TOWN OF MIDDLEBURY

MIDDLEBURY WASTEWATER TREATMENT FACILITY

PHASE 2 INTERIM PRELIMINARY ENGINEERING REPORT

VT CWSRF LOAN RF1-224

- 20-YEAR FACILITY EVALUATION
- FACILITIES PLANNING STUDY

(DRAFT 3/9/2020)

TATA & HOWARD

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TATA & HOWARD

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2.1 Location and Plant History

Wastewater treatment for the Town of Middlebury is conducted on two sites within the corporate limits of the Town. The Main Pumping Station is located on Lucius Shaw Lane at the site of the Town's original Wastewater Treatment Facility (WWTF). When the current WWTF facility was constructed on Industrial Avenue in 2000, the old WWTF site was repurposed into the Main Pumping Station. In 2010, the Main Pump Station was expanded to add a 200,000-gallon wet well and grit removal equipment.

The current WWTF is an activated sludge treatment plant employing Sequencing Batch Reactors (SBR) to provide biological treatment and phosphorus removal. The facility uses Ultraviolet Disinfection prior to discharging plant effluent to Otter Creek in accordance with a National Pollutant Discharge Elimination System (NPDES) permit.

Figure No. 2-1 presents the location of the Main Pumping Station and the Middlebury WWTF.

2.2 Environmental Resources Present

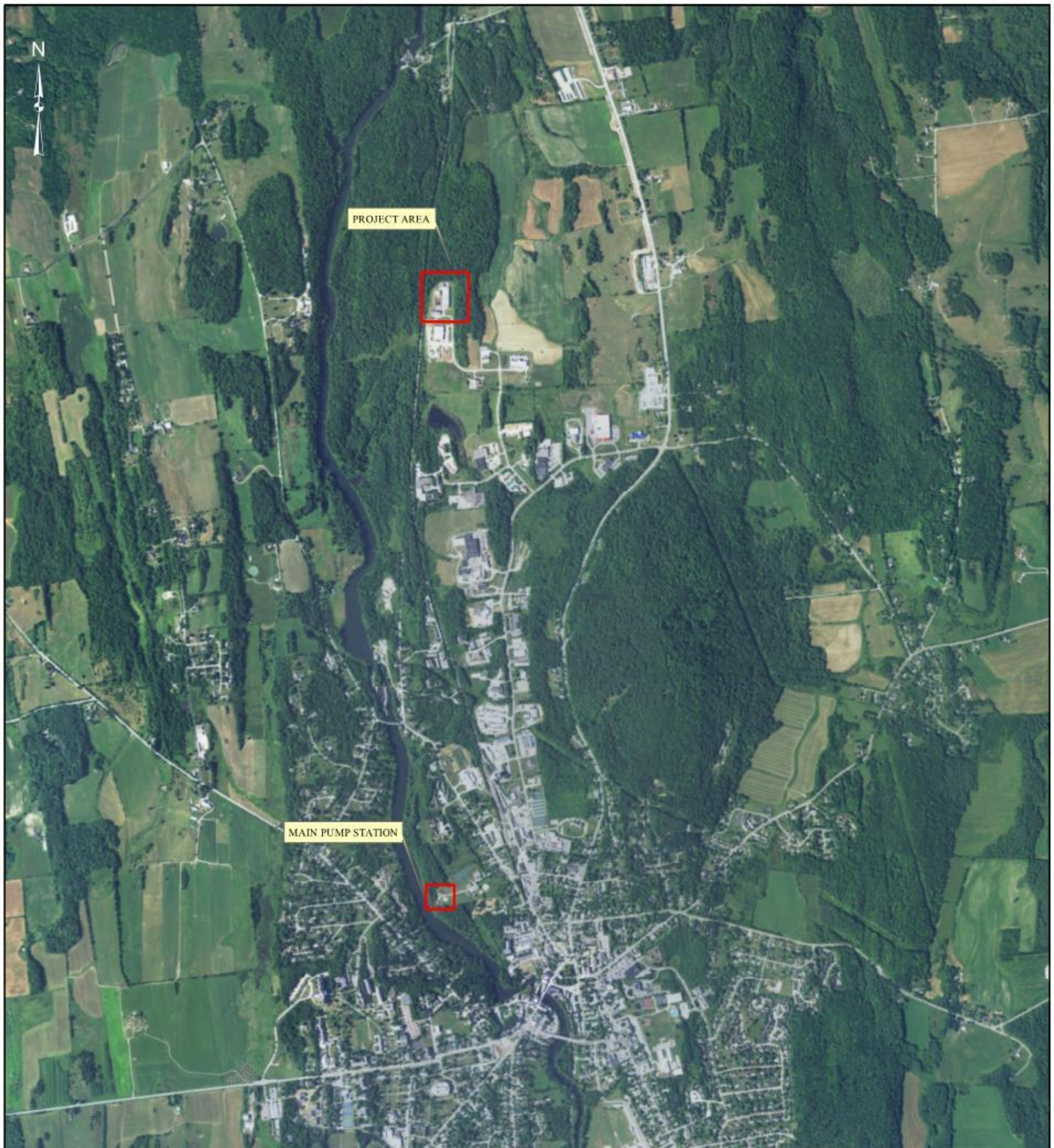
<<To be inserted at a later date, with BFE and important resource data>>

2.3 Population Projection

Based on regional population data and US Census data, the population growth trend in Vermont has been flat and on the decline. Census data shows that growth for Middlebury and Addison County have also been flat. A population projection prepared by the Town predicts the population in Middlebury, VT will decline from 8,496 people in 2010 to 8,287 people by 2030. Figure No. 2-2 presents the regional populations trends based on U.S Census data.

Tata & Howard analyzed the historic population of Middlebury from 1790 through 2017. The 2017 population is approximately 8,598 based on recent U.S. Census data. A linear trendline of the data predicts a total growth of five percent over the next 20 years. Based on a visual evaluation of the data and trend line, this growth rate is conservative. Figures No. 2-2 and 2-3 present Middlebury's historic population and projected growth rate.

The 2017 Middlebury Town Plan was reviewed and it estimates that the Town will experience growth of approximately 30 people per year through 2020. Extrapolating that growth rate through 2038 results in a 2038 population of approximately 9,228. The plan also projects college enrollment at Middlebury College to remain at current levels. The college represents 2,450 residents, approximately 28 percent, of the current population. The projections in the Town Plan are consistent with the population projection completed by Tata & Howard; therefore, a total growth of five percent over the next 20 years will be used in the evaluation of future wastewater loading to the WWTF.



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Date: February 2019
Approximate Scale: 1" = 2,000'

Locus Map

Wastewater Treatment Facility
Town of Middlebury, VT

Figure No.

2-1

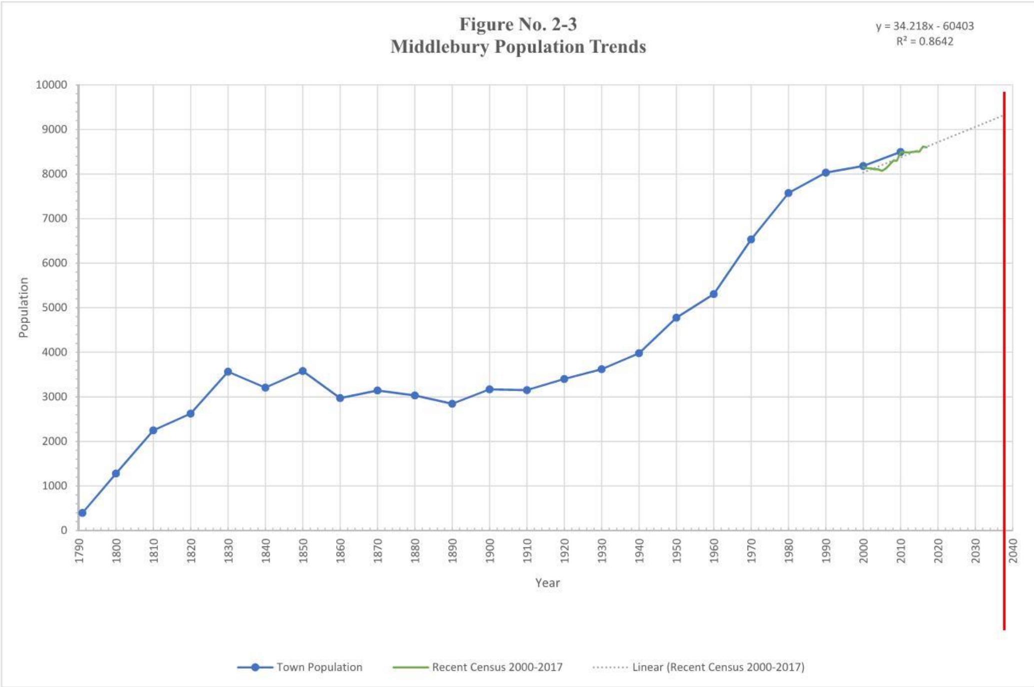


Figure No. 2-3
Middlebury Population Trends
y = 34.218x - 60403
R^2 = 0.8642
Population
Year
Town Population
Recent Census 2000-2017
Linear (Recent Census 2000-2017)

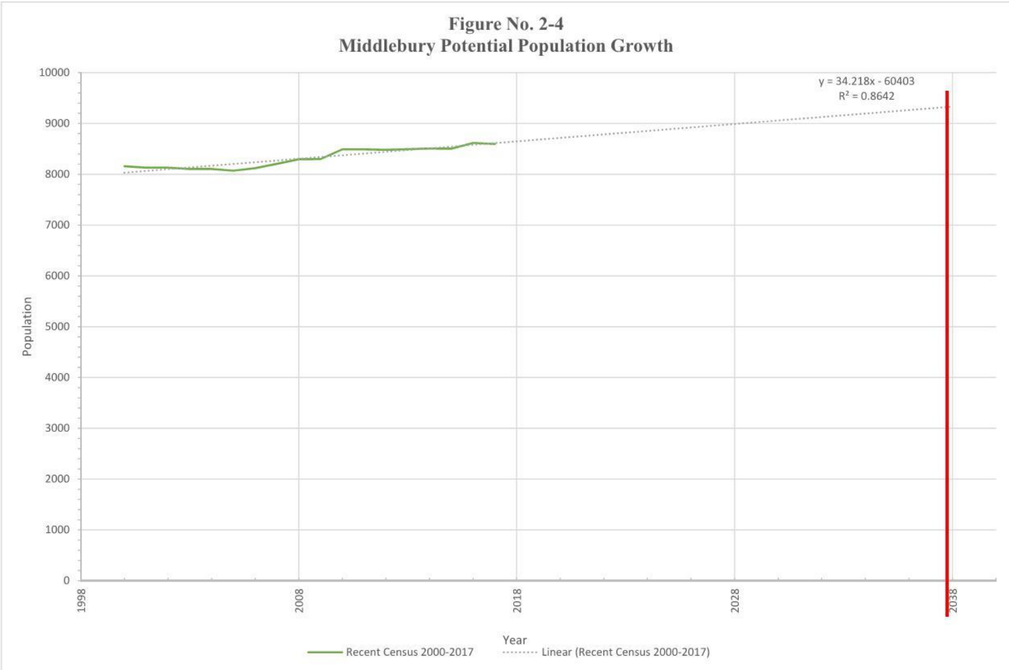


Figure No. 2-4
Middlebury Potential Population Growth
y = 34.218x - 60403
R^2 = 0.8642
Population
Year
Recent Census 2000-2017
Linear (Recent Census 2000-2017)

2.4 Plant Design Loading

To evaluate the condition and performance of the WWTF, the current influent loading to the plant and the effluent levels have been compared to the permit limits. Five years of plant data were evaluated to compare the actual WWTF flow and loading to the original design and projected future flows.

Review of the influent loading at the WWTF included a review of the hydraulic loading and the concentration of wastewater constituents. The hydraulic loading to the plant was considered at Average Daily Flow (ADF), the Maximum Daily Flow (MDF), and the peak hourly flow in accordance with TR-16 WWTF design standards. The ADF was used to evaluate the most common loading to the plant and must be considered for equipment sizing, operational practices, and energy efficiency. Peak flow is considered primarily for hydraulic purposes. The equipment and tanks must be able to process the peak flow without experiencing overflows or unpermitted discharges. The concentration of constituents at the peak flow are typically lower than at other hydraulic loading scenarios due to dilution by extraneous flows; therefore, the MDF is considered to capture the worst case for constituent concentration loading for certain unit processes.

Hydraulic Loading:
The ADF ranged from 0.99 to 1.04 million gallons per day (MGD) from 2013 through 2017, with an average of 1.01 MGD, as shown in Table No. 2-1 and Figure No. 2-5. The range of flow variation is less than five percent with no discernible upward or downward trend. The apparent consistency of ADF over the five-year period correlates with the relatively flat population growth experienced in Town over the same period. In order to be conservative, the 2017 ADF of 1.04 MGD was used for the WWTF existing conditions analysis.

The 2017 ADF of 1.04 MGD is approximately 65 percent of the design ADF of 1.58 MGD (ref Appendix __, Middlebury, Vermont, Wastewater Loading Projections, March 28, 1997). The impacts of the lower ADF loading at the WWTF will be evaluated in the existing condition assessment later in this report.

Table No. 2-1
Daily Flows

Year	Average Daily Flow (MGD)	Max Daily Flow (MGD)
2013	0.99	2.56
2014	1.03	3.32
2015	1.01	2.63
2016	0.99	2.52
2017	1.04	3.04
5-Year Avg.	1.01	2.81

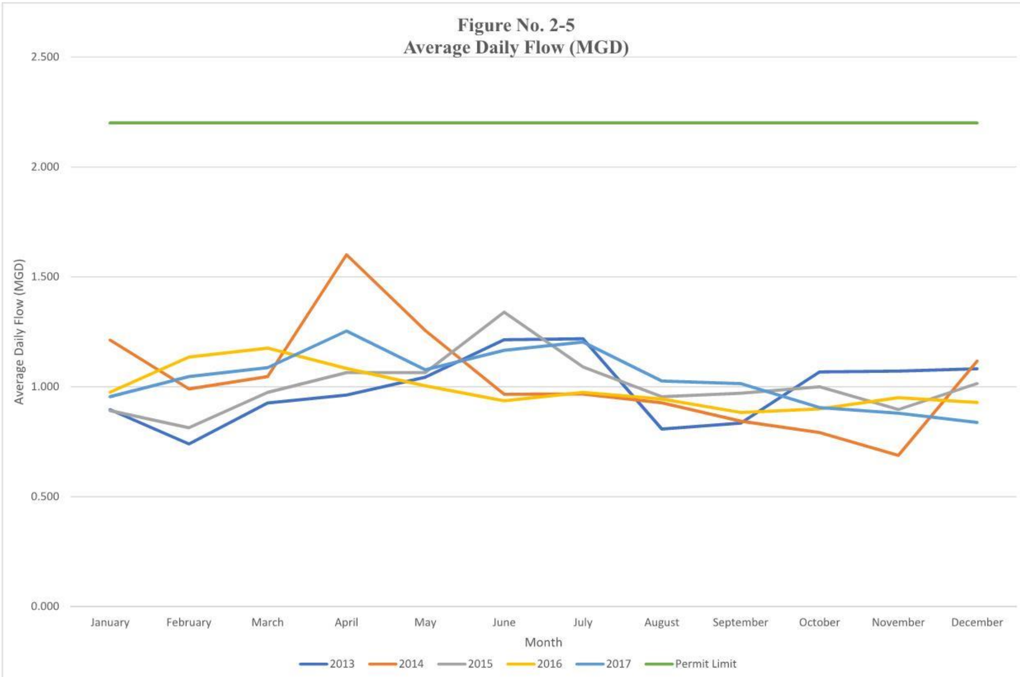


Figure No. 2-5
Average Daily Flow (MGD)
2.500
2.000
1.500
1.000
0.500
0.000
Average Daily Flow (MGD)
January
February
March
April
May
June
July
August
September
October
November
December
Month
2013
2014
2015
2016
2017
Permit Limit

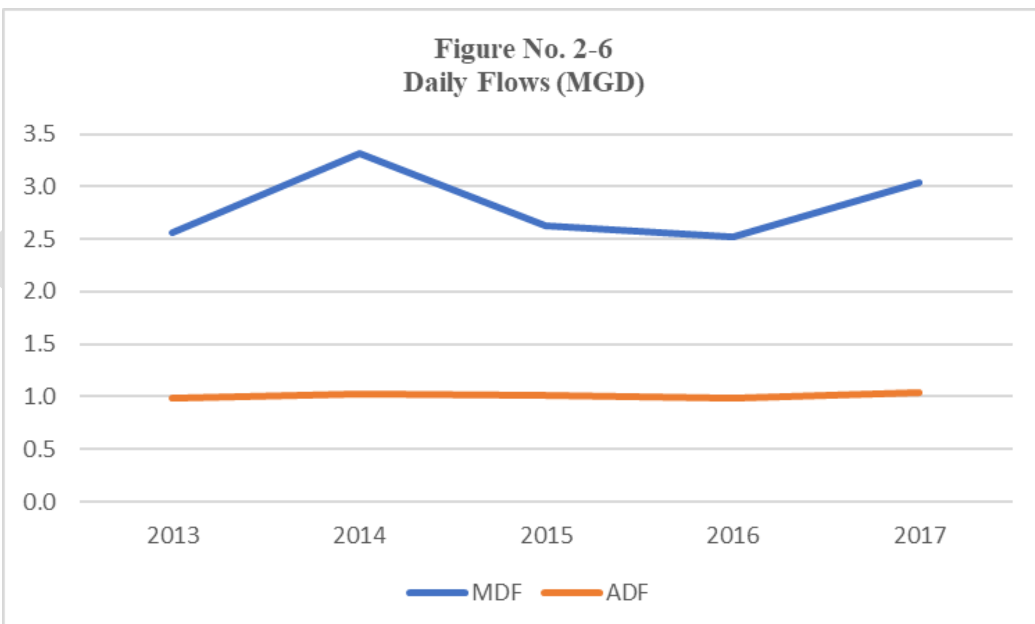


Figure No. 2-6
Daily Flows (MGD)
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
2013
2014
2015
2016
2017
MDF
ADF

The MDF over the same five-year period ranged from 2.52 MGD to 3.32 MGD with an average of 2.81 MGD as shown in Table No. 2-1 and Figure No. 2-6. Since the MDF in 2017 was higher than the five-year average, it was determined that the average value is not conservative enough; therefore, the maximum value for the five-year period, 3.32 MGD will be used. This value is approximately 72 percent of the design MDF. A review of the historical flow shows slight seasonal variations in flow, with April being a month with higher flows. June and July are also higher months. This may be due to increased activity in the industrial sector or the occurrence of extraordinarily high monthly rainfall.

Peak hour flows are not reported at the plant; however, instantaneous peak flows are recorded. As presented in Table No. 2-2, the instantaneous peaks have ranged from 4.50 MGD to 6.90 MGD; however, the instantaneous peak flow has been significantly lower the last three years. The instantaneous peak flows are directly related to the Main Pump Station pumping rates, and the highest peaks are observed to be due to pump operation, and not storm events or elevated influent flow to the pump station. The high instantaneous peak flows reported in 2013 and 2014 occurred as a result of flow meter error at the Main Pumping Station. A review of the pump station flow charts indicate the peak hour flow is approximately 4.50 MGD. The estimated peak hour flow using the Merrimack curve in accordance with TR-16 is approximately 4.00 MGD, which is a typical design value for a WWTF with an ADF of 1.04 MGD. Therefore, the observed peak hourly flow after flow equalization is 112% of expected value and 73% of the design value. The actual peak flow into the pumping station is greater.

Table No. 2-2
Instantaneous Peak Flows

Year	Peak Flow (MGD)
2013	6.79
2014	6.87
2015	4.89
2016	4.48
2017	4.48
5-Year Avg.	5.52

Wastewater Constituent Loading:
In addition to the hydraulic loading to the plant, the concentration, volume and mass of certain constituents must also be understood to evaluate the ability of the WWTF to remove these constituents and meet permit limits.

Organic loading is typically evaluated in terms of the daily concentration of biological oxygen demand (BOD) for 5 days at 20º C. BOD was evaluated using monthly data for the past five years as shown in Tables No. 2-3 and 2-4. BOD was evaluated as both a concentration relative to the influent flow and as total mass loading of pounds per day (lbs/d).

The average of the maximum annual influent BOD concentration over the five-year period was 661 mg/l. This value is 38 percent higher than the design concentration of 480 mg/l and is higher than typical influent values of 100 to 400 mg/l expected in municipal wastewater. The influent

BOD concentrations show a generally flat trend until 2017 where it shows a slight increase from the previous four years. The average annual daily concentration and mass loading of influent and effluent biochemical oxygen demand (BOD) were evaluated for the past five years as shown in Tables No. 2-3 and 2-4, and in Figure Nos. 2-7 and 2-8.

Although the influent concentration of BOD is high, the mass loading of BOD is within the plant's design values. The plant was designed for an influent BOD loading of 8,802 lbs; however, the five-year average is only 5,546 lbs which is approximately 63 percent of the design value.

The relationship of high influent BOD concentration to moderate mass indicates that the WWTF receives strong waste; however, due to reduced influent hydraulic loading, the total volume and mass of organics is within the WWTF design capacity. The high strength waste can be attributed to industry in town, especially food and beverage related customers. It is important that the future projected loading to the WWTF consider the high wastewater strength when determining organic capacity long-term. If the hydraulic loading to the plant approaches the design value, and the BOD concentration remain constant, the total BOD loading may exceed the design loading.

Table No. 2-3
Average Annual BOD Concentrations (mg/l)

Year	Influent	Effluent
2013	395	6
2014	501	8
2015	441	11
2016	508	6
2017	533	6
5-Year Avg.	488	8

Table No. 2-4
Average Annual BOD Loading (lbs)

Year	Influent	Effluent
2013	4,102	58
2014	4,147	86
2015	4,032	67
2016	4,029	67
2017	4,039	72
5-Year Avg.	4,052	70

The suspended solids loading to the WWTF was similarly reviewed. The average annual daily concentration and mass loading of influent and effluent total suspended solids (TSS) were evaluated for the past five years as shown in Tables No. 2-5 and 2-6, and in Figure Nos. 2-7 and 2-8. The influent TSS concentrations show a generally flat trend until 2016 where it shows a slight drop compared to the previous four years. The effluent concentration data provides a flat trend line over five years. The TSS loading for 2013 through 2015 are slightly elevated compared to typical influent values, however, the loading over the latest two years align well with typical values. The average daily design load for TSS is 4510 lbs. at a concentration of 342 mg/l.

