labour intensive and expensive process than a typical constructed lagoon liner. If the bentonite is applied in powder or pellet form (more economical than applying in slurry form), it would need to be hydrated during the mixing process. Both the mixing and hydration processes can be challenging to implement consistently. Hence, the licensing process will likely be scrutinized by CWS more than if a synthetic or constructed liner was selected. CWS provided an initial opinion that even with bentonite addition, CWS would not permit a reduction in the 1 m liner thickness requirement, so bentonite addition would not save on excavation or construction costs relative to a constructed liner. Lastly, bentonite is an expensive material that would need to be imported, typically from the US (Wyoming). Considering the compressed project timeline, limited regulatory precedent, material costs, and probability of a typical constructed liner achieving regulatory standards, Dillon does not recommend a bentonite mix liner for the new Wawanesa lagoon.

The third option is to employ a bentonite mat. This liner system utilizes a pre-fabricated product containing bentonite that has been woven into a geotextile fabric. Its installation is significantly less labour intensive and more controlled than with a bentonite mix liner. However, CWS has concerns with leakage at the seams of the bentonite mats and the inability to detect a tear or hole in the mat should one occur. According to CWS, there have not been any recent successful municipal bentonite mat applications in Manitoba. While Dillon does believe this is a technologically sound option that has been used in other jurisdictions, in the interest of an expedited licensing process, Dillon does not recommend a bentonite mat liner for the new Wawanesa lagoon.

5.1.3 Recommendation

Dillon recommends proceeding with the constructed soil liner option for the new Wawanesa lagoon. While it is labour intensive to construct, it is less expensive and more resilient if damaged as compared to a synthetic liner. It is also less costly and more likely to proceed quickly through the environmental licensing process than bentonite mix or bentonite mat liners.

5.2 Preliminary Surveys

On November 7 and 8, 2013, Dillon staff performed a preliminary survey of the existing lift station. On April 24, 2014, Dillon staff performed a detailed survey of the proposed lagoon site and a preliminary survey of the proposed forcemain route. The following table provides a sample of elevations shot in both surveys and calculated for use in design.

Table 3: Selection of Elevations from Surveys

Location	Elevations	
	Measured	**Calculated**
Bottom of Lift Station **(B)**	-	350.54
Top of Concrete at Lift Station **(D)**	357.04	-
Ground Elevation of Road at PR 340	366.06	-
Ground Elevation of Road at Western Property Border	385.80	-
Ground Water Detected in TH6	-	376.79
Proposed Lagoon Bottom **(C)**	-	378.13
Proposed Maximum Liquid Level **(A)**	-	379.63
Proposed Lagoon Top of Berm	-	380.63
Ground Elevation at South Perimeter of Lagoon Field	379.60	-
Ground Elevation at Culvert	376.45	-

As described in **Section 4.2**, the maximum static head the lift station pumps will need to overcome is approximately **(A)** – **(B)**, calculated to be 29.1 m. The minimum static head the lift station pumps will need to overcome would occur when the lift station is full and the lagoon is empty: approximately **(C)** – **(D)**, calculated to be 21.1 m. These parameters are used in conjunction with estimates of head loss through the proposed forcemain to select the lift station pumps (see **Section 6.3**).

The survey of the lagoon site also picked up the ditch on the north side of Commercial Street, the municipal road that runs east-west along the southern border of the property. This municipal road ditch flows east to a natural drainage route (seasonal creek) that traverses the farmer's field east of the proposed lagoon site. The natural drain flows northeast then southwest to the Souris River, as shown on Sheet M-4 (**Appendix A**). The estimated length of the natural drain is 1.8 km.

Selected photos from the November survey and the April soils investigation have been included in **Appendix D**.

6 PROPOSED DEVELOPMENTS

The following new developments and infrastructure upgrades are proposed to satisfy the design parameters in **Section 4**, taking into account the information gathered in **Section 5**.

6.1 Lagoon

Dillon proposes that Wawanesa construct a two-cell facultative lagoon adjacent to the roadway, in the southeast corner of SE1/4 25-7-17 WPM. The primary cell and truck dump will be closest to the road, with the secondary cell following to the north. The top of cell dimensions of the proposed lagoon cells are outlined in **Table 4**, below. The bottom of cell dimensions are shown on Sheet M-1 in **Appendix A**.

Table 4: Proposed Lagoon Configuration

Cell	Approximate Top of Cell Dimensions (m)		Approximate Storage Volume (m^3)	Approximate Treatment Surface Area (m^2)
Primary	82	194	7,530	12,247
Secondary	200	195	41,006	n/a
Total			**48,536**	**12,247**

Note that only one half of the liquid volume of the primary cell was considered as hydraulic storage capacity, as per CWS guidelines. The following additional design choices were made in keeping with CWS guidelines:

- Berm height of 2.5 m, corresponding to a maximum liquid depth of 1.5 m with 1.0 m freeboard;
- Berm side slopes and drainage ditch slopes at 4:1;
- Berm top width of 3 m;
- Tiering of cells with secondary cell approximately 0.4 m lower than primary cell; and,
- Drainage ditching approximately 6 m from the berm toe of slope.