Table No. 2-5
Average Annual TSS Concentrations (mg/l)

Year	Influent	Effluent
2013	374	6
2014	370	8
2015	322	6
2016	240	8
2017	215	6
5-Year Avg.	304	7

Table No. 2-6
Average Annual TSS Loading (lbs)

Year	Influent	Effluent
2013	3,104	50
2014	2,977	66
2015	2,634	48
2016	1,973	66
2017	1,832	54
5-Year Avg.	2,504	57

Figure No. 2-7
Average Annual Influent BOD and TSS Concentrations (mg/l)

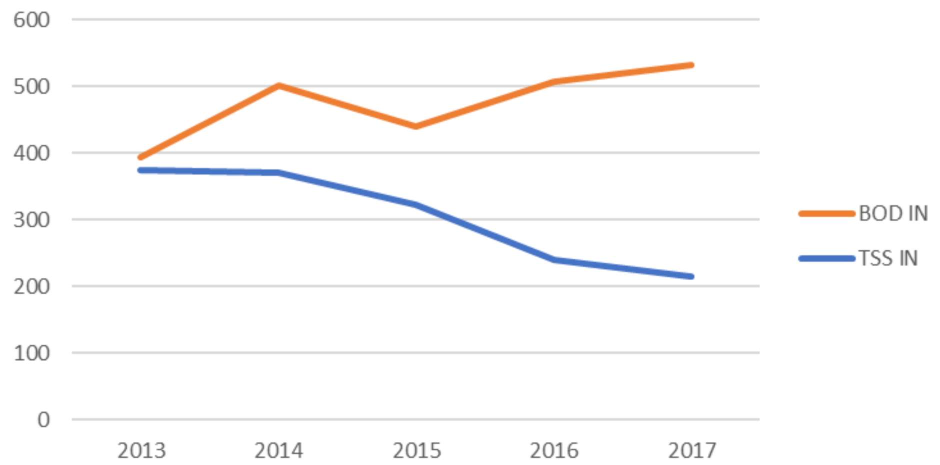
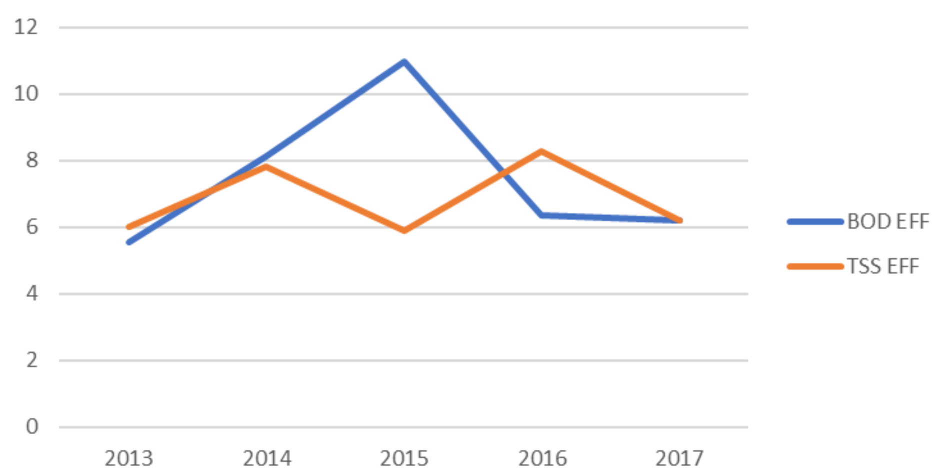


Figure No. 2-8
Average Annual Effluent BOD and TSS Concentrations (mg/l)



Infiltration and Inflow Study

For many communities, infiltration and inflow (I/I) contributes a large portion of the hydraulic loading to the collection system and WWTF. Groundwater infiltration into the collection system can use up valuable hydraulic capacity. Weather related I/I can lead to large spikes in flows, which can cause process upsets at the plant. The flow is often highly diluted which can impact the organic loading to the plant and potentially impact biological treatment. A limited infiltration and inflow (I/I) analysis was completed using available rain and WWTF data to determine potential impacts of I/I at the WWTF. The State of Vermont considers Middlebury Wastewater system a wet weather influenced sanitary sewer overflow. This means that the system experiences wet weather overflows without being recognized as having combined storm and sanitary sewers.

The analysis was completed utilizing the methodologies of the Guide for Estimating Infiltration and Inflow, Environmental Protection Agency (EPA), June 2014 and Guidelines for Performing Infiltration/Inflow Analyses and Sewer System Evaluation Surveys, Massachusetts Department of Environmental Protection (MADEP), May 2017. Rainfall and plant flow data from the 2015 through 2017 monthly plant operations reports were reviewed and various periods of high infiltration periods of high infiltration (high minimum flows)n (high minimum flows), storm events (largest reported rainfall over 2 days or less) and periods of low infiltration (low minimum flows and lack of rainfall). Plant data from 2013 and 2014 was not used because minimum flows were recorded as no flow, and therefore, was not suitable for estimating I/I for those years. When the pumps turn off the recorded flow is zero. It had been the practice of the operators to report this zero flow as the minimum. Since 2014, the operators report the lowest recorded flow greater than zero, which represents the minimum pumping rate of a single pump. The main pumping station's two wetwells and pump operations impact the I/I estimate by dampening flows upstream of the WWTF influent flow meter. The influent flow data used is after the station's wetwell, and the large volume of the two wetwells can act to buffer the I/I to the treatment plant by storing the increased flow.

Dry periods for each month were used to estimate the base sanitary flow and infiltration rate. The groundwater infiltration was determined by subtracting the Agri-Mark nighttime industrial discharge from the average minimum flow during the dry weather period studied. The average minimum flow was assumed to be the nighttime flow for the period. The average dry weather flow (ADW) is the average flow for a 24-hour period during the same period each month. The base sanitary flow was calculated by subtracting the groundwater infiltration from the ADW. The average annual inflow was calculated by subtracting the average dry weather flow from the overall annual average flow for the 3-year period. Table No. 2-7 includes the results of this analysis.

Table No. 2-7
Annual I/I Data

I/I Characteristic	Flow (gpd)
Average Annual Flow[1] (gpd)	1,010,000
Base Sanitary Flow (gpd)	776,000
Average Annual Infiltration Rate(gpd)	198,000
Average Annual Inflow Rate (gpd)	37,000
Average Annual I/I Rate (gpd)	235,000
Peak Infiltration Rate (gpd)	325,000
Peak Infiltration Rate[2] (gpd/idm)	810

1. Average annual flow presented in this table is the average for 2015 through 2017, the years analyzed in the I/I evaluation.
2. To be completed once collection system information is received.

As can be seen in Table No. 2-7, the average annual I/I contributes approximately 23% of the total flow to the plant. The peak infiltration rate is the maximum infiltration rate recorded during the springtime high groundwater period which occurred in the April 2017.

This peak infiltration rate equates to 810 gpd per inch diameter mile (gpd/idm) for the Middlebury collection system. An infiltration rate greater than 4,000 gpd/idm (ref. Guidelines for Performing Infiltration/Inflow Analyses and Sewer System Evaluation Surveys, Massachusetts Department of Environmental Protection (MADEP), May 2017) is considered excessive for sub-catchment areas containing approximately 20,000 linear feet of sewer. Continuous flow of individual sub-catchments of approximately this size would be required to further asses infiltration within the system. New pipe has an allowable infiltration rate of 200 gpd/idm.

The plant flow and rainfall data were reviewed to estimate peak inflow rates and inflow volume to the plant that occurred during significant rain events. The storm events that resulted in the highest peak inflow rate to the plant are listed in Table No. 2-8. The total rainfall is the sum of the rain that was recorded prior to the peak flows occurring and consist of one to three days of recorded daily rainfall. The duration of the storms was estimated using the KVTMIDDL19 Weather Underground station. The June 23, 2017 storm is the most severe and corresponds closely with a 2 year 12 hour storm depth; the intensity of the storm is unknown. This rainfall event is less severe than the 1 year 6 hour storm, which has a rainfall depth of 1.72 inches with a peak intensity of 0.87 in/hr. The peak inflow rate was calculated by subtracting the ADW before the storm event from the peak flow recorded during the storm. The total inflow volume was determined from the area under the flow charts for the storm event, again subtracting the daily volume of flow for the dry weather day before.

The drainage area estimates the area contributing run off to the WWTF and was calculated using the inflow volume and depth of rainfall. The drainage area for storms without plant flow charts was estimated using the average daily flow before, during, and after the rain event and the depth of rainfall. The average drainage area for these storms is 27 acres. It assumes 100% impervious area and will increase by up to 3 times that area, when actual land uses are considered. These are large drainage areas that could indicate a larger issue with I/I in the collection system.

Table No. 2-8
Inflow Analysis

Date	Total Rainfall (inches)	Max 24 hr Rainfall (inches)	Storm Duration (hr)	Peak Flow (mgd)	ADW (mgd)	Peak Inflow (mgd)	Inflow Volume (MG)	Drainage Area (acres)
5/11/2015	1.94	1.41	12	4.85	0.94	3.91	1.31	24.9
6/9/2015	1.92	1.53	11	4.72	0.91	3.81	1.46	28.1
6/23/2015	3.04	1.30	39	4.89	1.07	3.82	2.96	35.9
2/24/2016	2.74	1.92	26	4.48	0.98	3.50	1.69	22.7
7/1/2017	3.78	1.62	48	4.48	1.15	3.33	2.17	21.1
6/23/2017	2.62	2.08	13	4.28	1.04	3.24	2.12	29.8
5YR Design	3.00	3.00	24	6.2	2.2	4.0	2.2	27.0

Table No. 2-9 shows the corresponding annual volumes of I/I in the system based on the rates presented in Table No. 2-7. The Town of Middlebury is treating 85.52 million gallons (MG) of I/I per year, which relates to approximately $600,000 based on the annual operating budget. The annual operating cost to transport the infiltration is the pumping cost at the Main Pump Station, only. We estimate this to be $8,000 in 2019 and $12,000 in 2040. There is no operating cost for treatment of infiltration, as it is clear water. Infiltration represents 23% of total daily flow and requires pipe and unit process capacity.

The Town's collection system is approximately 240,000 in length, with a replacement cost of $48M (@$200/ft) and a life expectancy of 100 years. Pipe sizes are determine based on peak hourly flow. Assuming a peak factor of 4 for 1.6 MGD (2020 flow), or 6.4 MGD peak flow, and 1.5 for infiltration, or 0.34 MGD; infiltration consumes 5% of pipe capacity. The collection system capital cost on an annualized basis would be approximately 6% of 20 years/100 years of $48M or $576,000 per year, for a 20 year planning period. The infiltration portion of this cost is 5% or $28,800 per year.

The value of 0.23 MGD in treatment plant capacity is approximately $1.15M at a value of $5/MGD capacity WWTF capital. The WWTF capital cost on an annualized basis would be approximately 6% of $1.15M or $69,000 per year.

The total present cost of infiltration is $8,000 (pumping) + $28,800 (collection system capital replacement) + $69,000 (WWTF capacity) = $105,800/yr.

Table No. 2-9
Annual I/I Volumes

I/I Characteristic	Flow (MG)
Average Annual Infiltration Volume (MG)	72
Average Annual Inflow Volume (MG)	13
Average Annual I/I Volume (MG)	86
Average Annual Wastewater Volume (MG)	380

2.5 Projection of Future Flows

This study evaluates the plant's needs for a 20-year period through 2038. To best project future flows, the flow was evaluated by type of customer including residential, commercial and industrial, since each segment of the customer base may not grow at the same rate. Residential customers represent the largest component of the total flow to the plant, and projected flows correlate to changes in population.

As discussed previously in this report, the population growth is estimated to be zero to five percent over a twenty-year span. Based on a growth rate of five percent, the population in 2038 would be approximately 9,300 which is approximately 600 more people than presented in the 2017 census data. Assuming all 600 people join the sewered population with a water use of 70 gpcd and 100% discharge of water use to the sewer, the increase in wastewater flow to the collection system and

WWTF would be approximately 42,000 gpd. This value represents 4% of the current ADF. For the purpose of projecting the future flows to the WWTF, assuming that the 2038 wastewater flow will increase proportionally with the population growth rate (five percent) is likely conservative.

Commercial and industrial users discharge wastewater of varied flow and loading. Industrial users typically discharge waste with higher concentrations of organics, solids and other pollutants such as metals. Due to the wide variety of industry, projecting future flow and loading is difficult.

Table No. 2-10 summarizes the Design Flows for various classifications of users. Table 2-11 summarizes the Actual Flows for these same classification for 2017. Table 2-12 presents the 2017 Actual Flows as a percentage of Design Flow for each classification of user.

Table 2-10
Design Flow and Loads

Source		Flow	BOD (lbs)	BOD (mg/L)*	TSS (lbs)	TSS (mg/L)*	TP (lbs)	TP (mg/L)*
Residential		647,000	1,518	281	1,503	279	27	5
Commercial	College	200,000	469	281	465	279	8	5
Commercial	Other	100,000	235	281	232	279	4	5
Cabot		366,000	3,178	1,041	980	321	52	17
American Hard Cider		3,000	-	-	-	-	1	40
Otter Creek Brewery		100,000	250	300	-	-	8	10
Aqua Vita		-	-	-	-	-	-	-
Other industry		155,000	800	619	530	410	13	10
Septage		8,000	334	5,006	800	11,990	17	255
Sidestreams		-	-	-	-	-	-	-
Inflow/Infiltration		-						
Total (incl septage and sidestreams)		1,579,000	6,784	515	4,510	1,049,718	130	15
Total		1,579,000	6,784	515	4,510	342	130	10
Max Month		2,715,880	8,819	389	5,863	259	169	7

Table 2-11
Actual Flow and Loads

Source		Flow	BOD (lbs)	BOD (mg/L)*	TSS (lbs)	TSS (mg/L)*	TP (lbs)	TP (mg/L)*
Residential		171,421	244	171	475	332	7	5
Commercial	College	115,408	164	171	320	332	5	5
Commercial	Other	147,483	210	171	409	332	6	5
Cabot		377,300	2,343	745	889	283	86	27
American Hard Cider		21,864	575	3,153	27	148	1	4
Otter Creek Brewery		12,133	576	5,692	17	ND	ND	ND
Aqua Vita		2,631	47	2,142	4	182	0	5
Other industry		13,866	20	171	38	332	86	744
Septage		5,000	209	5,000	417	10,000	3	72
Sidestreams (BFP)*		72,700	411	678	206	339	20	33
Inflow/Infiltration		180,000	-	-	-	-	-	-
Total (incl septage and sidestreams)		1,119,806	4,388	470	937	100	196	21
WWTF		1,042,106	4,179	481	1,881	216	105	12
Max Month (excl septage + SS)		1,254,000	6,811	651	3,237	310	-	-

** - Not metered by inf flow meter*
Actual Industrial Loads 5/17-4/18

Table 2-12
Actual as Percentage of Design Flow and Loads

Source		Flow	BOD (lbs)	BOD (mg/L)*	TSS (lbs)	TSS (mg/L)*	TP (lbs)	TP (mg/L)*
Residential		26%	16%	61%	32%	119%	26%	98%
Commercial	College	58%	35%	61%	69%	119%	57%	98%
Commercial	Other	147%	89%	61%	176%	119%	145%	98%
Cabot		103%	74%	72%	91%	88%	165%	160%
American Hard Cider		729%	NA	NA	NA	NA	80%	11%
Otter Creek Brewery		12%	230%	1899%	NA	NA	NA	NA
Aqua Vita		NA	NA	NA	NA	NA	NA	NA
Other		9%	2%	28%	7%	81%	662%	7395%
Septage		63%	62%	100%	52%	83%	18%	28%
Sidestreams		NA	NA	NA	NA	NA	NA	NA
Inflow/Infiltration		NA	NA	NA	NA	NA	NA	NA
Total (incl septage and sidestreams)		71%	65%	91%	21%	0%	151%	141%
Total		66%	62%	93%	42%	63%	81%	122%
Max Month		46%	77%	167%	55%	120%	0%	0%

The Town has not identified any imminent industrial growth; therefore, rather than attempt to project industrial growth in Town, the projected non-residential flow and loading to the plant will be estimated at the same five percent growth. The difference between the design capacity and projected loading will be considered excess capacity reserved for future industrial growth. This approach is reasonable considering the Town has control over future industrial flows through the sewer connection permit process.

Table No. 2-13 presents a breakdown of the metered water flow by customer type and calculates the number of Equivalent Dwelling Units (EDU) for each classification, assuming that the average daily flow consumed per residential dwelling equals one EDU. An allowance for infiltration has been added to adjust the total flow to the 2017 ADF. Industrial flow values are based on actual metered wastewater flow, where available. The calculated EDU flow is 96 gpd. Based on an overall wastewater budget of $2,674,560, this equates to an annual cost per EDU of approximately $296/EDU. This value is below the approximate average annual user costs for municipal systems in Vermont of $500/EDU. The EDU is tool used by USDA to evaluate funding levels for a project.

Table No. 2-13
Calculation of Residential Equivalent Design Units (EDU)

	User Category	Flow (gpd)	Actual Connections
a	Residential (Full-time)	171,421	1,784
c	Commercial	111,779	433
d	Industrial	427,794	9
e	Institutional	151,112	3
f	Subtotal (Billable)	862,106	
g	Leakage (I/I)	180,000	
h	Present Average Daily Flow	1,042,106	
i	Number of full-time Residential EDUs*	1,784	
j	Flow per EDU = a/i	96	
k	Total EDUs = f/j	8972	
	*Residential EDUs = number of full-time residential units, for example: 1 apartment = 1 unit 1 mobile home = 1 unit 1 duplex = 2 units		
	This table adapted from USDA Form EDU 5, CALCULATION OF EQUIVALENT DWELLING UNITS (EDUs.		

In summary, the commercial and industrial flow to the plant is expected to remain constant unless future industrial customers are connected to the collection system. In order to be conservative, the total flow to the plant is projected to increase by five percent by 2038. The constituent loading, including BOD, TSS, TKN, and TP are not anticipated to change in concentration; however, with the increased flow, the total pounds will increase. Table No. 2-14 below summarizes a variety of design flow parameters for the WWTF.

Table No. 2-14
Summary of Various Design vs. Actual Flows

Flow Parameter	Design	Actual	% of Design
Average Annual Flow	1,579,00	1,042,106	66%
Peak Hourly Flow	6,200,000	4,500,000	73%
Maximum Daily Flow	4,600,000	2,810,000	61%
Maximum Monthly Flow	2,715,880	1,254,000	46%
Minimum Monthly Flow	1,249,305	838,000	67%
Minimum Weekly Flow	NR	NR	NR
Minimum Daily Flow	726,340	595,000	82%
Minimum Hourly	378,960	330,000	87%
Estimated From Merrimack Curves			

2.6 Wastewater Allocations

The Town allocates wastewater disposal capacity to four industries. The unused portion of the allocated capacity of BOD (4,259 lbs/day) represents 62% of the total influent BOD capacity of the WWTF (6,784 lbs/day) and 177% of the reserve BOD capacity (2,397 lbs/day). Although it is unlikely that the allocated capacities will ever be utilized, it is prudent for the Town to consider how to mange these allocations. A review of the allocation policy with consideration of sunsetting unused allocated capacities or applying a "put or pay" approach to recover the capital cost of holding the capacity available are examples of the types of considerations that may be made. If allocations far in excess of the design reserve capacities are to remain, consideration of increasing the design capacity of the WWTF to include this allocated capacity is recommended.

Table No. 2-14
Summary of Industrial Loads and Allocations

	FLOW (gallons per day)			BOD (pounds per day)			TSS (pounds per day)			TP (pounds per day)		
	Allocation	Actual	Balance	Allocation	Actual	Balance	Allocation	Actual	Balance	Allocation	Actual	Balance
Cabot	450,000	377,300	72,700	4,000	2,343	1,657	1,100	889	211	100	86	14
American Hard Cider	70,000	21,864	48,136	2,500	575	1,925	130	27	103	-	1	(1)
Otter Creek Brewery	20,000	12,133	7,867	1,000	576	424	-	17	(17)	-	ND	ND
Aqua Vita	8,500	2,631	5,869	300	47	253	25	4	21	-	0	(0)
Other industry	155,000	13,866	141,134	800	20	ND	530	-	530	13	ND	ND
BALANCE			275,706			4,259			848			13
DESIGN IND TOTAL	624,000	427,794	196,206	4,228	3,561	667	731	937	(206)	-	87	(87)
DESIGN WWTF TOTAL			459,194			2,397			1,942			3

2.6 Permit Requirements

The discharge of WWTF effluent is governed by a National Pollutant Discharge Elimination System (NPDES) permit. The permit sets limits on the levels of pollutants in the plant effluent. Table No. 2-15 presents the current permit limits and the current average levels of the regulated constituents. As presented in the table, the WWTF consistently meets its permit limits.

Table No. 2-15
Effluent Limits

Constituent	Units	Permit Limit	Average	Max. (5-YR)
Average Daily Flow	gpd	2,200,000	1,012,000	1,601,000
Biochemical Oxygen Demand Monthly (BOD_5)	mg/L	30	8.41	21.8
Total Suspended Solids Daily (TSS) Monthly	mg/L	30	6.86	22.0
Total Nitrogen	mg/L	Monitor Only, mg/L	5.821	-
Total Phosphorous Monthly Avg.	mg/L	0.800	0.331	-
Total Phosphorous Annual Avg.	lbs.	4,018	1,049	-
pH	Between 6.5 and 8.5 Standard Units			

NPDES permits are renewed every five years. At the time of renewal, or at the time of a modification of the plant, EPA may alter the limits and provisions of the permit; therefore, it is important to evaluate potential changes to the permit when evaluating the plant over a 20-year period. The Middlebury plant does not have a high potential for permit changes that would significantly impact the design of the current plant. The Middlebury permit includes a requirement to report Total Nitrogen, but does not have a discharge limit. There is potential long-term that EPA could institute a limit, such as 5 mg/L, in the future; however, without a trigger in the receiving water, the change is not expected to be imminent. NPDES TP mass loadings are expected to be held flat, so that any increase in permitted flow will require correspondingly lower concentrations and greater removal of TP.

2.7 Community Engagement

<<To be inserted at a later date>>

SECTION 3 – EXISTING FACILITIES

3.2 Condition of Existing Facility

3.2.0 Existing Process Evaluation

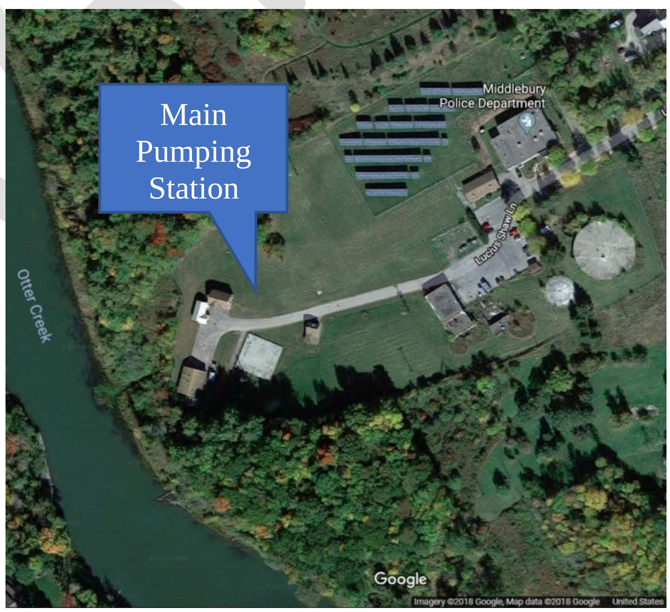
This section will review the existing conditions of the plant and will provide a process description, condition assessment and a review of the design and performance of each process. This information will serve as the basis for the recommendations for process improvements discussed in subsequent sections.

The process description and conditions assessments are based on a site visit conducted by Tata & Howard on May 31, 2018 and various discussions between Tata & Howard and the WWTF staff. The design and performance review include review of the design documents, manufacturer literature, and industry standard practice documents such as TR-16, 10 State Standards, and EPA Fact Sheet and Guidance Documents.

Any observed deficiencies or improvement opportunities will be identified herein, and further evaluated in the alternatives analysis later in this report.

3.2.1 Main Pumping Station

Process Description



Wastewater from the Middlebury collection system flows to the WWTF through the Main Pumping Station. The Main Pumping Station is located on Lucius Shaw Lane at the site of the Town's original WWTF. When the current WWTF facility was constructed on Industrial Avenue in 2000, the old WWTF was repurposed into the Main Pumping Station.

The Main Pumping Station includes screening and grit removal, flow storage, and pumping facilities.

Flow enters the pumping station through a Lakeside rotary screen. Influent screening removes large debris and particles to prevent damage to the pumps and equipment downstream in the treatment process. A manual bar rack serves as an emergency bypass.

Following influent screening, flow continues to the grit removal facilities. Grit removal was added to the Main Pumping Station as part of an expansion constructed in 2010. Grit removal is accomplished by means of a PISTA vortex grit chamber and a Smith & Loveless

Cyclone and Classifier. The grit chamber uses tangential velocity to create a vortex flow pattern causing dense grit particles to settle. The heavy particles settle into the hopper while the effluent exits from the top of the chamber. The collected grit is pumped to the grit cyclone and classifier which separate the water and fine organic matter from the grit resulting in more efficient grit disposal.

Next, flow enters a 200,000-gallon flow equalization wet well which was constructed as part of the 2010 upgrade. Due to the large size of the wet well, some flow attenuation can be provided while also providing approximately 134,000 gallons of emergency storage above the working level. The liquid levels in the two wet wells are the same, except the bottom of the main wet well is lower than the equalization wet well. The original wet well has a nominal capacity of 18,000-gallons. The original wet well is divided into two chambers and provides 9,900 gallons of emergency storage above the working level. Under normal, low flow conditions (0.6 MGD), there is one foot (28,000 gallons) of wastewater in the bottom of the equalization wet well and 6,000 gallons in the original wet well. Under these conditions, there is a total hydraulic retention time of 1.37 hours

Three 150-horsepower (hp) Ingersoll Dresser centrifugal pumps installed in the basement of the existing pump station building, draw suction from the smaller wet well and discharge wastewater to the WWTF via a 2.5 mile long 18-inch force main. The pumps utilize variable frequency drives and level sensors with the wet wells to match the pumping rate to the influent flow rate to the pump station. The current operation of the system maintains a level of approximately 5.5 feet in the original wet well which results in a level of approximately 1.5 feet in the equalization wet well.

Condition

Overall, the equipment and facilities at the Main Pumping station are in working order. The headworks room at the pump station is in generally good condition; however, some signs of age and corrosion were observed. The rotary screen is 18 years old, and while it is not currently exhibiting any issues, it may reach the end of its useful life in the next five to ten years. Operations staff have indicated occasional operation and maintenance issues in the screening building due to the buildup of rags.

The grit removal equipment is approximately eight years old and if maintained should have ten or more years of useful life remaining. The classifier cyclone was recently replaced, but otherwise has operated well.

While the new wet well is only ten years old, the original wet well is nearly 20 years old. The old wet well was drained and rehabilitated as part of the 2010 work, but no structural work was included in the construction contract. Since no significant concerns regarding the wet well's condition were documented at that time, the wet well is likely in adequate condition and should be inspected regularly.

The centrifugal pumps and motors are original to the plant; however, the pumps are being rebuilt as part of the WWTF maintenance program. One pump was rebuilt in 2017, another pump was being rebuilt as of the time of this report in 2018, and the final pump is due to be rebuilt in 2019.

Performance and Design

Table No. 3-1 below includes a summary of the design criteria for each major process component of the Main Pumping Station as provided by the manufacturer.

Table No. 3-1
Main Pumping Station Equipment Summary

Rotary Screen	
Unit:	1 x Lakeside- 63 FS-0.250-122
Design Capacity:	8875 gpm (6.16 MGD)
Screen Opening Size:	0.25 inches (~6mm)
Redundancy:	Manual Bar Rack
Vortex Grit Chamber	
Unit:	1 x Smith & Loveless PISTA 7.0 Concrete
Dimensions:	10' Diameter, 4.75' chamber height, 270-degree inlet
Grit Paddle:	1 hp, 1800 rpm
Capacity:	6.2 MGD
Grit Removal:	
Inlet Velocity:	0.5 – 3.1 ft/s
Detention Time:	5.2 – 31 seconds
Grit Pump	
Unit:	1 x Smith & Loveless PISTA Turbo Pump 4B2H
Design Point:	250 gpm @ 41.8 ft
Horsepower:	10 hp
RPM:	1800 RPM
Impeller Diameter:	8 1/8"
Grit Classifier	
Unit:	Smith & Loveless Model 15
Cyclone:	250 gpm Smith & Loveless Grit Concentrator
Centrifugal Pumps	
Unit:	3 x Ingersoll-Dresser 6MFC18-FR6A, 2 duty, 1 standby
Design Point:	3250 gpm @ 114 ft TDH
Efficiency:	~ 77% @ Design Point
Horsepower:	150 hp
Design flow 2 pumps pumping:	4,800 gpm (2,400 gpm each)
RPM:	1780 RPM
Impeller Diameter:	15.04"
Old Wet Well	
Dimensions:	10'x18', 7.7' height (from pump on to high alarm)
Working Volume:	6,700 gallons
Storage Volume:	9,900 gallons
New Wet Well	
Dimensions:	62'x62',6.25' height (from pump on to high alarm)
Working Volume:	28,000 gallons
Storage Volume:	151,000 gallons
Force Main	
Length, diameter, material	2.5 miles, 18" PVC

The capacity and design criteria for the headworks and pumping equipment were compared to standard design practices such as TR-16, 10 State Standards, and ASCE MOP 8. A summary of this analysis is provided below. The equipment is appropriately sized for a peak hourly sanitary flow of 6.2 MGD and the hydraulic profile for the Main Pumping Station is presented at 12.0 MGD on the 2009 upgrade plans.

The influent flow and I/I analyses in Section 2 determined flow to the WWTF based on a review of flows from the Main Pump Station to the WWTF, which are measured after the equalization wet well. Since the equalization wet well attenuates peak flows, the upstream equipment including the influent screen and grit removal equipment likely experience peak flows greater than those at the WWTF. To accurately determine the influent flow to the pump station, flow monitoring of the collection system is necessary upstream of the pump station. To estimate the impact of storm events on the pump station, the wet well levels were reviewed for the May 11, 2015 storm event evaluated in Section 2. The storm resulted in a peak hour flow of 4.5 MGD to the WWTF, with approximately 150,000 gallons of wastewater stored in the wet wells during the storm event. As noted in Section 2, this storm was not a five-year design storm, and larger storm and peak flows are possible. The 2038 projected peak hour flow entering the pump station is estimated to be equal to the design peak hourly sanitary flow plus the peak inflow rate. The peak hourly flow equals the ADF of 1.6 MGD times a peaking factor of 3.2 (source TR-16, Merrimack Curve), or 5.1 MGD. The 5-year peak inflow rate is 4.0 MGD. The combined peak design influent flow to the pumping station is 9.1 MGD. The pumping station has a total peak hourly flow capacity of 8.4 MGD. A 5 year storm is likely to cause an overflow.

Headworks Design:
Best design practices recommend installation of mechanically cleaned screens within NFPA 820 approved enclosures. The screens should be designed for peak flow and should have 100% redundancy. The screen installed at the Main Pumping Station is designed for a peak flow of 6.16 MGD, which is approximately equal to the 6.2 MGD design flow.

Grit removal is not compulsory for WWTFs unless the system includes combined sewers; however, use of grit removal is advised. The design and configuration of grit removal equipment is largely proprietary to the manufacturer, but there are some general design standards. The grit removal equipment should provide sufficient inlet velocities and detention times to provide 95% removal of particles passing a 65-mesh sieve size. The size of the unit can be evaluated based on the relative dimensions of inlet diameter, chamber diameter and chamber height. As presented in Table No. 3-2, the PISTA unit is designed to generally meet or exceed these criteria.

The PISTA system is designed for removal to the 100-mesh sieve size, which exceeds the 65-mesh standard. The unit diameter is 10 feet rather than the recommended 12 feet, and the approach velocity is less than the recommended 2 ft/s at low flow scenarios; however, the PISTA unit includes a mechanical paddle to increase velocity and improve performance through the unit, which is not accounted for in the design recommendations. Based on our engineering judgement, the PISTA unit is adequately sized for the design and projected flows.

Table No. 3-2
Main Pumping Station Headworks Design Standards

Design Standard*	Design Criteria	Provided in Design	Provided in Existing System	Provided at 2038 Projections
Rotary Screen				
Screen opening:	0.25-1.5”	0.25”	0.25”	0.25”
Approach Velocity:	2 ft/s @ peak flow	≤1.3 ft/s	≤1.3 ft/s	1.4 ft/s
Grit Removal System				
Removal goal:	95% at 65-mesh	95% at 100-mesh	Not Measured	Not Measured
Inlet Channel velocity:	2-3 ft/s for flow 40-80% of Peak; Min. of 0.5 ft/s	80% Peak:4.8 ft/s 40% Peak: 3.7 ft/s	ADF: 2.5 ft/s	ADF: 2.6 ft/s
Influent Channel L:W	7:1	10:1	10:1	10:1
Effluent Channel W: Influent Channel W	2:1	2:1	2:1	2:1
Chamber Diameter	12-13 feet	10 feet		
Chamber Height	4-5 feet	4.75 feet		
Detention Time:	20-30 seconds at peak	12 sec @ Peak	17 sec @ Peak	16 sec @ Peak

*Design standards used to evaluate the Headworks design included: TR-16, EPA Screening and Grit Removal Fact Sheet, EPA Preliminary Treatment Facilities Design and Operational Considerations, MOP 8, EPA The Swirl Concentrator as a Grit Separator Device.

The best indicator for headworks performance is observation of downstream processes. Operations staff do not currently record the volume of grit removed; however, the operators have indicated that the equipment performs well, and have not reported any issues with grit at the WWTF. Additional headworks equipment is provided at the WWTF including a Grit King grit chamber. The design and performance of that unit is discussed in detail later in this section.

Pump Station Design:

TR-16 includes comprehensive design standards for the design of wastewater pump stations. The Main Pumping Station design was compared to these standards to identify potential deficiencies. With the additional wet well and upgrades constructed in 2010, the pump station has adequate pumping, operating, and storage capacities for existing and projected flows. The facilities provide adequate redundancy, back-up power and meet NFPA 820 requirements. A summary of typical design standards for pump station equipment is provided in Table No. 3-3.

Table No. 3-3
Main Pumping Station Design Standards

Design Parameter*	Design Criteria	Provided in Design	Provided in Existing System	Recommended at 2038 Projections
Pumps				
Capacity:	Peak with largest out of service after flow equalization (5.5 MGD)	4.8 MGD	4.8 MGD	5.1 MGD
Max Starts per hours**:	< 4.5	N/A (VFDs)	N/A (VFDs)	N/A (VFDs)
Runtime per day:	N/A	@ADF:14 hrs at full speed, 24 hrs at 30% speed	@ADF:6 hrs at full speed, 18 hrs at 30% speed	@ADF:6 hrs at full speed, 19 hrs at 30% speed
Wet Wells				
Operating Range:	≤ 30 minutes	47 minutes	47 minutes	< 30 minutes
Emergency Storage:		151,000 gallons	151,000 gallons, 3.6 MGD Surge	151,000 gallons 3.6 MGD Surge
Detention Time (Min Pumping Rate):		2.2 Hours		
Force Main				
Velocity:	≥3 ft/s	4.1 ft/s at pump full speed, 1.2 ft/s at 30% speed		
Detention Time (ADF):	<2 = short, 2-5 = medium. >5 = long	2.6 hours	4.0 hours	2.6 hours
Detention Time (Min Hourly Flow (0.4 MGD):		11 hours	13 hours	11 hours

*Design standards used to evaluate the Pump Station design included: TR-16
** Based on NEMA guidelines for 150 hp 4-pole pumps and ADF; Min cycle time occurs when Q=1/2q

Based on a review of the WWTF influent flows, it appears that the large wet well is providing peak flow attenuation and equalization. It is possible the reduced peak flows at the plant during the last three years are a result of this attenuation, which can mitigate the impacts of I/I on the WWTF. The additional wet well storage also provides flexibility for future flows.

The operators have indicated that the performance of the pump system is adequate, and are pleased with the current set points, wet well cycles times and pump cycles times. Occasionally, an out of service pump will develop an air lock as air becomes trapped in the volute. Tata & Howard completed an evaluation of the wet well fill and drain times as well as the pump run times and start/stop times. The fill time for the wet wells is longer than the design standards. This could allow wastewater to age and potentially turn septic.

There have been hydrogen sulfide corrosion issues downstream at the WWTF including observed corrosion of the headworks building and equipment, corrosion of valve operators at the SBR splitter box, and failure of the ductile iron pipe between the headworks and the process tanks, and just upstream of the Grit King. Except for a short section near the WWTF, the force main is PVC and corrosion resistant. Hydrogen sulfide is generated in oxygen deprived conditions, which can be present in pump stations, tanks, and force mains with long detention times. The total detention time for the equalization wet well, original wet well and force main at the minimum pumping rate of 400 gpm is approximately 10 hours. This detention time may be extended during period where the pumps are not operating. Review of recent flow charts at the Main Pumping Station indicate that over the last three years the station has been operated continuously with fewer instances of the pumps turning off and wet well level rising. This operation should help mitigate hydrogen sulfide generation. Alternatives for reducing hydrogen sulfide impacts will be evaluated in the alternative evaluation later in the report.

In conclusion, the equipment at the Main Pump Station is adequately sized for current and projected flows and is performing well. The Operations staff should continue to monitor the grit generation for the PISTA unit and sewage age in the large wet well.

3.3.2 WWTF Headworks

Process Description

Flow from the main pump station flows to the WWTF where it is combined with flow from a few local industrial users and sanitary flow from WWTF facilities, such as bathrooms and laboratory sinks.

The WWTF flows through the Headworks building which includes a vortex grit chamber. The grit chamber at the WWTF is a Grit King as manufactured by HIL Technology and is designed using the same principles as the PISTA chamber; however, the Grit King relies solely on the inlet configuration to induce the vortex flow pattern, whereas, the PISTA unit has a rotating paddle to facilitate the desired conditions. The removed grit is processed through a US Filter grit classifier operating in the same fashion as the unit at the Main Pumping Station.

Condition
The headworks equipment at the plant is generally in working order however, the condition of the equipment range from fair to poor due to age and hydrogen sulfide corrosion.

The grit removal equipment is original to WWTF; therefore, it almost 20 years old and will reach the end of its useful life in zero to five years. The metal vessel structure of the Grit King is severely corroded.

The headworks building itself is showing considerable signs of corrosion. In several locations, metal components such as ductile iron pipe, unistrut support members, door frames etc., have surficial corrosion and oxidation. At the time of the site visit, it did not appear structural failure was imminent; however, if not addressed, the corrosion will continue to deteriorate the equipment and infrastructure in the building. The observed corrosion can be attributed to the generation of hydrogen sulfide gas at the upstream Main Pumping Station and the ventilation system operating at a reduced capacity and the odor control system being offline. Additionally, the reduced ventilation may compromise the NFPA 820 explosion proof environment.

Performance and Design
Table No. 3-4 below includes a summary of the design criteria for each major process component of the Headworks as provided by the manufacturer.

Table No. 3-4
WWTF Headworks Design Criteria

Vortex Grit Chamber	
Unit:	1 x Grit King by HIL Technology
Dimensions:	108” Diameter, ~5’ chamber height
Capacity:	6.2 MGD
Grit Removal:	95% at 106 microns (about 150 mesh)
Grit Classifier	
Unit:	US Filter
Capacity:	44 cu.ft per hour

The capacity and design criteria for the headworks equipment was compared to standard design practices such as TR-16, 10 State Standards, and guidance and design documents from regulatory agencies. A summary of this analysis is provided in Table No. 3-5.

Table No. 3-5
WWTF Headworks Design Standards

Design Standard*	Design Criteria	Provided in Design	Provided in Existing System	Provided at 2038 Projections
Vortex Grit Chamber				
Removal goal:	95% at 65-mesh	95% at 106 microns (about 150 mesh)	Not Measured	Not Measured
Inlet Channel velocity:	2-3 ft/s for flow 40-80% of Peak; Min. of 0.5 ft/s	80% Peak:4.8 ft/s 40% Peak: 3.7 ft/s	ADF: 2.5 ft/s	
Influent Channel L:W	7:1	7:1	7:1	7:1
Effluent Channel W: Influent Channel W	2:1	2:1.5	2:1.5	2:1.5
Chamber Diameter	13-18 feet	9 feet		
Chamber Height	3-4.5 feet	5 feet		
Detention Time:	20-30 seconds at peak	33 sec @ Peak	46 sec @ Peak	33 sec @ Peak
Grit Classifier				
Capacity:	44 cu.ft per hour			
Production:	< 44 cu.ft per hour		20-40 cu.ft per week	20-40 cu.ft per week

The Grit King unit was evaluated using the same criteria as the PISTA unit, as discussed in the previous section. Although the calculations indicate that the Grit King is slightly smaller, the proportions of the diameter and height are appropriate, and the unit is in series with another grit chamber; therefore, the size of the unit is likely sufficient.

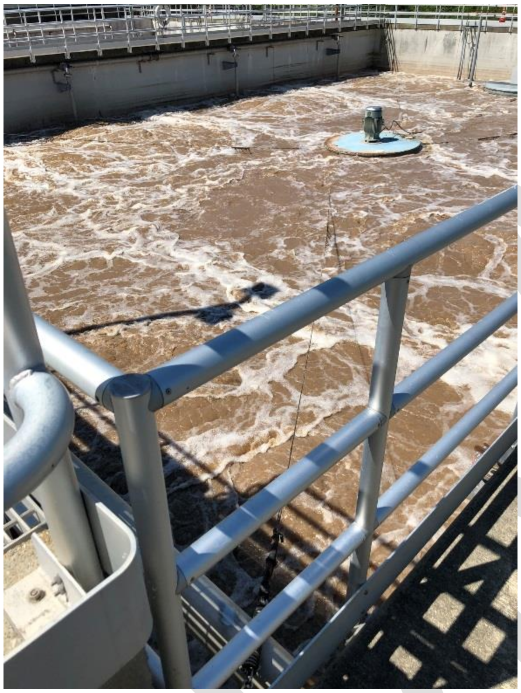
The Grit King is designed for removal to the 150-mesh sieve size, which exceeds the 65-mesh standard. The unit diameter is 9 feet rather than the recommended 12 feet, however, the diameter to height ratio is consistent with design standards. The approach velocity exceeds the recommended 2 ft/s at all flow scenarios. Based on our engineering judgement, the Grit King is adequately sized for the design and projected flows.

Since there is grit removal at the Main Pumping Station, the equipment at the plant is redundant. If adequate grit removal is achieved at the pump station, the grit system at the WWTF could be removed. This would reduce the energy and man-power used. Additionally, 19 feet of pressure head could be eliminated by removing the need to pump up to the grit chamber, resulting in an effective decrease of 3 brake horsepower for the Main Pump Station pumps. It is recommended that grit generation be monitored and recorded at both the pump station and the WWTF to confirm adequate removal is achieved.

3.2.3 Sequencing Batch Reactor Tanks

Process Description

Flow continues from headworks to the Sequencing Batch Reactor (SBR) tanks via a splitter box which diverts the flow to any of four tanks configured in parallel using automatically actuated valves. In addition, the Belt Filter Press filtrate drain returns filtrate to the process stream just before the SBR splitter. The filtrate becomes blended into the influent flow and contains BOD, TSS and P.



The SBR tanks are designed as part of an AquaSBR system as manufactured by Aqua Aerobics. The treatment process is proprietary to the manufacturer's equipment and process control system; however, the overall design and performance are consistent with typical SBR plants.

The SBR process consists of a large concrete structure divided into four equal tanks. Each tank is 82 feet long, 55 feet wide, and have a liquid depth that varies from 11.2 to 18 feet deep. The liquid level in the tank varies throughout the cycle as controlled by the SCADA based on opening and closing the influent and effluent valve in accordance with pre-programmed cycle settings.

The tanks provide aeration using a fixed Ott fine-bubble membrane aeration system in each tank. Air is provided using five (four duty, one standby) positive displacement rotary lobe blowers. The 100 hp blowers turn on and off with pre-programmed cycle settings in the SCADA. The operation of the blowers is constant speed. The number of blowers operating is controlled with real-time dissolved oxygen (DO) level in each tank as measured using DO probes.

Mixing during anoxic stages is accomplished using submersible Aqua-Aerobic mixers. The 20 hp mixers turn on and off with pre-programmed cycle settings in the SCADA.

Waste sludge is transferred from the SBR tank to the sludge holding tank using a 3 hp Flygt pumps installed within each tank. The pumps operate at a constant speed and turn on and off with pre-programmed cycle settings in the SCADA.

Effluent flow is controlled using floating decanters with automatic effluent valves. The valves are opened and closed based on pre-programmed cycle settings in the SCADA. The floating decanter configuration maintains a constant flow over a circular weir with a peak discharge flow of 6.2 MGD.

An inventory of equipment is provided in Table No. 3-8, and a description of the pre-programmed cycles is included below.

SBR Cycles:
The SBR process is designed to provide biological treatment of the wastewater. The SBR process is an alternate form of the activated sludge process, where multiple stages of treatment are conducted sequentially in a single tank. Each active SBR tank completes five batches per day, and each batch completes the following stages: mixed-fill, react-fill, react, settle, decant/waste.

With five batches per tank per day, each cycle has a duration of 4.8 hours. Each stage has a duration of approximately one hour.

The SBR process is designed for biological phosphorus removal. The influent Total Phosphorus to the WWTF has been as high as 24.2 mg/L, which is significantly higher than the 6.0 mg/L design loading. The process alternates aerobic and anoxic conditions to improve the amount of phosphorus taken up by bacteria. The biological process is able to remove up to 20.0 mg/L of phosphorus and Alum is added to provide additional removal to achieve the 0.8 mg/L permit limit.

Mix-Fill Stage:
The first half of the cycle includes the fill stage. For the first half of the fill stage, the tank is mixed, but not aerated creating anoxic conditions. During this stage, the raw wastewater becomes completely mixed with the solids remaining in the tank from the previous batch. The oxidized nitrogen from the previous batch is reduced; however, denitrification does not occur since the raw wastewater has not experienced nitrification. The anoxic conditions also allow phosphorus in the solids retained from the previous batch to be released and mixed into the blended wastewater.

React-Fill and React Stages
The second half of the fill stage activates the aeration system providing dissolved oxygen. The tank remains mixed and transitions from anoxic to aerobic enabling organic reduction and nitrification. The tanks remain mixed and aerated through the react phase, but no additional raw wastewater enters to tank to facilitate further biological organics reduction. The system has the ability to cycle the aeration to alternate oxygen-rich and oxygen-deficient conditions to achieve denitrification. Since the Middlebury plant does not have effluent limits for nitrogen, the aeration system remains continuous for additional polishing of the organics reduction and nitrification. Alum is added to the SBR tanks at the end of the react stage to allow for coagulation of phosphorus, which will settle during the next stage.

Settling Stage:
During the settling stage, the mixers and aeration system are deactivated. No additional raw wastewater enters the tank during this stage. The quiescent conditions allow the solids to settle.

The solids-liquid separation achieved in the settle stage allows for the supernatant to be decanted from the surface of the tank at a rate matching the rate of that cycle's fill stage.

Decant/Waste/Idle Phase:

The decant stage for this plant is set for 44-minute duration to nearly an hour, to minimize the flow rate to the downstream disinfection system. The operators adjust the duration when large rain events are anticipated Since the decant volume is equal to the fill volume, extending the decant cycle time reduces the effective flow rate to the UV system providing some flow attenuation. Finally, the solids accumulated during the treatment cycle are wasted to the sludge holding tank during the end of the decant cycle. The wasting rate is set by the operators to maintain a target Mixed Liquor Suspended Solids (MLSS) concentration in the SBR.

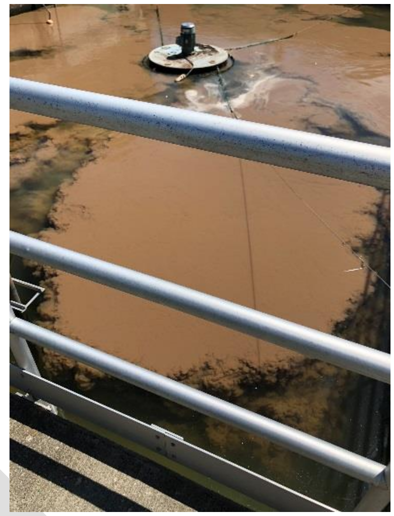
Condition

Overall, system is in good condition and is operating well. Each year, one SBR tank is removed from service and the diffuser membranes and effluent valve are replaced. All other SBR equipment, including the blowers and pumps, are original to the plant and are therefore, almost 20 years old. The operators have observed a few operation and maintenance (O&M) related issues. A summary of the issues observed are listed below.

SBR O&M Challenges:

- Splitter box has corrosion and serviceability issues
    - The pipe from headworks to the splitter box has been replaced due to H2S corrosion of the original ductile iron pipe
    - Valve operators show signs of corrosion
    - Automatic valves are due for replacement and have severe space limitations making the work unreasonably difficult
- The SBR influent valve vaults provide O&M challenges
    - Vaults are confined space
    - Vaults contain sumps, but do not have pumps to remove collected water
    - Operators would prefer open channel rather than the existing configuration which has pipe running through the vault
- Effluent channel algae build up
    - Excessive algal growth can lead to performance issues for the downstream disinfection system; solutions such as covering the channel will be addressed in subsequent sections of this report
- The sludge pumps are located in SBR tanks
    - To service the pumps, the entire SBR has to be taken offline; solutions such as relocating the pumps will be addressed in subsequent sections of this report
- The aeration piping does not allow for balancing of flow between units
- The control system is outdated and due for updating
- The replacement of the Ott diffusers is difficult.
- The in-tank location of the SBR effluent control valve makes it difficult to service

Performance and Design



The design concepts for the SBR process are similar to other activated sludge plants. The performance of the plant was evaluated at design and projected flows and loadings. The original design calculations were completed using all four SBR tanks in parallel, which does not provide any redundancy in the process. The use of four tanks is in accordance with standard design practices; however, for short durations of time, WWTF staff take an SBR tank offline to conduct maintenance. Best practices allow for uninterrupted processing with a unit out of service; therefore, the process was also evaluated for its ability to treat flow through only three tanks.