This results in a total hydraulic storage capacity of **48,500 m^3**, above the minimum design storage capacity of 44,500 m^3. The proposed lagoon also has approximately **12,250 m^2** of treatment area, above the minimum design surface area of 12,100 m^2. As per **Section 4.1**, any proposed developments or upgrades associated with nutrient management have been considered separately and not included in this report.

The parcel of land that Wawanesa is buying must not only include the direct lagoon footprint, but also a "buffer" as specified by CWS. CWS recommends a minimum of 30.48 m between the toe of the lagoon berms to the property line, which is applicable on all sides of the lagoon. As the neighbouring properties on all sides are agricultural and unlikely to be residentially developed, Dillon is instead recommending a typical buffer distance of 30.48 m. As such, the minimum parcel of land that Wawanesa will need to purchase is **279 m (E-W) x 372 m (N-S)**, or 10.4 hectares (**25.6 acres**). This is shown on Sheet M-1 in **Appendix A**.

The following components are proposed:

- **Access Gate** – to prevent unauthorized access and/or septage disposal into the lagoon, a manually operated, lockable swinging gate is proposed to control access to the site. The Municipality would issue a key to authorized haulers and require submission of hauling records to provide to CWS when requested. Dillon has not made an allowance for an automated system, but this is also a possibility that Wawanesa could consider. An automated gate or digital padlock would assist Wawanesa in maintaining detailed septage discharge records (i.e., number and volume of loads per day/week, name of the hauler, and the name, location, and type of waste in each load).
- **Approach Road** – A short approach road would be constructed along the currently unused government road allowance running along the eastern border of the site. The approach road would lead to a truck turnaround and truck dump at the southeast corner of the primary cell.
- **Truck Dump and Spillway** – a concrete pad installed in front of the truck-hauled septage discharge pad (i.e., spillway). This reduces the potential for trucks to "miss" the lagoon spillway whilst discharging septage from their trucks.
- **Rip Rap** – lining the interior berms of both lagoon cells with rock to reduce wave action erosion.
- **Fencing** – surrounding the entire perimeter of the lagoon facility at the property line to prevent unauthorized access to the site.

After the wastewater has been treated at the proposed lagoon, it must be discharged to a surface water body. The proposed lagoon effluent discharge ditch will start at the discharge point of the storage cell and flow south approximately 215 m in a new ditch. It will cross the approach road via a new culvert and join the existing municipal road ditch at the northeastern corner of the new approach and existing municipal road intersection (see Sheet M-1 in **Appendix A**). Effluent will then flow approximately 95 m to the east in the existing municipal road ditch, where it will join an existing natural drainage feature that meanders approximately 1.8 km before terminating at the Souris River. This proposed drainage route will require that both the existing culvert crossing the N-S government road allowance and the municipal ditch east of the government road allowance be regraded.

Figure 5: Natural Drainage Feature Near Proposed Effluent Discharge

It is anticipated that in addition to purchasing the land for the lagoon, Wawanesa will also need to obtain permission from the municipality to discharge along their roadway ditch, create an approach road on the government road allowance, and regrade a culvert and portion of the ditch.

6.2 Forcemain

To convey wastewater from Wawanesa to the proposed lagoon site, a new forcemain will need to be constructed connecting the Village's existing lift station (by the WWTP) to the lagoon primary treatment cell. The proposed alignment for the forcemain is shown on Sheet M-3 in **Appendix A**. It is as follows:

- South from the lift station to Commercial Street;
- East along Commercial Street to PR 340;
- Cross under PR 340 and continue east along Commercial Street for approximately 2.6 km; and,
- Enter the lagoon site and discharge into primary cell.

This route is approximately 4.0 km in total length. Based on the minimum flow rate of 9 L/s and optimal scouring velocity of 0.9 – 1.4 m/s (as per **Section 4.2**), the forcemain must be a minimum of 100 mm in diameter. As the pipe diameter increases, friction losses along the length of the pipe decrease, but the flow rate needed to maintain the minimum scouring velocity increases. After calculating minimum flow rates and head losses for 100 mm, 150 mm, and 200 mm pipe diameters, Dillon has selected **200 mm DR 11 HDPE** pipe as the best balance between flow and head loss.

It is anticipated that a valve chamber will also be required to house isolation and check valves for the forcemain. An underground valve chamber has been proposed to reduce confined space entry requirements. The forcemain installation will be achieved via directional drilling. An allowance has been made for air release valves at two locations along the forcemain route where the pipe slopes downwards.

6.3 Lift Station

Since the existing lift station barrel is relatively new and considered in good condition (see **Section 2.2**), it would be to Wawanesa's advantage to reuse the existing lift station, connecting it with the new forcemain. To make this feasible, the pumps would need to be upgraded. In addition to overcoming the static head of 29.1 m due to the elevation difference between the bottom of the lift station and the liquid level in the lagoon, the pumps must also provide enough energy to offset head losses in the 4.0 km forcemain. To select an appropriate pump, Dillon generated an approximate system curve for the forcemain, plotting a variety of flow rates against total head required to pump wastewater through the forcemain. The optimal flow for the specified forcemain is between 22 – 34 L/s, which corresponds to 16-36 m of head losses, assuming a smooth pipe[3].