The evaluation showed that even with one SBR tank out of service, the capacity of the SBR unit process is 1.6 MGD. Table No. 3-6 presents the loading scenarios that were evaluated.

Table No. 3-6
SBR Influent Loading

Parameter	Design Loading	2017 Loading	2038 Projected Loading	Reserve
Maximum Monthly Flow (MGD)	2.72	1.25	1.32	1.40
Maximum Daily Flow (MGD)	4.60	2.80	2.94	1.66
Peak Hourly Flow (MGD)	6.20	5.50	4.6	1.6
Biochemical Oxygen Demand (mg/l)	388	473	473	-
Biochemical Oxygen Demand (lbs/day)	8,802	5,542	5,819	2,983
Total Suspended Solids (mg/L)	258	215	215	-
Total Suspended Solids (lbs/day)	5,853	1,832	2,731	3,122

Table No. 3-7
SBR Performance Criteria

Parameter	Design Loading	2017 Loading	2038 Loading	2038 Loading with Train out of Service
Hydraulic Retention Time (hr) @ MMF	13.3	34.9	33.2	24.9
Hydraulic Retention Time (hr) @ PHF	5.0	8.1	5.9	4.4
MLSS (mg/L)	3,400	2,800	4,122	4,122
F:M	0.15	0.12	0.15	0.15
Oxygen Required* (lbs)	15,263	8,579	11,713	11,713
Air Required per Tank (scfm)	2500	2100	3000	4000
Sludge Wasting Rate** (gpm)	205	205	205	205
Sludge Retention Time (days)	9-14	7-12	7-12	7-12

*Oxygen required for organics reduction and nitrification.
** Design WAS rate was 2/3 of solids, Current operation WAS is ¾ of solids

Table No. 3-8
SBR Equipment Design Criteria

SBR Tanks	
Quantity:	4
Dimensions:	82’ x 55’ feet wide, liquid depth varies from 11.2 to 18’
Volume at Min Level:	1.51 MG
Volume at Max Level:	2.43 MG
Volume required for Design MLSS	1.51 MG
Aeration System	
Aeration System:	Aqua-Aerobic Fine Bubble Membrane Diffusers
Dimensions:	8 sets of diffusers per tank, 20’x 4’8” each
Immersion Depth:	14.2 feet
Diffuser efficiency:	1.5% per foot of immersion (21.3% total)
Blowers	
Unit:	Roots 616-RAM-J Rotary Lobe
Quantity:	5 (4 duty, 1 swing)
Horsepower:	100 HP
RPM:	1750
Capacity:	1250 scfm @ 9.6 psi each
Air Required per Tank (scfm)	944 (at design loading)
SBR Mixer	
Unit:	Aqua-Aerobic FSS Aqua DDM Floating Mixer
Capacity:	High Volume
Horsepower:	20 HP
RPM:	88
HP required for mixing:	15HP (25HP per MG for DDM mixer)
WAS Pumps	
Unit:	Flygt CP-308S 4” Submersible, 434 impeller
Capacity:	205 gpm @ 26 ft TDH
WAS Rate Required:	205 (at design loading)
Horsepower:	3 HP
RPM:	1700

A review of the last five years of effluent data indicated that the process is operating well. The facility has consistently met permit and has achieved BOD and TSS removal rates of 98 percent compared to the design goals of 92 percent and 88 percent, respectively. These results confirm the operator’s observation that the SBR process is operating well. The SBR process also provides biological phosphorus removal. Although, the process has been optimized for biological phosphorus removal, WWTF staff have indicated Alum addition is used for phosphorus removal goals and also improves the sludge characteristics. The plant has consistently achieved effluent P concentrations of less than 0.5 mg/L. The plant does not currently have a Total Nitrogen limit.

As demonstrated in Tables No. 3-6 through 3-8, the SBR process and related equipment are adequately sized to meet the design loading, the current loading, and the projected 2038 loading with one SBR out of service, while keeping operating conditions within the design ranges.

The tanks, blowers, pumps are all sufficient to handle the flow, even with one train out of service:

- Mixers: Provide 25 hp per MG, 15 hp per MG recommended
- Blowers: Provide 4,800 scfm of air (duty), 3,776 required for worst case loading (design loading)
- WAS pumps provide 250 gpm, which equals worst case loading (design loading)

The actual flows and loadings to the plant are lower than the design; therefore, the hydraulic retention time (HRT) is longer than typical for an activate sludge process. The HRT at current flows is approximately 35 hours; which, is within the typical range of 15 to 40 hours, but is longer than the design value of 13 hours. The longer HRT is consistent with an extended aeration process, which is a more energy extensive process than is required for treatment at the Middlebury plant.

The longer HRT and lower flows and loading also impact the Sludge Retention Time (SRT), MLSS concentration, and food to microorganism (F:M) ratio. Given the fixed volume of the four SBR tanks, maintaining the design values of F:M and MLSS would lead to excessive SRT values. Extended SRT would lead to poor sludge characteristics which would adversely impact the solids handling process and disposal. As a result, the operators maintain a MLSS concentration of 2,800 mg/L rather than the design value of 4,500 mg/L. The operators also maintain a F:M of 0.2 which is greater than the design value 0.155. The MLSS concentration falls within the typical range of 1,500 to 5,000, and the F:M is within the typical range of 0.05 to 0.30. Opportunities to improve process efficiency through operational adjustments, such as taking tanks offline, or adjusting cycle times will be evaluated in the alternatives analysis.

Although it is not anticipated that the plant will have a nitrogen limit added to future permits near term, the SBR process has the ability and flexibility to achieve nitrification and denitrification to meet typical permit effluent levels. Improved nitrogen removal can be achieved by optimizing the aerobic and anoxic cycles of the process. This process would require more air and some operational changes, but otherwise, would not require any modification to the process. As discussed previously, the blowers and aeration system have adequate capacity to provide necessary air for nitrogen removal.

Lower influent flows and loading can lead to oversized equipment. A review of equipment sizing relative to the current flow show that the equipment has operational flexibility in place to mitigate potential performance issues due to oversized equipment. For example, the blowers could operate using VFDs to match air provided to the DO required based on DO set points within each SBR. Additionally, the blower discharge piping is configured such that each blower can provide air to any SBR tank and can provide air to multiple tanks; therefore, there is no need to replace or alter equipment to accommodate low flow conditions. However, no control of air flow rate or balancing between reactors is available.

Finally, the SBRs were evaluated to determine the available capacity for future commercial and industrial users in Town. The SBR process has a capacity of 2.72 MGD with all four tanks in service and 2.09 MGD with only three tanks in service, based on design average day BOD concentration of 515 mg/l and design maximum monthly BOD concentration of 389 mg/l.

Each new industrial user must be considered on a case by case basis to determine pretreatment needs and other constituent loadings such as phosphorus, ammonia nitrogen and metals.

3.2.4 Solids Handling

Process Description

Sludge wasted from the SBR tanks is stored in two sludge holding tanks adjacent to the solids handling building. The tanks have a capacity of approximately 200,000 gallons each, or 400,000 gallons total. Two 50 hp rotary lobe blowers provide air to a coarse bubble aeration system to keep the sludge well mixed. This aeration system is operated continuously. Timers for blower operation exist to improve efficiency, the of VFDs and DO control on the blowers may offer better process control and improved energy efficiency.

Two sludge transfer pumps convey the sludge for processing. The 15 hp sludge transfer pumps have a flow range from 14 to 140 gpm, and were both rebuilt in 2017. A flow of 140 gpm equates to approximately 650 pounds per hour of operation. The transfer pumps are started manually by the operators to begin the dewatering process. Two 3 hp inline sludge grinders are installed after the sludge pumps to provide uniform sludge for processing.

Sludge is processed in two stages. The first stage is dewatering the sludge by means of a belt filter press. The second phase involves conditioning the sludge through thermal blending and pasteurization of the dewatered sludge with lime to achieve Class A Biosolids for use in land application.

The plant uses two 1.5-meter Komline-Sanderson belt filter presses in parallel for dewatering. Polymer is added to the sludge to condition it, increasing the flocculation of solids improving dewatering. The Polyblend 3 polymer system include two 750 gallon dry batch systems that automatically mixes and doses polymer to the sludge prior to the belt filter presses.

The filter press achieves dewatering in two steps. First, the conditioned sludge travels through the gravity zone where the water can drain from the sludge through the filter belt. The unit installed in Middlebury includes an extended gravity zone, for additional dewatering. Next, the sludge travels through the pressure zone, where it is compressed between 1.5-meter wide belts and drums to achieve additional dewatering. The removed water and wash water have been modified from the original design to return by gravity flow to the SBR Influent Chamber. Wash water is provided

by two booster pumps connected to the potable water supply at the plant. The dewatered sludge is discharged to a conveyor belt where it is transported to the adjacent room for additional conditioning. The conveyor has a capacity of 10 tons per hour.

The EPA Part 503 Rules established requirements for the beneficial use and disposal of biosolids generated at WWTF. Middlebury has made beneficial reuse of their waste sludge a priority and have partnered with a local farm to reuse the sludge for land application. To achieve the Class A biosolids required, additional processing of the dewatered sludge is required. To meet Class A land application standards, the WWTF employs a high-pH, high temperature process as outlined by the Part 503 rule. This process includes raising the pH to 12 and raising the temperature to 50 degrees C (122 degrees F).

These parameters are achieved in a two-step process including a thermal blender followed by a pasteurization vessel. The thermal blender mixes the dewatered sludge with lime which is high in calcium and alkalinity resulting in a chemical reaction that releases heat. Additional heat is applied to maintain a temperature of 70 degrees C (158 degrees F) for a duration of thirty minutes. The lime is stored in a 50-ton lime silo adjacent to the sludge handling building. A lime screw conveyor transports the lime from the silo to the thermal mixer. A sludge recycle screw conveyor provides the option to recycle treated sludge back into the thermal mixer for improved performance The recycle function does not actually exist due to the conveyor's ability and location.

Following the thermal mixer, the treated sludge is held in the pasteurization vessel where the treated sludge remains at temperature for another 30 minutes as required by the Part 503 Rule. The Class A biosolids exits the pasteurization vessel and is conveyed to an adjacent garage. Operations staff use a loader to move and spread the final biosolids material to a biosolids storage area. The material is stored until hauled away to land application offsite.

Condition

The sludge holding tanks and associated pumps, blowers and equipment are in good working order with no observed issues; however, the solids handling building and equipment present operational challenges and the equipment is showing signs of age and requires repair.

The belt presses are operational and performing well. The units are almost 20 years old and are nearing the end of their anticipated useful life. The operators have indicated that several elements, including the bearings, are showing signs of wear. The units will require maintenance and/or rehabilitation in the next few years.

The sludge conveyors are missing guards which results in occasional discharge of dewatered sludge onto the building floor. Wet sludge sticks to the belts and there was more spillage than the guards could contain. The spilled quantities are manageable through equipment modification made by WWTF staff and require time to address.

The thermal blender and pasteurization equipment require repair or replacement. Parts of the equipment enclosure have been removed due to moisture issues and wiring is exposed. The conveyor belt, which provides the required retention, is losing flights. In addition to the mechanic issues, the lime addition system causes significant operation and maintenance challenges for the operations staff. Some examples of the observed challenges include:

- Sludge recycle for the lime addition system is non-operational
- The lime system air filter is inaccessible and overdue for replacement, replaced in 2019
- Lime mixing equipment is worn. An intermediate step to allow lime mixing indoors is preferred by operations staff
- Lack of redundancy/flexibility
- Conveyor guards removed because of the frequency of cleaning required
- Insufficient storage space is available for finished biosolids

The ventilation system in the solids handling building is not operated, except occasionally to clear lime dust from the room. The intake louvers are operated manually, and the odor control system is inoperable. Prior automatic operation of the ventilation system resulted in freezing of the louvers. Due to the environmental conditions for sludge handling and lime application, it is important that the ventilation system be rehabilitated.

Performance and Design

Each element of the solids handling systems is adequately sized to meet current and projected solids loading. The system is able to consistently achieve sludge with a solids content of 14 to 18 percent while also meeting the requirements for beneficial reuse of sludge through land application. The plant disposes 4,500 to 5,000 wet tons per year, which equates to approximately 900 dry tons per year.

The equipment limiting capacity of the processing system is the belt filter presses, which can each process 600 dry pounds per hour. Based on this loading rate and the current and projected sludge generation, both units must be operated in parallel to process the generated sludge within the 40-hour work week at design loading. Current operation is approximately 2-4 days per week, at 8.5 hours per day. The remaining equipment has the capacity to process up to 6 dry tons per hour. The staff currently operates the thickening process two to four days per week.

The sludge to be dewatered is from two primary sources: Septage and the SBR waste sludge. The SBR waste sludge is mostly solids removed from the wastewater during treatment, but also includes loads from the belt filter press filtrate drain.

Since the septage load represents a significant portion of the total sludge loading, the plant's ability to increase septage received in the future will be limited by the capacity of the sludge handling process. Septage receiving is discussed in further detail later in this section.

Table No. 3-9 presents a summary of the system elements and Table No. 3-10 compares the design of each system element to typical performance standards and regulatory requirements.

Table No. 3-9
Solids Handling Design Criteria

Sludge Holding Tank	
Quantity:	2
Dimensions:	65’ x 28’ 9’ depth
Volume at Min Level:	200,000 gallons
Days of Storage (Design sludge production):	6 days
Days of Storage (Avg sludge production):	8 days
Holding Tank Blowers	
Quantity:	2 (SBR swing blower can be used as 3rd sludge blower)
Unit:	Spencer RBLP-90L Rotary Lobe
Capacity:	1043 scfm @6.8 psi
Horsepower	50 hp
RPM:	1780
Sludge Transfer Pumps	
Quantity:	2
Unit:	Seepex BN52-6L/26-12
Capacity:	14-140 gpm
Horsepower:	15hp
RPM:	1765
Sludge Grinder	
Quantity:	2
Unit:	Franklin Miller TM 8512-06
Horsepower:	3
RPM:	1760
Belt Filter Press	
Quantity:	2
Unit:	Komline-Sanderson G-GRSL-1.5
Capacity:	600 dry lbs/hr
Horsepower:	3 hp
RPM:	1800 (233:1 ratio) to 7.7
Hydraulic Power Unit:	Rexroth- CK 065-10042DB 2.5 gpm @ 800 psi, 20 gal reservoir
Dewatered Sludge Conveyor:	
Unit:	Serpentix PW
Dimensions:	26” wide
Capacity:	10 ton/hr
Horsepower:	3 hp
RPM:	1750 (24:1 ratio) to 73
Lime Delivery System:	

Quantity:	1
Unit:	RDP- 95018
Capacity:	4,000 lbs/hr
Lime Storage:	42 tons
Thermoblender	
Quantity:	1
Unit:	RDP- 95018-106
Capacity:	12,890 lbs/hr (6.4 ton/hr)
Sludge pH (1st 2 hours):	>12.0
Sludge pH (1st 22 hours):	>11.5
Operating Temperature:	900 – 1,000 deg F
Total throughput Capacity:	258 ft^3/hr Biosolids
Total throughput Capacity:	1,600 dry lbs/hr
Pasteurization Unit	
Quantity:	1
Unit:	RDP- 95018-107
Capacity:	12,890 lbs/hr (6.4 ton/hr)
Hopper Volume:	392 ft^3
Retention Time:	30 Minutes
Throughput at Retention Time:	474 ft^3/hr Biosolids
Final Sludge Temperature:	158 deg F
Operating Temperature:	400 – 1,000 deg F
Finished Sludge Conveyor:	
Unit:	Serpentix PW
Dimensions:	26” wide
Capacity:	10 ton/hr
Horsepower:	3 hp
RPM:	1750 (24:1 ratio) to 73

Table No. 3-10
Sludge Dewatering/Conditioning Performance Criteria

Design Standard*	Design Criteria	Provided in Design	Provided in Existing System	2038 Projections
Sludge Generated by SBR* (gpd)		69,656	57,775	60,664
Sludge Generated by SBR* (lbs/day)		5,809	4,159 (72% of capacity)	4,367
Septage (gpd)		30,000 MDF	5,500 ADF 27,000 MDF	5,750 ADF 28,350 MDF
Sludge Loading per hour(lbs/hour)	Less than process capacity	1,270 @ 4 days/week	1,003 @ 4 days/week	1,054 @ 4 days/week
Sludge Process Capacity** (lbs/hour)		1,200		
Sludge Process Capacity at 8 hours/day (lbs/day)**		9,600		
Sludge Tank Storage (days)		2.0	3.2	3.0

*Includes Filtrate from BFP
** BFP is limiting process

Overall, the process is operating as designed, but there are several areas with potential for improvement. The dewatering onsite achieves 14 to 18 percent solids in the sludge, which is typical for sludge generated by processes performing biological phosphorus removal.

In addition, the age of the equipment suggests a need for mechanical upgrade for continued satisfactory performance. The sludge tanks, pumps and blowers operate in parallel and are therefore redundant. The belt presses are also installed in parallel; however, both units are operated together to process sludge within the 40-hour work week. Failure of one of the presses may result in weeks where the sludge cannot be processed within the staff's work week; therefore, the process does not provide adequate redundancy. The sludge conveyors, lime addition and mixing equipment, and sludge thermal mixing and pasteurization equipment consist of a single train. Failure of any unit or conveyor would render the whole system out of order. The lack of redundancy throughout the whole process results in limitations of the operational flexibility and poses a risk of significant down-time of sludge processing should the plant experience equipment failure. Down time of sludge processing would be costly as the non-processed sludge cannot be land applied. The Town would have to pay for hauling and disposal of the sludge.

Finally, the plant disposes the dewatered and conditioned sludge to a local agricultural customer. The disposal is conducted through a mutually beneficial arrangement; however, the Town does not have a formal agreement in place. Long-term, the lack of formal agreement could be a limiting factor in planning for future expansion of sludge and septage receiving and processing.

3.2.5 Disinfection

Process Description

The SBR effluent flows to the ultraviolet (UV) disinfection unit installed within a 42-inch wide channel. The UV disinfection uses short-wave ultraviolet light to deactivate microorganisms by disrupting their DNA. The system installed is a Trojan UV4000 which is a high-capacity, high-intensity, medium-pressure with self-cleaning technology. The system includes banks in series to provide redundancy. The disinfected effluent flows to the 30-inch outfall with a diffuser into Otter Creek.

Condition

Overall the UV system is currently operating and performing well; however, the system is almost 20 years old and is nearing the end of its useful life. Since the unit is at the end of its expected lifespan, support and purchasing replacement parts are challenging. The older technology requires more energy and has higher operation and maintenance costs than newer available systems. Staff have also noted that the control system is obsolete.

Performance and Design

Table No. 3-11 below includes a summary of the design criteria for the UV system as provided by the manufacturer.

Table No. 3-11
UV Disinfection Design Criteria

UV Disinfection	
Unit:	Trojan UV 4000
Capacity:	8.4 MGD (4.2 MGD per bank)
UV Dose:	24,000 Mw sec/cm^2
UV Transmission:	65%
Retention Time:	0.193 sec
Disinfection Requirements:	300 CFU/100ml E. Coli.

Table No. 3-12
Disinfection Performance Criteria

Design Standard*	**Design Criteria**	**Provided in Design**	**Provided in Existing System**	**Provided at 2038 Projections**
Peak Flow from SBR Decant (MGD)*		6.1	4.4	6.2
Detention Time		0.080 @ADF 0.136 @MDF	0.030 @ADF 0.098 @MDF	0.032 @ADF 0.103 @MDF
UV Dose (Mj/cm^2)	30			
UV Intensity (uW/cm^2)	To Achieve 30 Mj/cm^2	374 @ADF 221 @MDF	979 @ADF 307 @MDF	932 @ADF 292 @MDF

*Based on 4 tanks and 5 cycles per basin per day

Industry standards provide general guidance on UV dose, but ultimately recommend a system specific design in lieu of using standard values. The UV system designed for the Middlebury WWTF provides a UV dose of 24,000 Mw sec/cm^2 to achieve E. Coli less than 300 CFU/100ml E. Coli. at a UV transmission of 65 percent. The unit achieves this by adjusting the lamp intensity to match the influent flow rate. The UV system is operating well and the WWTF is meeting effluent limits for E. Coli. Standard design practice includes banks in series to protect against potential bank failure. The system provided includes two banks in series each rated for 4.20 MGD. This is the effective capacity of the facility, since it is calculated with the largest unit out of service. The hydraulic loading to the UV system is equal to the decant rate from the SBRs (6.2 MGD). The configuration of the SBR decanters is such that a constant decant rate is provided. The peak decant rate is equal to the hydraulic capacity of the decanter, which is 6.2 MGD for the Middlebury Plant. The UV system can provide redundancy for the average the design peak decant rate of 6.2 MGD. The actual peak flows to the WWTF are less than design because of the hydraulic limitations at the Main PS and the influence of the large wet well and the Main Pumping Station. A reduction in the discharge rate from the SBRs may be a means of mitigating the limitations of the UV units. Presently, there is no post-SBR flow equalization or flow control mechanism, to prevent the UV from experiencing peak flow rates.

Although the decant rate should be constant, the reported effluent flow varies significantly. The WWTF staff have indicated that the effluent flow measurement is unreliable. Effluent flow is measure using a 14-foot weir. Due to the length of the weir, small variations in the depth of flow result in significant variations in measured flow. Reliable effluent flow measurement is critical to the WWTF staff for plant operations. Understanding the effluent is also critical to monitoring the peak flow rates to the UV system. The alternatives evaluation later in this report will evaluate options for addressing accurate effluent flow measurement.

3.2.6 Septage Receiving

Process Description

The Middlebury WWTF receives and treats septage from local haulers. The NPDES permits requires the WWTF to receive up to 8,800 per day of septage. Trucks discharge septage into two 15,000-gallon septage storage tanks. The septage is processed through a Lakeside septage receiving unit. The septage receiving unit includes a rotating mechanical screen to remove debris and large solids. The removed solids are collected in a receptacle. The screened septage is stored in the septage tanks where submersible mixers keep solids suspended. Design of the septage system originally included pumping to the SBR tanks for treatment; however, due to performance issues in the sludge process, the septage pumps now transport the septage to the sludge holding tank where the septage in blended into the WAS. In 2019 , the plant received a total of 325,000 gallons, and a maximum of 29,500 in one day.

Condition

The septage receiving system is almost 20 years old and is approaching the end of its anticipated useful life. The unit does not include a rock trap, and as a result, the unit has been damaged and repaired. Due to the age of the unit, replacement parts are no longer available, and future damage could result in the need for replacement. The system lacks a rock trap and is susceptible to damage from large objects discharged to it.

The septage truck unloading area does not include containment, so a spill or truck failure would result in release of the raw waste. There is a need for stormwater drainage improvements in the receiving area.

Performance and Design

The septage receiving system operates well and has adequate capacity for current septage loading, and excess capacity is available for the future. Unlike other processes at the WWTF, future loadings of septage are optional. The equipment capacity was evaluated for its ability to handle projected flows proportional to the anticipated growth in town; however, if the customer base exists, the Town may elect to use excess capacity at the plant to generate revenue.

To determine the financial impact of expanding septage receiving, the cost to treat and dispose of the septage was compared to the revenue generated. Septage at the plant is pretreated and then discharged to the sludge holding tank bypassing the SBR processes; therefore, the cost to treat the septage is approximately equal to the cost to process sludge at the plant, $0.03 per gallon. Based on Fiscal Year 2017 budget data, the revenue for septage is approximately $0.08 per gallon resulting in a surplus of $0.05 per gallon.

The limiting factors for the potential revenue are the customer base and the capacity of the sludge handling equipment. As subsequent sections of this report explore alternatives for sludge processing improvements, the potential for expanded septage receiving will be considered.

Table No. 3-13 presents a summary of process equipment and Table No. 3-14 compares the process design to standard design standards and regulatory requirements.

Table No. 3-13
Septage Design Standards

Septage Screen	
Unit:	Lakeside- 31SAP-0.250
Capacity:	2,060 gpm (2.97 MGD)
Max Day Loading:	27,000 gallons
Septage Tank	
Capacity:	30,000 gallons
Max Day Loading:	27,000 gallons
Septage Mixers	
Quantity:	2
Unit:	ITT Flygt SR 4630X
Horsepower:	2.5
RPM:	855
Septage Pumps	
Quantity:	2
Unit:	Komline Sanderson- KSK-7.5 Plunger Pump
Capacity:	30 gpm
Horsepower:	3hp
RPM:	1725

Table No. 3-14
Septage Performance Criteria

Design Standard*	Provided in Design	Provided in Existing System	Provided at 2038 Projections
Septage Received (gpd)	30,000 MDF	5,500 ADF 27,000 MDF	5,750 ADF 28,350 MDF
Storage (gallons)	30,000		
Screen Capacity (MDG)	2.97		

3.2.7 Site

The WWTF site is generally flat and includes some paved and unpaved roadways and parking area, lawn areas, perimeter fencing and stormwater systems. No problems were observed or reported except for a small drainage issue at the septage receiving station. A small amount of resurfacing to correct some poor drainage due to settling is recommended to correct this problem. The pavement shows minimal wear and some routine cracking. The cracking appears to be due to freeze/thaw and frost issues inherent in a Vermont location. Regular scheduled crack repair could prolong pavement life at minimal cost.

3.2.8 Administration/Operations Building

The Administration/Operations building (6,200 sf +/-) has offices, control room, locker room, bathroom, conference room, lab, boiler room, shop and chemical room. The Disinfection Facilities structure is immediately adjacent to the building and was included in this building review. Ample space is provided for the functional needs of the facility and staff, except for the shop. No structural problems with the building were observed or reported. The building plumbing, HVAC and electrical systems were reviewed during the site visit. The only noted issue with these systems is that the original air conditioning (AC) units are not operational. The units have been replaced with portable AC units and building ventilation is disabled in the summer. A permanent solution to building AC should be developed. The exterior of the building is in generally good condition. Trim painting and minor repair, as needed, are recommended.

3.2.9 Headworks/Solids Handling Building

The Headworks/Solids Handling building (16,000 sf +/-) has septage receiving, grit removal, generator, electrical, dewatering, lime stabilization and sludge storage. Ample space is provided for the functional needs of the facility and staff. No structural problems with the building were observed or reported, except that a steel cross-brace in the roof framing of the sludge storage area was observed. Replacement of the damaged cross-brace is recommended. The building plumbing, HVAC and electrical systems were reviewed during the site visit. The only noted issue with these systems is that the building ventilation is operated intermittently and should be operated continuously when occupied to provide the appropriate environment for staff. An odor control system is integrated with the building exhaust systems. A permanent solution to building ventilation should be developed. The exterior of the building is generally good. Trim painting and minor repair, as needed, are recommended.

3.2.10 Sludge Building

The Sludge building (3,600 sf +/-) has a blower room comprising the upper level and a sludge pump room in the lower level. Ample space is provided for the functional needs of the facility and staff. No structural problems with the building were observed or reported. The building plumbing, HVAC and electrical systems were reviewed during the site visit. There are no noted issues with these systems in this building. Trim painting and minor repair, as needed, are recommended.

3.2.11 Vehicle Storage Building

The Vehicle Storage building (1,200 sf +/-) is a two-bay garage. Insufficient space is provided for the functional needs of the facility and staff. The building is used to store large spare process equipment and other items. Shop, equipment, and vehicle storage needs for the entire WWTF should be further assessed. No structural problems with the building were observed or reported. The building lacks plumbing and HVAC systems. There are no noted issues with this building. Trim painting and minor repair, as needed, are recommended.

3.2.12 All Buildings HVAC

The HVAC systems are 20 years old, with many components reaching the end of their useful life. Either a comprehensive replacement of these systems or a capital improvements plan with longer-term replacement is recommended. Either of these options will accomplish energy savings as well. An important ventilation function is dealing with dust (lime) releases within the Sludge Processing building, since it is one ventilation function used by the operators regularly.

3.4 Summary of Facility Conditions

Process	Criteria	Recommended	Provided	Status
Main Pumping Station	Peak Hourly Flow	10.2 MGD	8.2 MGD	Adequate and reduces peaks to WWTF to 4.8 MGD
Grit Removal	Peak Hourly Flow	10.2 MGD	10.2 MGD	Redundant Grit Removal a WWTF to be abandoned
SBR	BOD Removal Peak Hourly Flow	4510 lbs/day 4.8 MGD	4510 lbs/day 6.2 MGD	Physical upgrades due to wear and aging of equipment
UV Disinfection	Peak Hourly Flow	4.8	4.2	Requires upgrade for redundancy
Outfall	Peak Hourly Flow	4.8 MGD	6.2 MGD	Adequate
Sludge Storage	Storage Time	10 days	10 days	Adequate
Dewatering	Processing Rate	600 lbs/hr	600 lbs/hr	Physical upgrades due to wear and aging of equipment
Residuals Management	Processing Capacity	4.8 dtpd	4.8 dtpd	Physical upgrades due to wear and aging of equipment
Site Buildings	Overall Condition	Good	Fair	Various Improvements Required

3.4 Financial Status of any Existing Facilities

The Town's Wastewater Treatment Budget has operated with a surplus for the period of 2013 through 2016. The budget includes multiple Debt Retirement line items, including the 1999 WWTF Bond. The annual payment on that bond is $409,165, representing approximately 14% of the total budget. The bond is scheduled to retire in 2022. The Town is operating the WWTF with fiscal responsibility and planning the next upgrade to follow retirement of its largest current WWTF debt budget line item.

3.5 Water/Energy/Waste Audits

An energy study of the WWTF was performed in 2014. A copy of the Energy Evaluation is included in APPENDIX ___. Several of the recommendations have been implemented and others will be considered as part of the upgrade improvement project. A list of recommended improvements is:

- Interlock SBR mixer with blower
- Reduce DO setpoint in SBR
- Switch to 2 basin SBR operation
- Reduce thermostat settings
- Throttle SBR Decant Flow
- Adjust UV Intensity
- Eliminate BFP Spray Water Pumps
- Add VFD’s to the blowers
- Complete sludge processing in off-peak hours
- Participate in Demand Response Program

SECTION 4 – NEED FOR PROJECT

4.1 Health, Sanitation, and Security

4.2 Aging Infrastructure

4.3 Reasonable Growth

SECTION 5 – ALTERNATIVES CONSIDERED

5.1 Description

A variety of alternative approaches to upgrading the WWTF are evaluated in this section. The evaluations are completed for sections of the facilities that may be evaluated separately. There is one upgrade component for which no alternatives are considered: elimination of the vortex grit unit (Grit King) at the WWTF. Primary clarifiers have been added as an upgrade component at the request of the Town. The purposes of consideration of the clarifiers is to increase organic loading capacity of the entire WWTF, reduce organic loading on the secondary biological process, increase energy potential of sludge for anaerobic digestions alternatives, and to improve sludge thickening and dewatering characteristics.

5.2 Design Criteria

The recommended design criteria for the upgraded facility are presented in Section 3. The basic design criteria are restated below in Table No. 5-1. Additional criteria for relevant individual components are provided in the section relative to those components.

Table No. 5-1
Design Criteria

Constituent	Units	Design Value		Permit
		Influent	Effluent	
Average Daily Flow	gpd	1,560,000	1,560,000	2,200,000
Maximum Monthly Flow	gpd	2,200,000	2,200,000	none
Peak Hourly Flow	gpd	6,200,000	4,800,000	none
Biochemical Oxygen Demand (BOD_5)	mg/L	520	30	30
	lbs.	6,800	390	390
Total Suspended Solids Daily (TSS) Monthly	mg/L	350	30	30
	lbs.	4,500	390	390
Total Nitrogen	mg/L	54	10	Monitor Only
	lbs.	700	130	none
Total Phosphorous Monthly Avg.	mg/L	10	0.6	0.800
Total Phosphorous Annual Avg.	lbs.	130	7.8	11
pH	SU	6.5-8.5	6.5-8.5	6.5-8.5

5.3 Map

5.4 Main Pumping Station

5.4.1 Description

Improvements to the Main Pumping Station consist of installation of a chemical feed system for odor and corrosion control in the force main. Alternatives such as modification of the collection or aeration of the wastewater at the pumping station were ruled out due to cost and inability to prevent hydrogen sulfide production in the force main. The production of hydrogen sulfide in the force main will still occur and require treatment for any alternatives implemented ahead of it. Chemical treatment is the only identified option for this problem. A schematic site plan for the improvements is presented in Figure No. 5-1.

The components of this item include:

- Chemical feed system
- Liquid odor/control chemical storage tank
- Process piping, electrical, instrumentation and safety improvements

5.4.2 Environmental Impacts

These improvements will be completed within the footprint of existing buildings and infrastructure. There will be no environmental impacts of new construction for this work.

5.4.3 Land Requirements

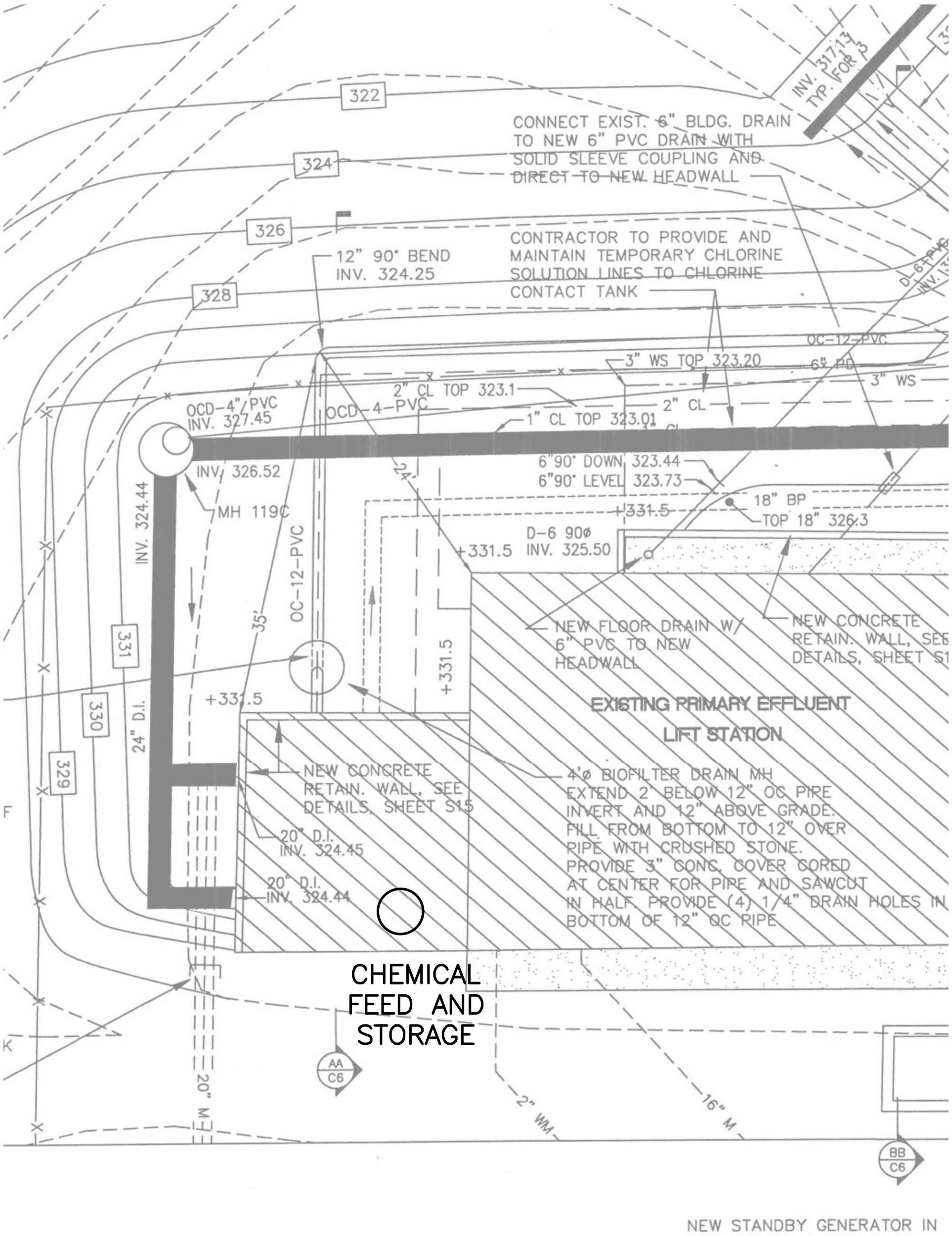
There are no land requirements for this project element.

5.4.4 Potential Construction Problems

There are no special construction problems identified for this work. There will be typical complications and difficulties associated with installation in an existing building and associated demolition.

5.4.5 Sustainability Considerations

The technology under consideration are proven from a sustainability standpoint.



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TATA & HOWARD

MIDDLEBURY WWTF
ODOR/CORROSION CHEMICAL SYSTEM

PRELIMINARY ENGINEERING REPORT
MAIN PUMPING STATION IMPROVEMENTS
MIDDLEBURY, VERMONT

Figure No.

5-1

5.4.6 Water and Energy Efficiency
These improvements will have no effect on water and energy efficiency.

5.4.7 Green Infrastructure
Implementation of green infrastructure is not applicable to this project element.

5.4.8 Operational Simplicity
This project element will increase operational simplicity by reducing corrosion of equipment and structures at the WWTF.

5.4.9 Opinions of Probable Cost and Life Cycle Cost Analysis
The Opinion of Probable Cost for the force main odor/corrosion control system at the Main Pumping Station is presented in Table No. 5-2. A life cycle cost analysis is not performed because there are no alternatives for comparison.

Table No. 5-2
Main Pumping Station Improvements Probable Estimated Cost

		Main PS Odor Control
Construction Cost		$124,000
Contingency	15%	$37,200
Other Project Costs	30%	$48,360
Total		$209,560

5.5 Headworks

5.5.1 Description
The existing headworks at the Main Pumping Station is deemed adequate for the planning period. Although, mechanical rehabilitation of the equipment is anticipated to be required within that time frame. The redundant grit removal system (enclosed swirl separator) will be abandoned. This construction will include the followed:

- Demolition, removal and disposal of existing swirl separator equipment.
- Modification of influent force main to a new discharge point at either the secondary treatment splitter box or a new flow control device associated with new primary treatment facilities.

5.5.2 Environmental Impacts

These improvements will be completed within the footprint of existing buildings and infrastructure. There will be no environmental impacts of new construction for this work. A slight energy savings from the improvements will have positive environmental impacts off site.

5.5.3 Land Requirements

There are no land requirement for this project element.

5.5.4 Potential Construction Problems

There are no special construction problems identified for this work. There will be typical complications and difficulties associated with installation in an existing building and associated demolition.

5.5.5 Sustainability Considerations

This project element removes a unit process which will lower the discharge pressure on the influent pumps located in the Main Pumping Station.

5.5.6 Water and Energy Efficiency

The revise piping arrangement and elimination of the swirl separator will reduce energy consumption by the pumps at the Main Pumping Station. The annual power savings is approximately $3,200 per year.

5.4.7 Green Infrastructure

Implementation of green infrastructure is not applicable to this project element.

5.4.8 Operational Simplicity

The elimination of a unit process will make WWTF operations simpler.

5.4.9 Opinions of Probable Cost and Life Cycle Cost Analysis

The Opinion of Probable Cost for the headworks demolition is presented in Table No. 5-3. A life cycle cost analysis is not performed because there are no alternatives for comparison.

Table No. 5-3
Headworks Demolition Probable Estimated Cost

		Headworks Demolition
Construction Cost		$150,000
Contingency	15%	$45,00
Other Project Costs	30%	$58,000
Total		$253,500

5.6 Primary Treatment

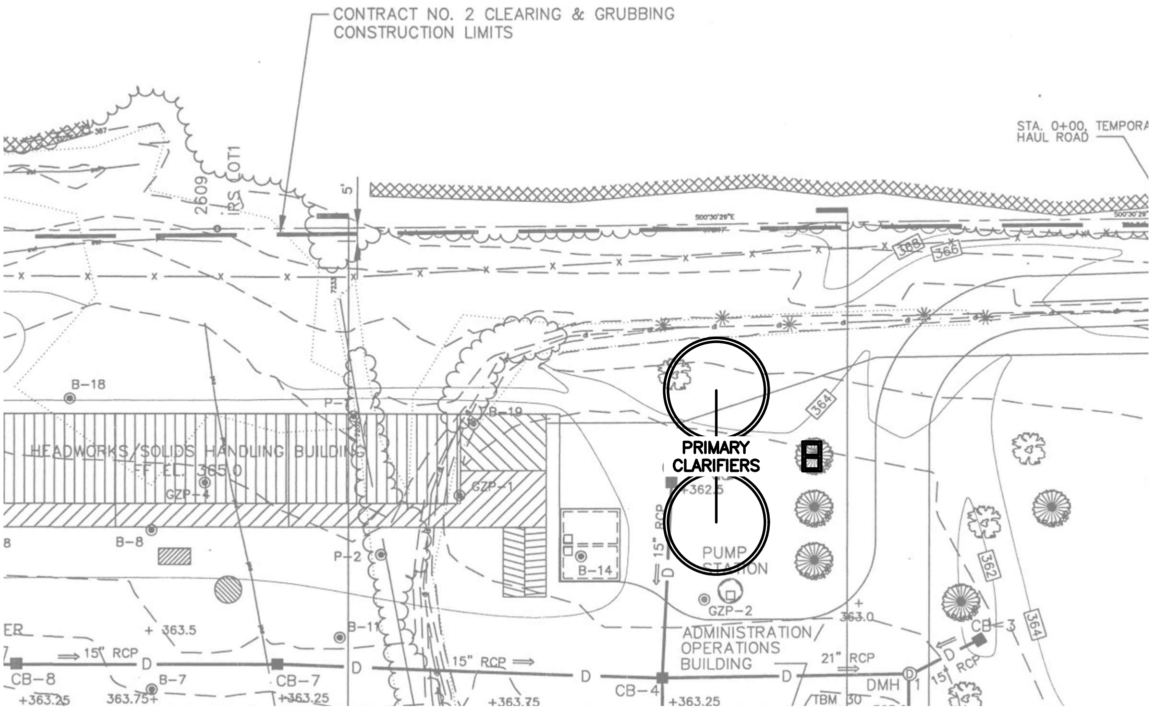
5.6.1 Description

The purposes of this project element are to increase energy potential of residuals for anerobic digestion and to reduce the organic loading on the biological treatment process. The reduction in organic loading will create reserve capacity within the constraints of the existing SBR tanks. Three options for primary treatment have been considered:

<u>Circular Primary Clarifiers</u> – Two-36 foot diameter clarifiers with a side water depth of 14 feet are proposed. Each clarifier will handle 75% of the peak hourly flow to provide redundancy in accordance with the requirements of TR-16. The clarifiers would be located just south of the existing WWTF Headworks Building. Associated construction include modified influent force main, primary clarifier splitter box, primary sludge pumps, electrical, instrumentation, and other piping improvements. A schematic site plan for the improvements is presented in Figure No. 5-2.

<u>Rectangular Primary Clarifiers</u> – This option is sized and configured the same as for the circular clarifier option. The size of the rectangular tanks would be 20 feet by 52 feet each, with a 14 foot side water depth. A schematic site plan for the improvements is presented in Figure No. 5-3.

<u>Mechanical Primary Treatment</u> – Three rotary screens, commonly known as drum filters, provide a similar function to the primary clarifier options. It is assumed that the drum filters will fit within the existing headworks area. The space appears to be adequate, however, if this option is pursued a further analysis will be required. Full capacity is provided with one unit out of service to satisfy the redundancy requirements of TR-16. A schematic site plan for the improvements is presented in Figure No. 5-4.



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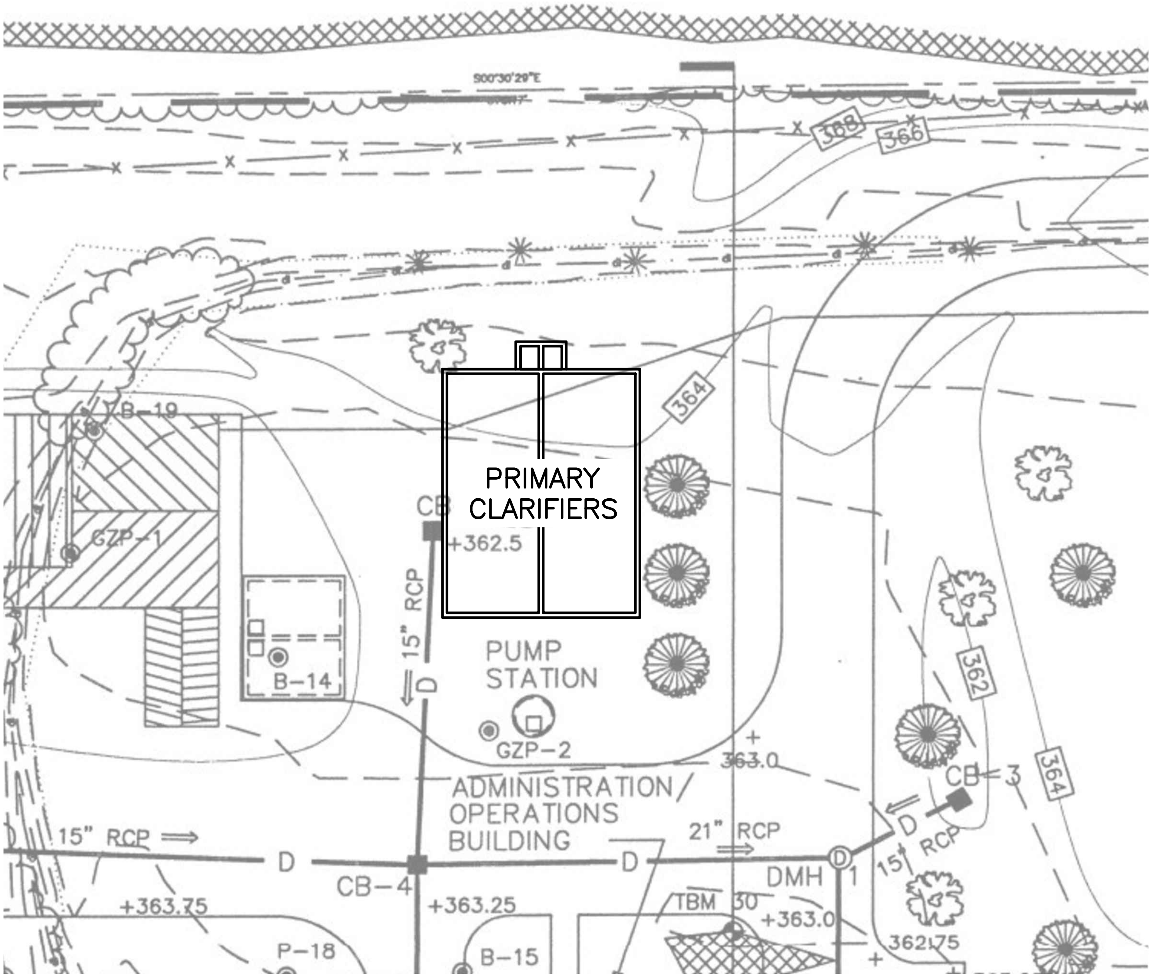
TATA & HOWARD

CONCEPTUAL SITE PLAN
CIRCULAR PRIMARY CLARIFIERS

PRELIMINARY ENGINEERING REPORT
PRIMARY TREATMENT OPTIONS
MIDDLEBURY, VERMONT

Figure No.

5-2



TATA & HOWARD

MIDDLEBURY WWTF
RECTANGULAR PRIMARY CLARIFIERS

PRELIMINARY ENGINEERING REPORT
PRIMARY TREATMENT OPTIONS
MIDDLEBURY, VERMONT

Figure No.