[3] Head loss was calculated using a Hazen-Williams roughness factor of C = 150, a typical design value for HDPE. HDPE retains its smoothness well throughout its lifetime. A worst case scenario of maximum static head and a Hazen-Williams roughness factor of C=120 is shown on the above figure as an outside design range. This worst case scenario was not used to select pumps.

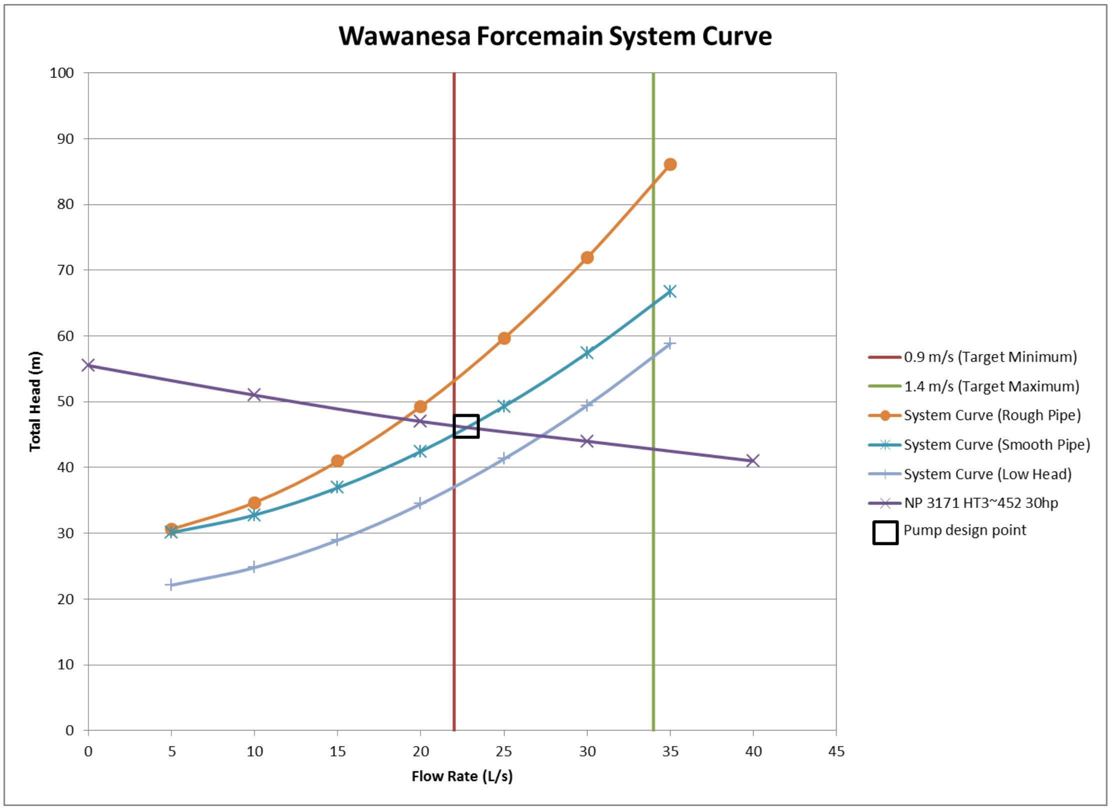


Figure 6: System Curve for Pump Selection

The wastewater pump that best fit the system curve was a 30 hp Flygt 3171 303 mm impeller four pole pump (see **Appendix E** for the pump data sheet). Dillon recommends installing two pumps in a duty-standby configuration. As seen on **Figure 6**, at the duty point, the flow rate through the forcemain is **23 L/s** and the Total Dynamic Head (TDH) is **17 m**.

Upgrading to 30 hp pumps would be a significant change from the existing 3 hp pumps. Dillon and Contec initially had concerns about whether or not the upgraded pumps would fit inside the existing lift station. After examining the sketches from the initial lift station construction in 2007 (see **Appendix F**) and further consultation between Dillon, Xylem, and Contec, all parties have reached a consensus that a retrofit of the existing lift station to fit the specified pumps would be feasible. A retrofit would involve:

- Replacing the top of the lift station, as the current hatch opening is not wide enough;
- Removing the existing guide rails and reinstalling them further apart from each other;
- Replacing the existing PVC headers with stainless steel for reliability reasons;
- Installing a steel anchor frame to the bottom of the lift station, onto which the new pump discharge connections must be bolted;
- Installing a new control panel; and,
- Upgrading the electrical from single phase to 3-phase power, as required by the new pumps.

Given the complexity and customization of this retrofit to Wawanesa's existing system, which was installed by Contec, Dillon recommends that the lift station upgrade not be included in either the lagoon or the forcemain tender packages. Dillon recommends issuing a separate request for quotation.

7 SCHEDULE

Given the temporary and inadequate nature of the existing wastewater treatment process at Wawanesa, an accelerated construction schedule is required. As such, construction is anticipated to start in late summer 2014 and conclude in spring 2015.