5-3

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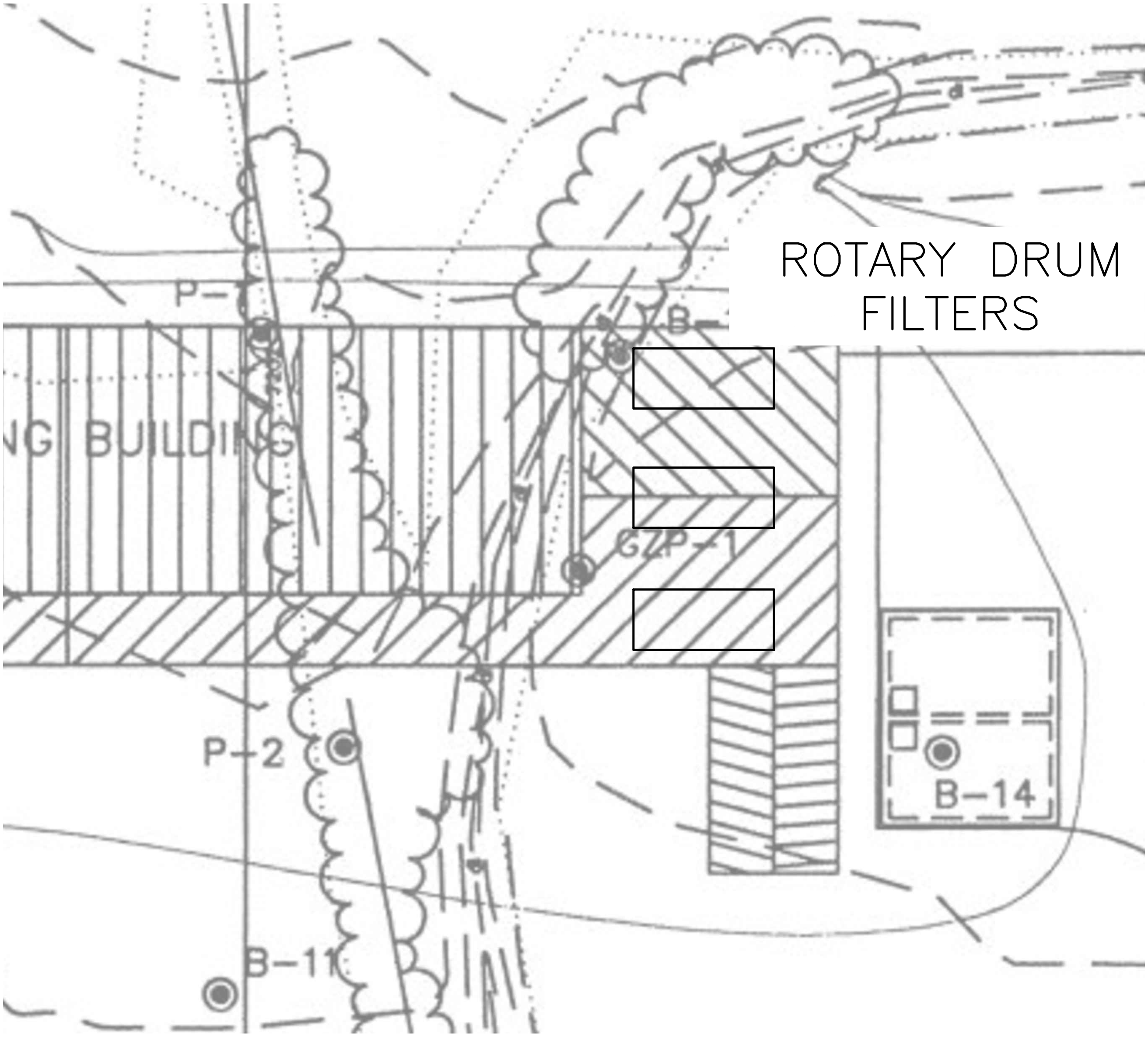


TATA & HOWARD

MIDDLEBURY WWTF
DRUMFILTER PRIMARY TREATMENT

PRELIMINARY ENGINEERING REPORT
PRIMARY TREATMENT OPTIONS
MIDDLEBURY, VERMONT

Figure No.

5-4

5.6.2 Environmental Impacts

The two new clarifier options will be built within the existing WWTF site and will therefore, have minimal environmental impacts. The improvements associated with mechanical primary treatment will be completed within the footprint of existing buildings and infrastructure. Although each alternative will create a new power load, net energy savings resulting from reduced power consumption by biological processes and increased energy production with the anaerobic digestion alternative will result in net positive environmental impacts.

5.6.3 Land Requirements

There are minimal land requirements for this project element. The land requirements for each alternative are approximately 10,000 sf for rectangular clarifiers, 13,000 sf for circular clarifiers, and none for mechanical primary treatment.

5.6.4 Potential Construction Problems

There are no special construction problems identified for this work. There will be typical complications and difficulties associated with installation in an existing building and associated demolition for the mechanical primary treatment alternative.

5.6.5 Sustainability Considerations

5.6.6 Water and Energy Efficiency

These alternatives will have no effect on water efficiency but will improve energy efficiency if the anaerobic digestion alternative is selected.

5.6.7 Green Infrastructure

Implementation of green infrastructure is not applicable to this project element.

5.6.8 Operational Simplicity

The addition of this unit process will reduce overall WWTF operational simplicity by adding a unit process. The processes themselves are simple when compared to other wastewater treatment processes. The energy efficiency and production benefits of primary treatment outweigh the added complexity of operations.

5.6.9 Opinions of Probable Cost and Life Cycle Cost Analysis

Table No. 5-4
Primary Treatment Options

	Option 1 Circular	Option 2 Rectangular	Option 3 Drumfilter
Probable Cost	$763,830	$573,150	$1,173,775
Contingency (15%)	$114,575	$85,973	$176,066
Other Project Costs (30%)	$263,521	$197,737	$404,952
SUBTOTAL:	$1,141,926	$856,860	$1,754,793
Power	$5,000	$5,000	$10,000
Labor	$6,240	$6,240	$12,480
	$11,240	$11,240	$22,480
PW Factor @ 2.8%	22.10	22.10	22.10
PW	$226,115	$226,115	$452,231
TOTAL	$1,368,041	$1,082,975	$2,207,024

5.7 Secondary and Advanced Treatment

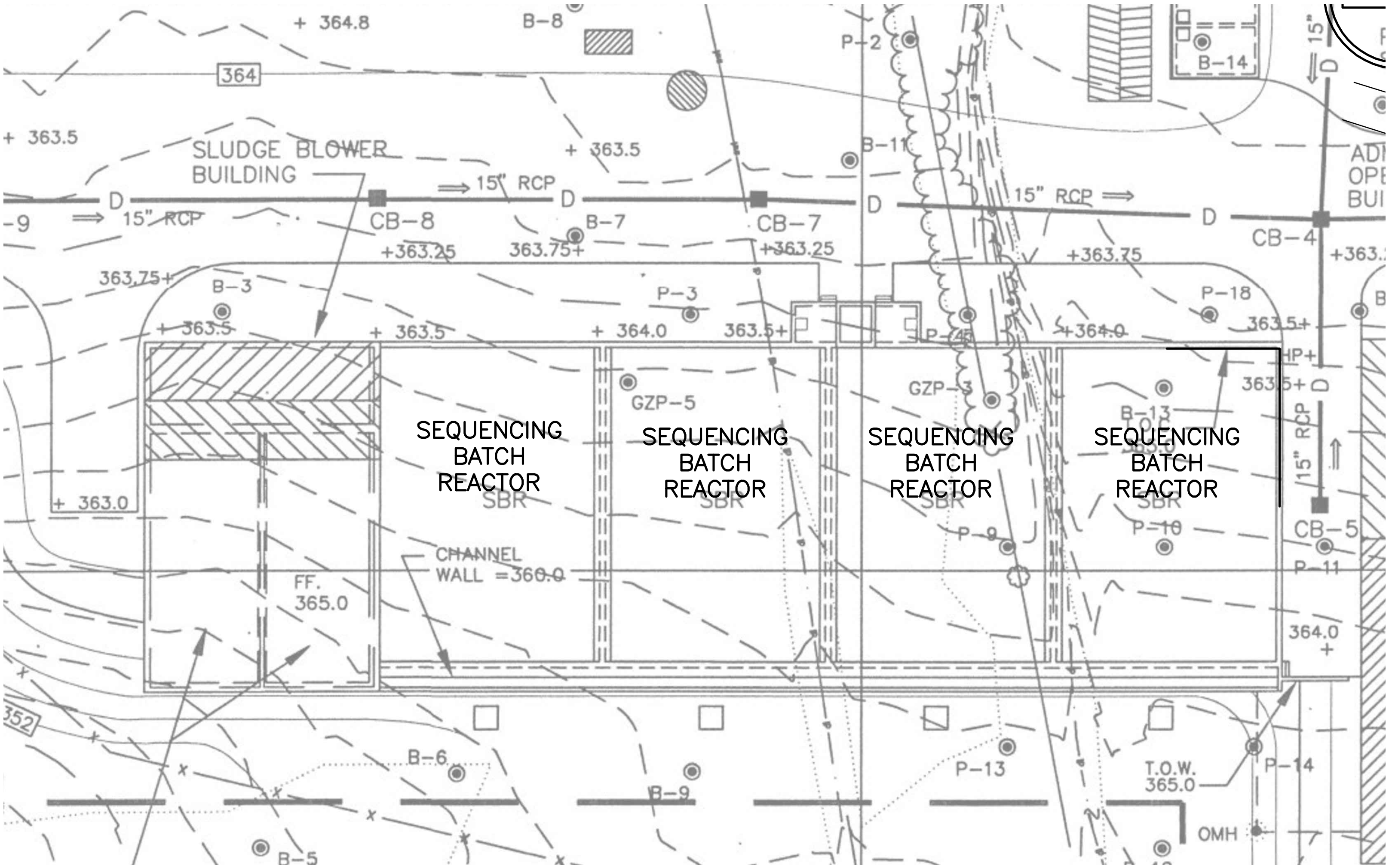
5.7.1 Description

Three technological options were considered for upgrade of the biological treatment process. The first option is to continue with the existing SBR process. Other options were identified with the intent of implementing within the footprint of the existing SBRs. Two viable options were identified that meet this criteria: Moving Bed Biological Reactor and Multi-Stage Anoxic/Oxic Treatment. Each option was analyzed with and without primary treatment upstream of the process. A summary of design criteria for each alternative is presented in Table No. 5-5.

Sequencing Batch Reactors – This alternative is to continue with the existing SBR process. The process biology is single activated sludge with nitrification, with an anoxic phase that acts as a phosphorous selector and provides denitrification. The existing four (4) SBR configuration meets existing and future requirements for treatment. Only three (3) SBRs would be required if primary treatment is provided. Schematic site plans for the improvements are presented in Figures No. 5-5 and 5-6.

Table No. 5-5
Design Parameters for Wastewater Treatment Options

	Option 1A SBR	Option 1B SBR W/ Prim. Clar.	Option 2A MBBR	Option 2B MBBR W/ Prim. Clar.	Option 3A AO	Option 3B AO W/ Prim. Clar.
	Full Operation					
Design Flow	2,100,000	2,800,000	2,200,000	3,100,000	2,100,000	2,100,000
Number of Basins in Service	4	3	2	2	3	2
Reactor Volume (gallons)	1,511,319	1,133,489	1,214,453	1,214,453	1,821,679	1,214,453
Hydraulic Retention Time (hours)	17	10	13	9	21	14
Solids Retention Time (days)	32	18	22	16	30	20
Mixed Liquor Suspended Solids (mg/l)	4,500	4,500	4,000	4,000	3,500	3,500
Solids Wasted (lbs/day)	4,519	6,025	4,734	6,670	4,519	4,519
	Largest Unit Out of Operation					
Design Flow	1,575,000	2,100,000	1,650,000	2,325,000	1,575,000	1,575,000
Number of Basins in Service	3	2	1	1	2	1
Reactor Volume (gallons)	1,33,489	755,660	607,226	607,226	1,214,453	607,226
Hydraulic Retention Time (hours)	17	9	9	6	19	9
Solids Retention Time (days)	13	9	6	6	10	6
Mixed Liquor Suspended Solids (mg/l)	4,500	4,500	4,000	4,000	3,500	3,500
Solids Wasted (lbs/day)	25,000	25,000	25,000	25,000	25,000	25,000



TATA & HOWARD

MIDDLEBURY WWTF
SEQUENCING BATCH REACTOR

PRELIMINARY ENGINEERING REPORT
SECONDARY/ADVANCED TREATMENT OPTIONS
MIDDLEBURY, VERMONT

Figure No.

5-5

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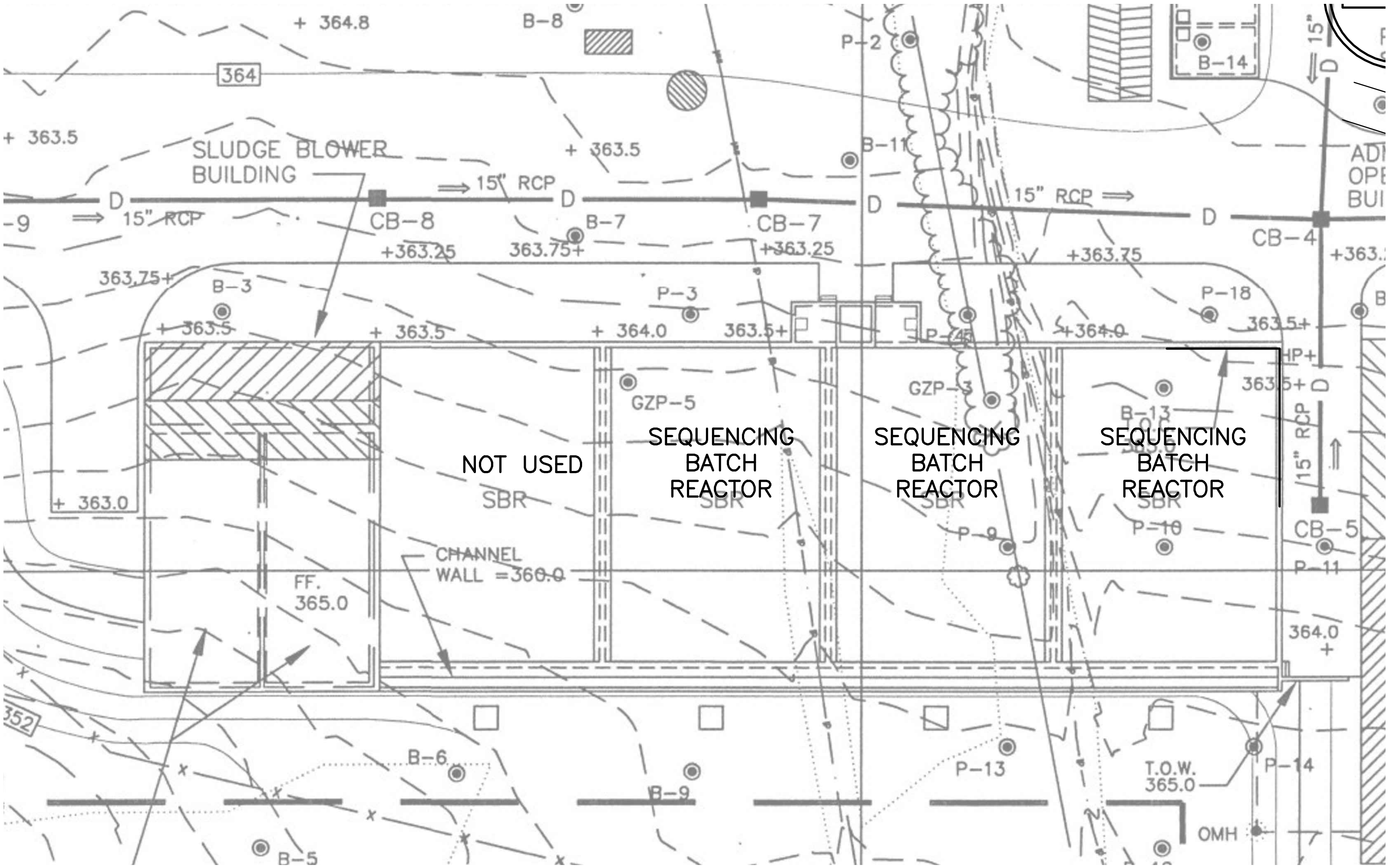

TATA & HOWARD

MIDDLEBURY WWTF
SEQUENCING BATCH REACTOR W/PRIMARIES

PRELIMINARY ENGINEERING REPORT
SECONDARY/ADVANCED TREATMENT OPTIONS
MIDDLEBURY, VERMONT

Figure No.

5-6

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Moving Bed Biological Reactor (MBBR) – This alternative uses the same biology as SBR, but each biological activity occurs within isolated tanks. A first stage anaerobic selector provides biological P removal and enhances other biological activities. The moving bed is the aerobic treatment section and employs the use of mixed suspended growth and attached growth. A synthetic media is contained within the biological reactor to support the attached growth. This allows for more treatment capacity for unit reactor volume. The anoxic treatment promotes biological phosphorous removal and low energy BOD removal. Nitrification occurs in a segregated longer retention time aerated (oxic) reactor. Secondary clarifiers follow the biological process to remove suspended solids after biological treatment for sludge return and wasting. Schematic site plans for the improvements are presented in Figures No. 5-7 and 5-8.

Multi-Stage Anoxic/Oxic (A/O) Treatment – This alternative also employs the same biological processes as MBBR. The difference between MBBR and A/O is that there are no mixed growth reactors. The reactors are all suspended growth with this alternative. The exact configuration is to be determined during design. A 3-stage anaerobic/anoxic/oxic process has been assumed for the cost opinion. For example, the four stage Bardenpho process offers significant denitrification and better BPR within the same footprint, which would far exceed permit requirements for TN. The final selection A/O configuration should be considered in a pre-design phase. Schematic site plans for the improvements are presented in Figures No. 5-9 and 5-10.

5.7.2 Environmental Impacts

These improvements will be completed within the footprint of existing SBRs. There will be no environmental impacts of new construction for this work. A slight energy savings from the improvements will have positive environmental impacts off site.

5.7.3 Land Requirements

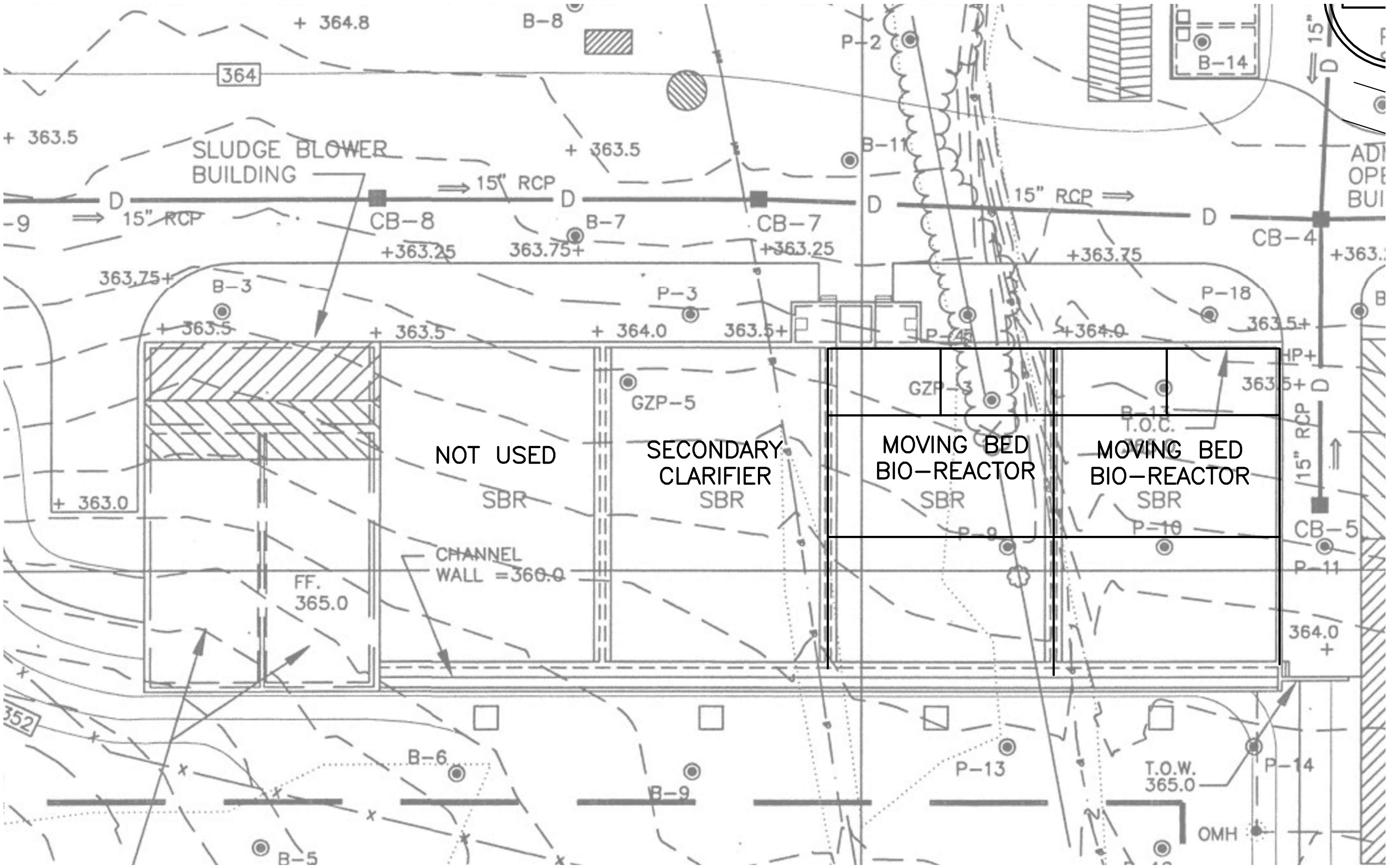
There are no land requirement for this project element.

5.7.4 Potential Construction Problems

There are no special construction problems identified for this work. There will be typical complications and difficulties associated with installation in existing structures, including associated demolition.

5.7.5 Sustainability Considerations

All the technologies under consideration are proven from a sustainability standpoint.



TATA & HOWARD	MIDDLEBURY WWTF MOVING BED BIOLOGICAL REACTOR (MBBR) PRELIMINARY ENGINEERING REPORT SECONDARY/ADVANCED TREATMENT OPTIONS MIDDLEBURY, VERMONT	Figure No. 5-7

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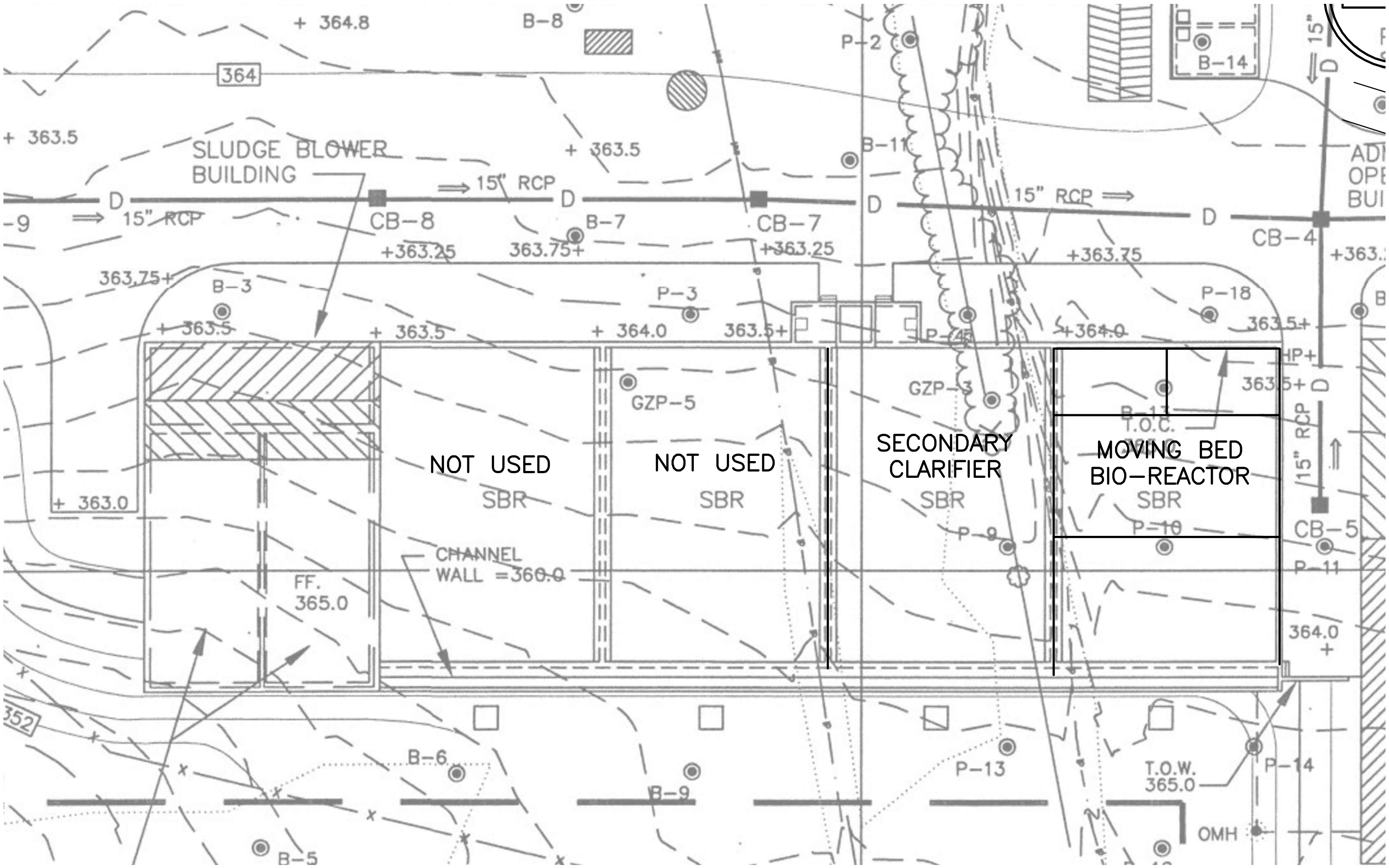

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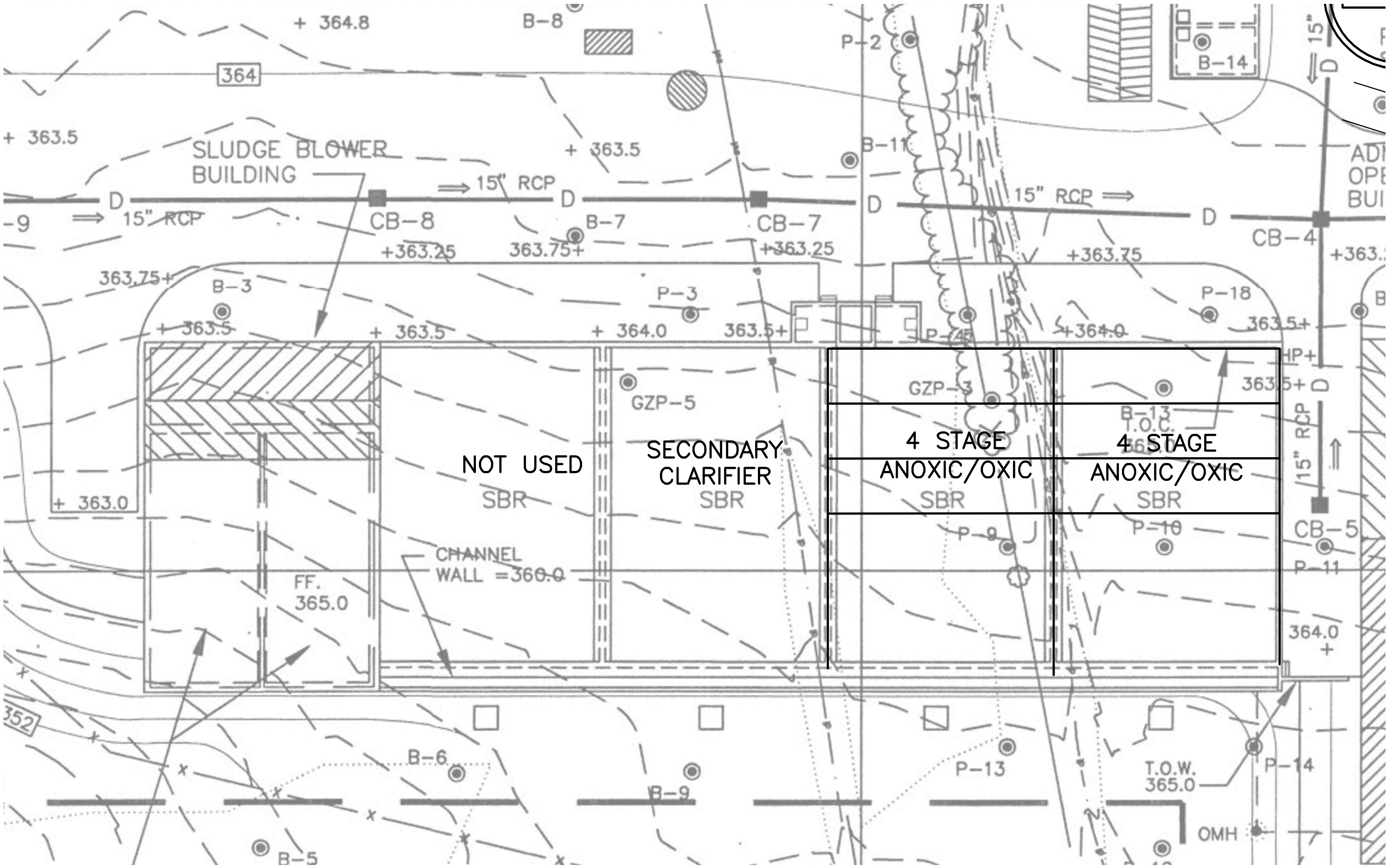
MIDDLEBURY WWTF
MOVING BED BIO-REACTOR W/PRIMARIES

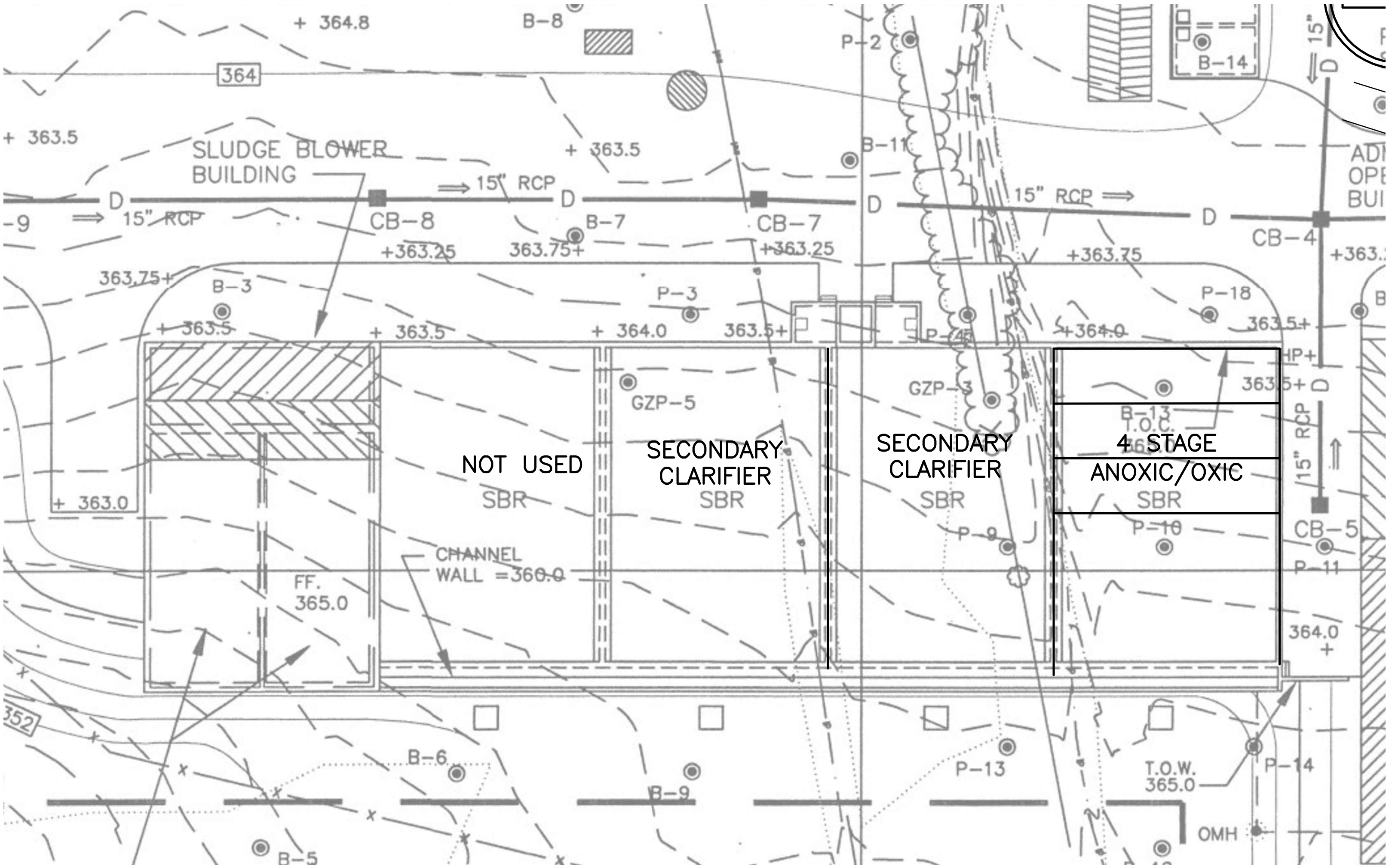
PRELIMINARY ENGINEERING REPORT
SECONDARY/ADVANCED TREATMENT OPTIONS
MIDDLEBURY, VERMONT

Figure No.

5-8



TATA & HOWARD	MIDDLEBURY WWTF 4-STAGE ANOXIC/OXIC PRELIMINARY ENGINEERING REPORT SECONDARY/ADVANCE TREATMETN OPTIONS MIDDLEBURY, VERMONT	Figure No. 5-9



TATA & HOWARD	MIDDLEBURY WWTF 4 STAGE ANOXIC/OXIC W/PRIMARY TREATMENT PRELIMINARY ENGINEERING REPORT SECONDARY/ADVANCED TREATMENT OPTIONS CITY/STATE/CLIENT	Figure No. 5-10

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5.7.6 Water and Energy Efficiency

These improvements will have no effect on water efficiency. The MBBR and A2O2 alternatives will result in reduced energy consumption,

5.7.7 Green Infrastructure

Implementation of green infrastructure is not applicable to this project element.

5.7.8 Operational Simplicity

All these unit processes are considered equal in terms of operational simplicity.

5.7.9 Opinions of Probable Cost and Life Cycle Cost Analysis

Table No. 5-6 presents the opinions of probable cost and life cycle analysis of each biological treatment option with and without primary clarifiers. Construction costs are presented for each alternative based on the calculations of probable cost provided in Appendix ___. Selected operating costs are provided for comparison purposes only and are based on manufacturers data and calculations made as part of the study. The energy credit for primary clarifiers assumes a 20% increase in energy production from anaerobic digestion with those options. The capital cost of rectangular primary clarifiers is assumed. The hydraulic capacity and number of tanks required for each alternative is unique because the study places various technologies in tanks of fixed size. Credits for both the value of excess reserve capacity and the availability of unused tanks has been added under an alternative calculation called "Life Cycle Cost with Credits".

Table No. 5-6
Evaluation of Wastewater Treatment Options

		Option 1A SBR	Option 1B SBR W/ Prim. Clar.	Option 2A MBBR	Option 2B MBBR W/ Prim. Clar.	Option 3A AO	Option 3B AO W/ Prim. Clar.
Hydraulic Capacity (ADF)		2.1	2.8	2.2	3.1	2.1	2.1
Construction Cost		$1,310,500	$1,593,025	$3,052,550	$2,418,825	$2,307,550	$2,175,075
Other Project Costs	30%	$393,150	$477,908	$915,765	$725,648	$692,265	$652,523
Total Project Cost		$1,703,650	$2,070,933	$3,968,315	$3,144,473	$2,999,815	$2,827,598
Power		$199,850.00	$139,895.00	$133,899.50	$93,729.65	$133,899.50	$93,729.65
Chemicals		$14,000.00	$14,000.00	$14,000.00	$14,000.00	$14,000.00	$14,000.00
Labor		$60,000.00	$60,000.00	$60,000.00	$60,000.00	$60,000.00	$60,000.00
Equipment Replacement		$39,315.00	$47,790.75	$91,576.50	$72,564.75	$69,226.50	$65,252.25
Energy credit W/AD only			-$40,000.00		-$40,000.00		-$40,000.00
TOTAL		$313,165.00	$221,685.75	$299,476.00	$200,294.40	$277,126.00	$192,981.90
PW Factor @ i=	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%	2.8%
Planning period (years)		30	30	30	30	30	30
Present Worth of O&M		$6,299,946	$4,459,656	$6,024,564	$4,029,326	$5,574,949	$3,882,220
Life Cycle Cost		$8,003,596	$6,530,589	$9,992,879	$7,173,799	$8,574,764	$6,709,818
CREDITS (Volume $/gal of tank volume)							
Reserve Capacity*	$5.00	$0	$333,754	$76,725	$268,538	$76,725	$0
Tank Reserve	$2.50	$0	$191,813	$191,813	$191,813	$0	$191,813
Life Cycle Cost w/Credits		$8,003,596	$6,005,022	$9,724,342	$6,713,449	$8,498,039	$6,518,005

5.8 Disinfection

5.8.1 Description

The existing ultra-violet (UV) disinfection system was determined to be inadequately sized for the existing conditions. The wastewater has had excessively low UV transmittance during 2019 that has resulted in effluent coliform violations. The existing facility has limited mechanical redundancy but lacks full hydraulic redundancy. The existing space is too small to accommodate a new system in parallel to the existing. Rehabilitation of the existing facility would require temporary disinfection because of the lack of true redundancy. Both conditions require a new facility, which can be located adjacent to the existing. The existing can be repurposed with improved effluent flow metering or made available in parallel to the new facility for true redundancy. The alternatives evaluated are open channel UV disinfection, enclosed UV disinfection and chlorine disinfection.

Open Channel UV – This alternative is for continued operation of the existing technology. In order to provided redundancy, two channels with bypass capabilities will be provided. Each channel will be equipped with full capacity disinfection system. The existing channel could be retrofitted as one of these channels. It is assumed that the best arrangement is to locate both channels in a new building, as there is little cost in creating a second channel in the new space and all the equipment will be in one location.

Enclosed UV – Medium pressure ultraviolet disinfection is an effective method of wastewater disinfection. The units consist of fabricated steel or stainless steel housings containing UV bulbs, with external wiring and controls. The units are connected in-line with the effluent piping. It is proposed that the units would be in a new slab on grade building with the piping and UV units installed above the floor. This building would also include space for power and control systems and equipment required for this unit process.

Chlorination – This alternative is for chemical disinfection of the wastewater with chlorine. The wastewater is held in contact with free chlorine at an initial dosage of approximately 5-10 mg/l for a minimum of 30 minutes to kill bacteria to permit limits. The chlorine contact chambers must meet specified length to width ratios to ensure plug flow and avoid short-circuiting. The use of liquid chlorine in the form of sodium hypochlorite solution has been assumed for ease of operation. The NPDES Permit requires that the WWTF effluent have a chlorine residual of not more than 1.0 mg/l. Chemical dechlorination with dry sodium bisulfite will be provided to reduce the chlorine residual within permit limits. The cost opinion assumes construction of a new chlorine contact tank and chemical feed building structures for this purpose. Although, conversion and use of a former SBR tank is viable for use as a chlorine contact tank, and the UV building may also provide a space for chemical feed and storage systems.

5.8.2 Environmental Impacts

Each option will be built within the existing WWTF site and therefore, has minimal environmental impacts.

5.8.3 Land Requirements

There are minimal land requirements for this project element. The land requirements for each alternative are approximately 5,000 sf for either UV alternative, 10,000 sf for a chlorine contact tank, and none if the chlorine contact is placed with an unused SBR.

5.8.4 Potential Construction Problems

There are no special construction problems identified for this work. There will be typical complications and difficulties associated with installation in an existing building and associated demolition for the mechanical primary treatment alternative.

5.8.5 Sustainability Considerations

All the technologies under consideration are proven from a sustainability standpoint.

5.8.6 Water and Energy Efficiency

The chlorination results in energy savings in comparison to the two UV disinfection options.

5.8.7 Green Infrastructure

Implementation of green infrastructure is not applicable to this project element.

5.8.8 Operational Simplicity

All these unit processes are considered equal in terms of operational simplicity.

5.8.9 Opinions of Probable Cost and Life Cycle Cost Analysis

Table No. 5-7
Evaluation of Disinfection Options

	Open Channel UV	Enclosed UV	Chlorination & Dechlorination
Construction cost	$825,600	$606,338	$1,038,900
Contingency (15%)	$247,680	$181,901	$311,670
Other Project Costs (30%)	$321,984	$236,472	$405,171
Total Project Cost	$1,395,264	$1,024,710	$1,755,741
Power	$50,000	$50,000	$2,000
Chemicals	$2,000	$2,000	$30,000
UV Lamp Replacement	$14,400	$14,400	$0
Labor	$6,240	$6,240	$6,240
TOTAL	$72,640	$72,640	$38,240
PW Factor @ 2.8%, 30 yrs	20.12	20.12	20.12
PW	$1,461,300	$1,461,300	$769,275

5.9 Sludge Dewatering

5.9.1 Description

Three technological options were considered for sludge dewatering. The first option is to continue with the existing belt filter press process. Two other options were identified: centrifuge, rotary fan press and screw press. Each alternative has been sized to match the existing combined processing capacity of the two existing belt filter presses of 1,200 dry-lbs/hr. At this processing rate, the dewatering facilities will operate 26 hours per week dewatering raw sludge only and 52 hours per week with SSO added. With ATAD or anaerobic digestion, the run times are 15 hours per week dewatering raw sludge only and 30 hours per week with SSO added. It has been assumed that all technologies produce 20% solids content for this analysis. The BFPs presently put out 14-16%. We expect this could be optimized to 16-18% and this would be typical of other technologies with the same sludge. It is expected that anaerobically digested sludge will dewater to the range of 22-24%, with any technology optimized with polymer and loading rate. It is noted that for screw presses, centrifuges, and fan presses, many facilities run the equipment continuously without supervision. This results in reduced equipment sizing and capital costs. This is considered an item for further consideration by the Town and will be dependent upon the compatibility of that operation with the selected Class A treatment process. It may provide an opportunity to provide redundancy at similar cost.

Belt filter press – Belt filter presses are a proven technology that dewater sludge by applied pressure to a layer of sludge that is placed on one belt in an open section and then pressed between a second belt. The belts move at slow speed and pressure is created by running the belts over a series of rollers. Dewatered sludge falls off the belts as they separate after the pressurization process. Belt filter presses use a large amount of wash water. Two variations of the alternative are considered: one being rehabilitation of the existing equipment and the other full replacement with new equipment. The equipment will be installed in the current dewatering room. Minimal demolition is anticipated only for the alternative of installing the complete new equipment.

Centrifuge – A sludge dewatering centrifuge uses a fast rotation of a “cylindrical bowl” to separate wastewater liquid from solids. The wastewater centrifuge dewatering process removes a higher amount of water than most methods. Centrifuges have a small footprint, high energy cost and use little wash water in comparison to other options. The centrifuge equipment will easily fit within the existing dewatering room. The equipment is completely enclosed in a sealed housing and makes for a better work environment than for the BFP alternatives.

Rotary fan press – A rotary fan press consists of fan shaped blade that rotates at low speed between discs. Water is expelled through the discs and a dewatered cake remains. These are simple low rotational devices with a small footprint. Rotary fan presses have a modest energy requirement and use very little wash water. The fan press equipment will easily fit within the existing dewatering room. The equipment is also completely enclosed in a sealed housing and makes for a better work environment than for the BFP alternatives.

Screw press – A screw press is the lowest speed dewatering technology considered. A slow moving screw moves sludge along a perforated screen as water is pressed and expelled from the unit. The sludge cake is discharged from the top of the machines. These are similar to rotary fan presses in terms of footprint, energy use, and wash water needs. The screw press will easily fit within the existing dewatering room. The equipment is completely enclosed in a ventilated housing and makes for a better work environment than the BFP alternatives.

5.9.2 Environmental Impacts

These improvements will be completed within the footprint of existing buildings and infrastructure. There will be no environmental impacts of new construction for this work. An increase in energy consumption for the centrifuge, rotary fan press, and screw press alternatives is offset by a reduction in water consumption.

5.9.3 Land Requirements

There are no land requirements for this project element.

5.9.4 Potential Construction Problems

There are no special construction problems identified for this work. There will be typical complications and difficulties associated with installation in an existing building and associated demolition.

5.9.5 Sustainability Considerations

All the technologies under consideration are proven from a sustainability standpoint.

5.9.6 Water and Energy Efficiency

An increase in energy consumption for the centrifuge, rotary fan press and screw press alternatives is offset by a reduction in water consumption.

5.9.7 Green Infrastructure

Implementation of green infrastructure is not applicable to this project element.

5.9.8 Operational Simplicity

All these alternatives are considered equal in terms of operational simplicity

5.9.9 Opinions of Probable Cost and Life Cycle Cost Analysis

Table No. 5-8
Sludge Dewatering Options

	Centrifuge	Belt Filter Press-Rehab	Belt Filter Press-New	Fan Press	Screw Press
Demolition	$150,000.00	-	$15,000.00	$150,000.00	$150,000.00
Equipment	$556,250.00	$383,000.00	$915,625.00	$482,500.00	$596,250.00
Odor Control	$40,000.00	$40,000.00	$40,000.00	$40,000.00	$40,000.00
Miscellaneous	-	-	-	-	-
	$746,250.00	$423,000.00	$970,625.00	$672,500.00	$786,250.00
Ann. Bond Payment	$74,625.00	$42,300.00	$97,062.50	$67,250.00	$78,625.00
Labor	$12,480.00	$24,960.00	$24,960.00	$12,480.00	$12,480.00
Water	$5,000.00	$45,000.00	$45,000.00	$5,000.00	$5,000.00
Chemicals	$20,000.00	$20,000.00	$20,000.00	$20,000.00	$20,000.00
Electric	$30,000.00	$12,500.00	$12,500.00	$25,000.00	$25,000.00
	$67,480.00	$102,460.00	$102,460.00	$62,480.00	$62,480.00
PWF	20.12	20.12	20.12	20.12	20.12
PW	$1,357,697.60	$2,061,495.20	$2,061,495.20	$1,257,097.60	$1,257,097.60
Life Cycle Cost	$2,103,947.60	$2,484,495.20	$3,032,120.20	$1,929,597.60	$2,043,347.60

Annual Sludge Production*: 20,075,000.00 gallons

5.10 Residuals Management

5.10.1 Description

The driving force for upgrade of the existing lime pasteurization residuals management facilities is the corrosion and wear of the existing equipment comprising the lime stabilization system. The existing system needs replacement within the next five years. The Town desires to consider only management options that will produce EPA, Part 503 Class A Biosolids. The Town has also requested that anaerobic digestion with and without source separated organics (SSO) (food waste) be considered. At total of five technological alternatives have been evaluated: lime pasteurization, heat drying (pasteurization), aerobic digestion, anaerobic digestion and composting. Anaerobic digestion is considered with two variations: thermophilic and thermal hydrolysis first stages. Composting is similarly considered with two variations: in-vessel agitated bin and windrow. Each Alternative and variation is considered with and without source separated organics.

WWTF solids production rates presented in Table No. 3-10 are 4,400 lbs/day (2.2 dtpd) for the design year 2038. An equal amount of incoming SSO as measured by dry solids content has been assumed for the alternatives processing SSO for a total input of 8,800 lbs/day (4.4 dtpd).

<u>Lime Pasteurization</u> – This is technology currently in place at the WWTF. The past approximately 20 years of operation have been successful overall. The facility is producing Class A Biosolids and paying a single farmer to take it all. The EPA 503 Rule requires that certain standards be met for biosolids to be Class A. The technology meets this through the combined effects of high pH through lime addition and heat applied to dewatered sludge. The same technological approach is considered for the upgrade as for the existing system. The current version of the systems uses stainless steel vessels as an improvement over the carbon steel used in 2000. The sludge and lime are mixed in a heated thermal blender, then held for additional time in a pasteurization unit at high pH and temperature and then discharged to a storage area. The finished product is a brownish moist biosolids.

<u>Heat Drying (Pasteurization)</u> – Heat drying combines thermal heating and evaporation to produce a dry fertilizer product. Since the end product is more than 90% solids, the process discharges a much smaller mass than other alternatives. Dewatered biosolids are discharged to the dryer to achieve Class A quality through high temperature pasteurization for a specified time. The end product has a higher value and broader market appeal than other options. Based on a preliminary review, the new equipment will fit within the footprint of the existing equipment.

<u>Auto-Thermal Aerobic Digestion</u> – Autothermal aerobic digestion (ATAD) can be described as liquid composting for biosolids. Liquid sludge is contained in a large covered tank and held and aerated to reach sufficient temperature-time combination to achieve Class A quality. The liquid sludge is then dewatered and may be distributed without restriction. The end product is a dark moist biosolids. A schematic site plan for the related improvements is presented in Figure No. 5-11. The equipment for this alternative has been sized for 24 hour a day operation.

<u>Anaerobic Digestion</u> – Anaerobic digestion uses higher temperatures than ATAD to create an environment for liquid to decompose anaerobically. The digestion process produces biogas, which is high in methane and has a significant energy value. After digestion, the liquid sludge is

dewatered and may be distributed without restriction. The end product is a dark moist biosolids similar to ATAD. Schematic site plans for the improvements are presented in Figures No. 5-12 and 5-13.

Composting – Composting is natural decomposition that occurs when sludge is combined with a carbon-rich amendment like sawdust or chips to produce a stabilized soil-like end product. The carbon in the amendment and nitrogen in the sludge provide ideal fuel for aerobic decomposition of the volatile solids in the dewatered sludge. This biological activity creates a high temperature, when maintained for a minimum time, produces a Class A product. This process produces a greater volume of end product than volume of sludge received because of the addition of the amendment. The end product has more value than ATAD or anaerobic digestion, since it is a user friendly soil amendment. Two technical variations are considered: agitated bin and windrow composting. Agitated bin composting is performed in long bins in an enclosed building with a rail mounted automated pile turner, aeration, and odor control. Windrow composting is performed on a covered slab with wheeled turner with no supplemental aeration or odor control. Schematic site plans for the improvements are presented in Figures No. 5-14 and 5-14.

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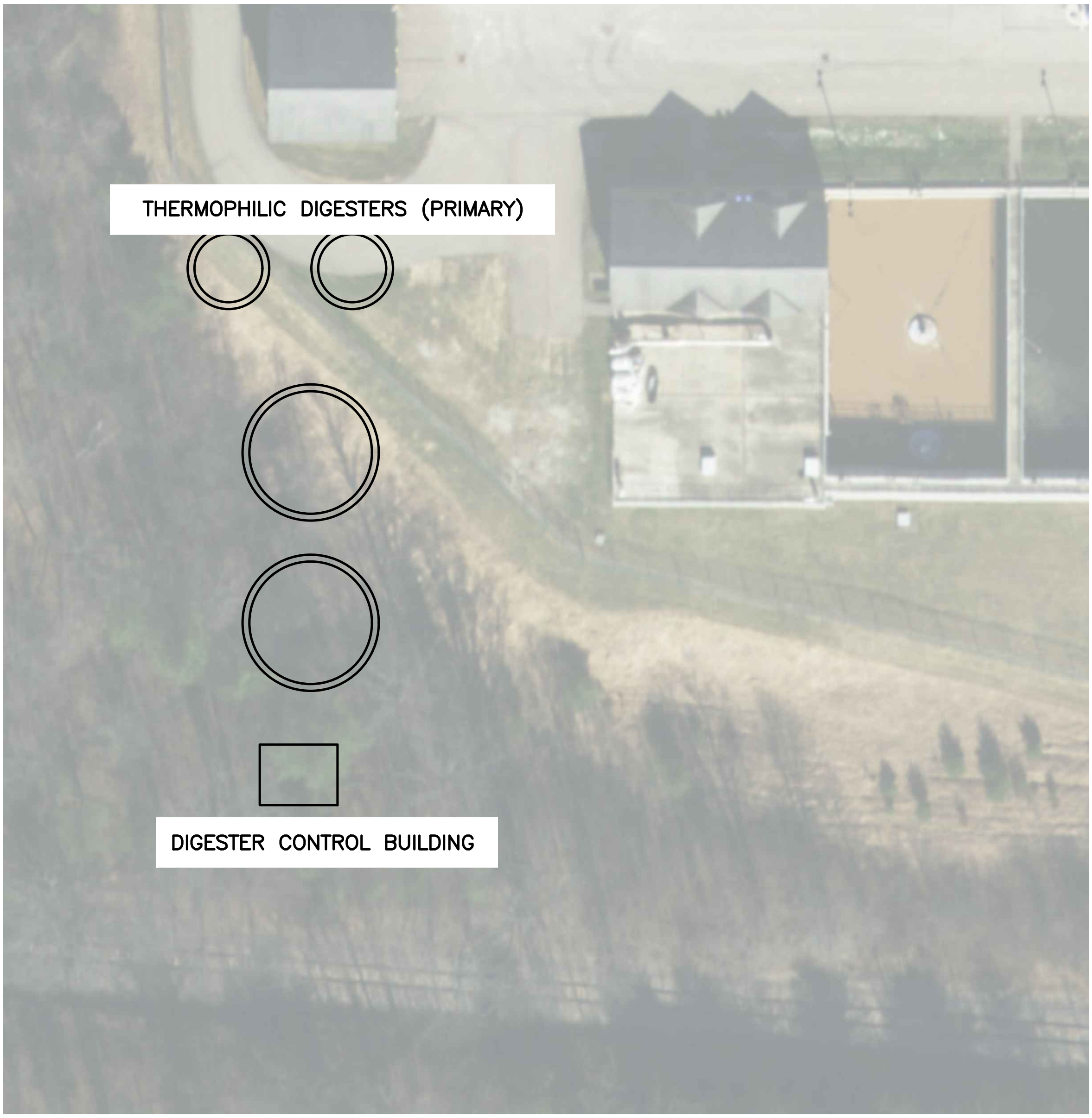
TATA & HOWARD

MIDDLEBURY WWTF
ATAD

PRELIMINARY ENGINEERING REPORT
RESIDUALS MANAGEMENT OPTIONS
MIDDLEBURY, VERMONT

Figure No.

5-11



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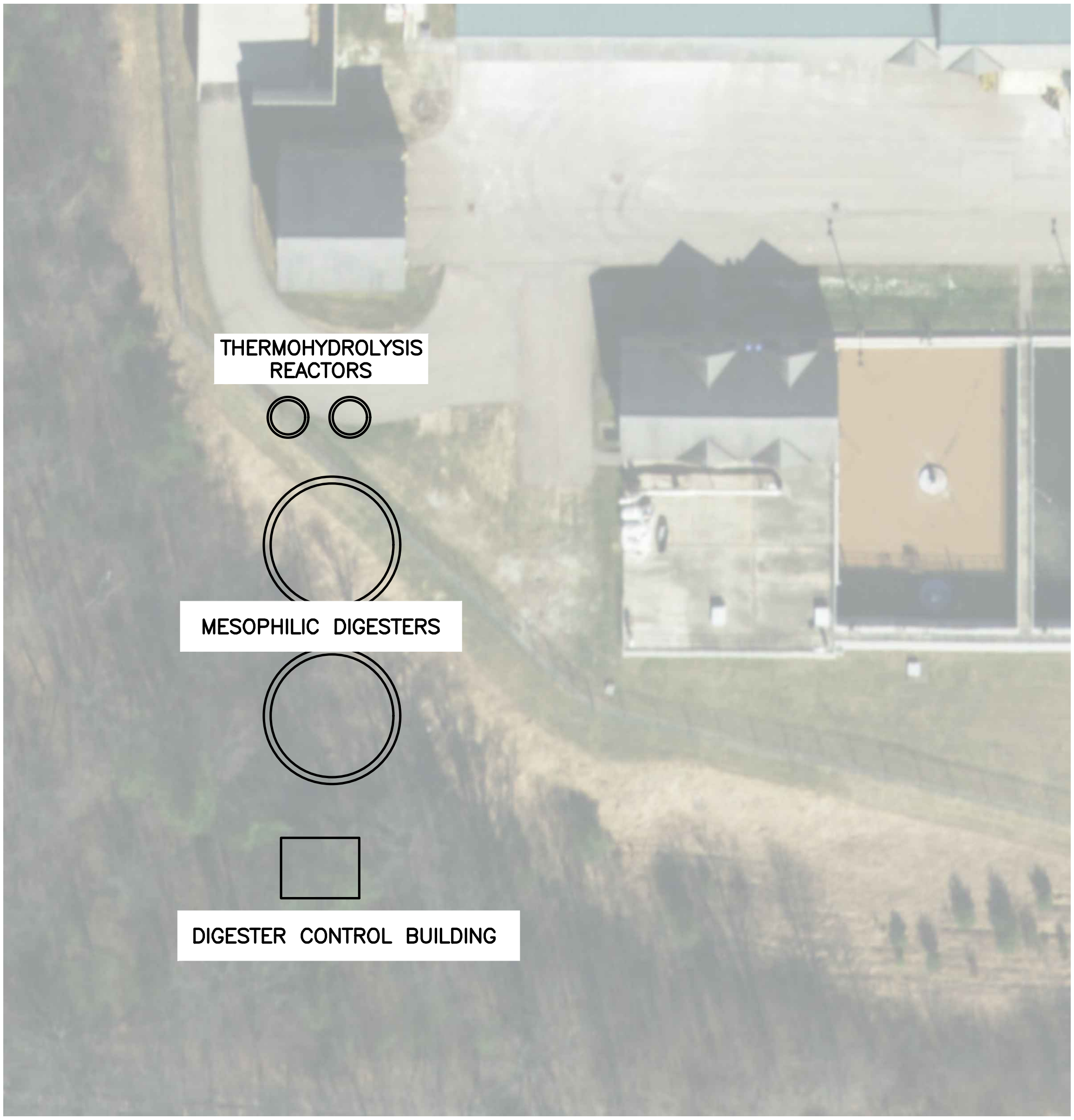
TATA & HOWARD

CONCEPTUAL SITE PLAN
ANAEROBIC DIGESTION

PRELIMINARY ENGINEERING REPORT
RESIDUALS MANAGEMENT REPORT
MIDDLEBURY, VERMONT

Figure No.

5-12



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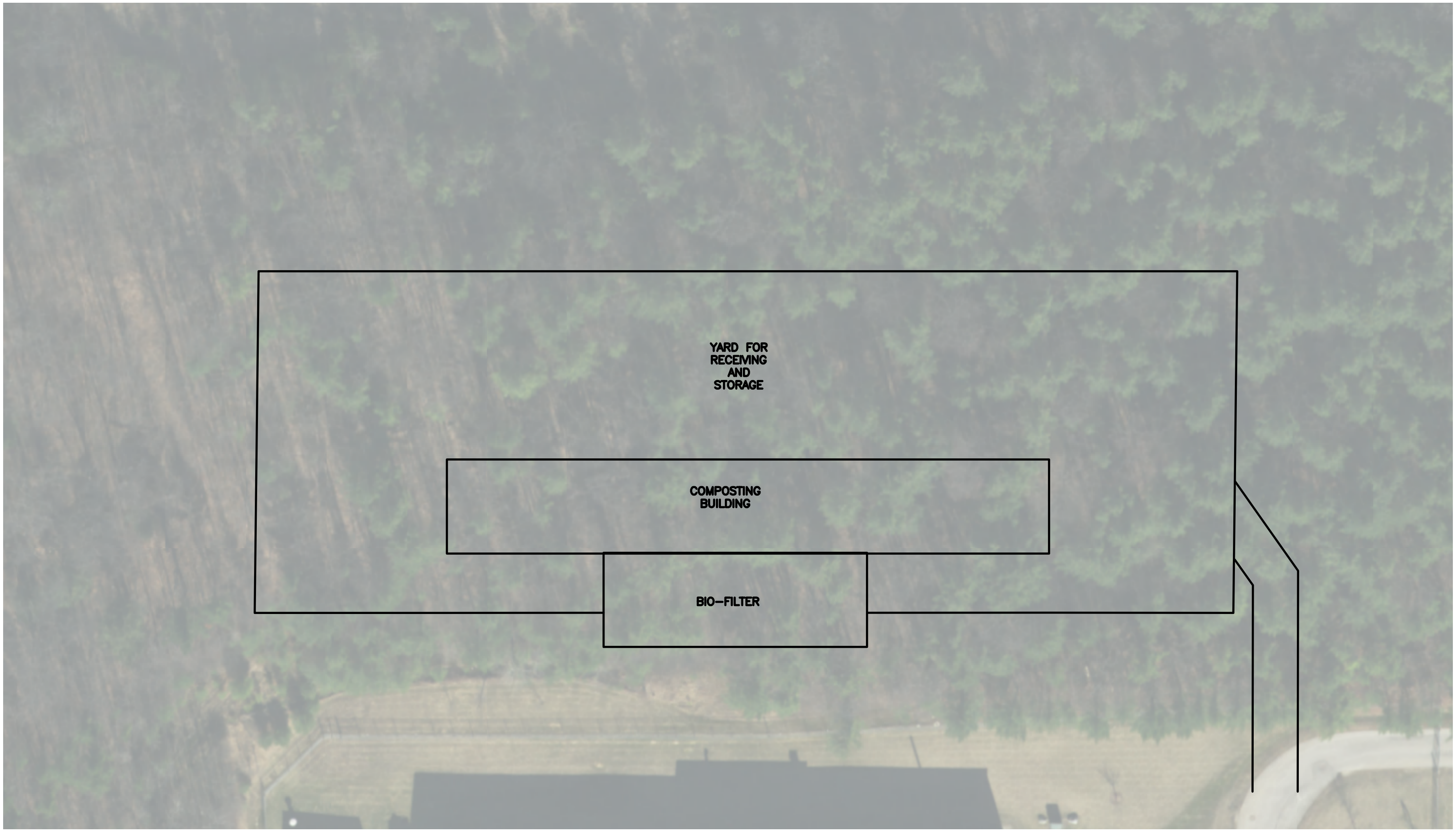
TATA & HOWARD

CONCEPTUAL SITE PLAN
ANAEROBIC DIGESTION WITH HYDROLYSIS

PRELIMINARY ENGINEERING REPORT
RESIDUALS MANAGEMENT REPORT
MIDDLEBURY, VERMONT

Figure No.

5-13



TATA & HOWARD

MIDDLEBURY WWTF
AGITATED BIN COMPOSTING

PRELIMINARY ENGINEERING REPORT
RESIDUALS MANAGEMENT OPTIONS
MIDDLEBURY, VERMONT

Figure No.

5-14

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TATA & HOWARD

MIDDLEBURY WWTF
WINDROW COMPOSTING

PRELIMINARY ENGINEERING REPORT
RESIDUALS MANAGEMENT OPTIONS
MIDDLEBURY, VERMONT

Figure No.

5-15

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5.10.2 Environmental Impacts

The lime pasteurization and drying alternatives have had minimal environmental impacts because they will be built within the existing WWTF building. The remaining alternatives have varying amounts of impact based on each alternative's land requirements. Composting has greater air emissions among the alternatives. Anaerobic digestion produces energy sufficient to power the entire WWTF.

5.10.3 Land Requirements

There are variable land requirements for this project element. There are no land requirements for lime pasteurization and drying. Approximately 15,000 sf for ATAD and anaerobic digestion. The land requirements for agitated bin composting are approximately 2 acres and for windrow composting, 4 acres.

5.10.4 Potential Construction Problems

There are no special construction problems identified for this work. There will be typical complications and difficulties associated with development of new land for all alternatives except lime pasteurization and drying.

5.10.5 Sustainability Considerations

All the technologies under consideration are proven from a sustainability standpoint.

5.10.6 Water and Energy Efficiency

These alternatives will have no effect on water efficiency, but will improve energy efficiency if the ATAD, anaerobic digestion, or composting alternative is selected.

5.10.7 Green Infrastructure

Implementation of green infrastructure is not applicable to lime pasteurization, drying ATAD or anaerobic digestion. Green infrastructure is an inherent part of either composting option in the form of Best Management Practices for stormwater runoff from the composting facility.

5.10.8 Operational Simplicity

All these unit processes are considered equal in terms of operational simplicity.

5.10.9 Opinions of Probable Cost and Life Cycle Cost Analysis

Tables No. 5-9 and 5-10 present the probable costs and life cycle analysis of each residuals management option with and without Source separated Organics (SSO). Construction costs are presented for each alternative based on the calculations of probable cost provided in Appendix ___. Selected operating costs are provided for comparison purposes only and are based on manufacturers data and calculations made as part of the study. Process and dewatering side stream and treatment costs are calculated from the existing operating cost for treatment of BOD, TP, and nitrification and adjusted for each process. Credits are considered for energy production and reduced dewatering costs for anaerobic digestion, which produce biogas for energy production and a 40% reduction in net solids output.

Table No. 5-9
Evaluation of Residuals Management Options – Municipal Wastewater Only

	Option 1A HT lime	Option 2 Dryer	Option 3 ATAD	Option 4A AD	Option 4B AD Plus	Option 5A Compost Agitated Bin	Option 5B Compost Windrow
Probable Construction Cost	$2,240,000	$3,259,375	$4,022,000	$6,073,813	$6,705,813	$5,349,900	$8,577,100
Contingency (15%)	$336,000	$488,906	$603,300	$911,072	$1,005,872	$802,485	$1,286,565
Other Project Cost (30%)	$772,800	$1,124,484	$1,387,590	$2,095,465	$2,313,505	$1,845,716	$2,959,100
Total Project Cost	$3,348,800	$4,872,765	$6,012,890	$9,080,350	$10,025,190	$7,998,101	$12,822,765
Power	$50,000	$90,000	$92,199	$50,000	$50,000	$90,000	$2,000
Chemicals	$150,000	-	-	$2,000	$2,000	$80,000	$80,000
Fuel	$211,554	$423,108	-	$105,778	$105,777	$10,000	$20,000
Process Sidestream Treatment Cost	-	$36,000	$36,000	$36,000	$36,000	-	-
Sidestream Treatment Cost	$18,000	$18,000	$18,000	$13,680	$13,680	$18,000	$18,000
Dewatering Credit	-	-	($37,488)	($37,488)	($37,488)	-	-
Labor	$65,520	$65,520	$65,520	$65,520	$65,520	$98,280	$131,040
Final Disposal Cost	$88,000	$21,000	$42,000	$42,000	$42,000	$31,000	$31,000
Energy Production	-	-	-	($200,000)	($200,000)	-	-
TOTAL	$583,074	$653,628	$216,231	$77,490	$77,489	$327,280	$282,040
PW Factor @ 2.8%, 30 yrs	20.12	20.12	20.12	20.12	20.12	20.12	20.12
PW of O&M	$11,729,710	$13,149,046	$4,349,923	$1,558,858	$1,558,848	$6,583,898	$5,673,804
Life Cycle Cost	$15,078,510	$18,021,812	$10,362,813	$10,639,207	$11,584,037	$14,581,999	$18,496,569

Table No. 5-10
Evaluation of Residuals Management Options – Municipal Biosolids plus SSO

	Option 1A HT lime	Option 2 Dryer	Option 3 ATAD	Option 4A AD	Option 4B AD Plus	Option 5A Compost Agitated Bin	Option 5B Compost Windrow
Probable Construction Cost	$1,600,000	$4,626,563	$4,834,500	$8,360,719	$9,677,469	$10,073,800	$16,117,100
Contingency (15%)	$240,000	$693,984	$725,175	$1,254,108	$1,451,620	$1,511,070	$2,417,565
Other Project Cost (30%)	$552,000	$1,596,164	$1,667,903	$2,884,448	$3,338,727	$3,475,461	$5,560,400
Total Project Cost	$2,392,000	$6,916,711	$7,227,578	$12,499,275	$14,467,816	$15,060,331	$24,095,065
Power	$50,000	$90,000	$92,199	$50,000	$50,000	$90,000	$2,000
Chemicals	$150,000	-	-	$2,000	$2,000	$80,000	$80,000
Fuel	$423,108	$846,216	-	$211,555	$211,555	$10,000	$20,000
Process Sidestream Treatment Cost	-	$36,000	$72,000	$72,000	$72,000	-	-
Sidestream Treatment Cost	$18,000	$18,000	$18,000	$13,680	$13,680	$18,000	$18,000
Dewatering Credit	-	-	($74,976)	($74,976)	($74,976)	-	-
Labor	$65,520	$65,520	$65,520	$65,520	$65,520	$98,280	$131,040
Final Deposit Cost	$174,516	$41,063	$69,806	$69,806	$69,806	$51,328	$51,328
Energy Production	-	-	-	(400,000)	(400,000)	-	-
Increased Revenue	($1,825,000)	($1,825,000)	($1,825,000)	($1,825,000)	($1,825,000)	($1,825,000)	($1,825,000)
TOTAL	($943,856)	($728,201)	($1,582,451)	($1,815,415)	($1,815,416)	($1,477,392)	($1,522,632)
PW Factor @ 2.8%, 30 yrs	20.12	20.12	20.12	20.12	20.12	20.12	20.12
PW of O&M	($18,987,576)	($14,649,243)	($31,834,190)	($36,520,741)	($36,520,751)	($29,720,719)	($30,630,813)
Life Cycle Cost	($16,595,576)	($7,732,532)	($24,606,612)	($24,021,466)	($22,052,935)	($14,660,388)	($6,535,748)

5.11 Miscellaneous Improvements

5.11.1 Description

a. Plant water – The WWTF uses approximately $50,000 per year (11,500 gpd) of Town water, representing approximately 10% of the metered water production for the Town. More than 90% of water within the WWTF is used as wash water for belt filter presses. If continued use of the BFPs is proposed, a plant water system that uses recycled effluent is recommended.

b. Motor Control Center – The MCC components are largely obsolete. Upgrade of the MCC with contemporary components is recommended.

c. Effluent channel – The effluent channel is a recurring maintenance problem. Widening the channel and converting to a pipe gallery with closed effluent piping is recommended.

d. Green Infrastructure – The WWTF is exempt from regulations requiring treatment of stormwater. It is recommended that an allowance be made in the construction budget to allow for construction of stormwater Best Management Practices that will improve the water quality of stormwater discharges from the WWTF.

e. Upgrade of Septage Receiving – Replace the existing septage receiving unit with a new unit with grit removal included. Providing a roof and spill containment for the receiving area is included in this item.

5.11.2 Environmental Impacts

The improvements will have minimal environmental impacts because they will be built within the existing WWTF. The stormwater improvements identified as Green Infrastructure will have positive impact on the environment.

5.11.3 Land Requirements

There are no land requirements for these alternatives, except for Green Infrastructure. It is anticipated that the Green Infrastructure improvements will be made within the WWTF site.

5.11.4 Potential Construction Problems

There are no special construction problems identified for this work. There will be typical complications and difficulties associated with rehabilitation of an existing WWTF.

5.11.5 Sustainability Considerations

All the technologies under consideration are proven from a sustainability standpoint.

5.11.6 Water and Energy Efficiency

Most of these alternatives will have no effect on water efficiency, except for the plant water system. The plant water system will implement effluent recycling and improve water efficiency.

5.11.7 Green Infrastructure

The only alternative with a green infrastructure consideration is the allowance to make stormwater improvements on site.

5.11.8 Operational Simplicity

All these unit processes are considered equal in terms of operational simplicity.

5.11.9 Opinions of Probable Cost and Life Cycle Cost Analysis

Table No. 5-10
Miscellaneous Improvements Estimated Costs

	Option 1A - SBR
Plant Water System	$500,000
Motor Control Center Upgrade	$500,000
Effluent Channel	$620,000
Green Infrastructure	$500,000
Septage Receiving Upgrade	$630,000
Exterior Building Repairs	$500,000

SECTION 6 – SELECTION OF ALTERNATIVE

6.1 Life Cycle Cost Analysis

6.2 Non-Monetary Factors

SECTION 7 – PROPOSED PROJECT (RECOMMENDED ALTERNATIVE)

7.1 Preliminary Project Design

7.1.1 Wastewater/Reuse

7.1.2 Solid Waste

7.2 Project Schedule

7.3 Permit Requirements

7.4 Sustainability Considerations

7.4.1 Water and Energy Efficiency

7.4.2 Green Infrastructure

7.4.3 Operational Simplicity

7.5 Total Project Cost Estimate (Engineer's Opinion of Probable Cost)

7.6 Annual Operating Budget

7.6.1 Income

7.6.2 Annual O&M Costs

7.6.3 Debt Repayments

7.6.4 Reserves

7.6.5 User Cost

SECTION 8 – CONCLUSIONS AND RECOMMENDATIONS