APPENDIX E

WASTEWATER TREATMENT FACILITY PLAN

Wastewater Treatment Facility Plan

for

WATERTOWN

M·I·N·N·E·S·O·T·A

BMI Project No. C12.36530

February 2006

Prepared by:

BOLTON & MENK, INC.

Consulting Engineers & Surveyors

WASTEWATER TREATMENT FACILITY PLAN

for the

CITY OF WATERTOWN, MINNESOTA

February 2006

BMI Project No. C12.36530

I hereby certify that this plan, specification or report was prepared by me or under my direct supervision, and that I am a duly Licensed Professional Engineer under the laws of the State of Minnesota.

Signature:____________________ Typed or Printed Name: Lana Tullis

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BOLTON & MENK, INC.
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1.0 INTRODUCTION

1.1 Purpose

This report is intended to provide the City of Watertown, Minnesota with recommended improvements for the municipal wastewater treatment facility. The plan evaluates the existing wastewater treatment facility and alternatives to meet the projected increase in demand. The plan includes the following:

1. Evaluation of the population, flow and loading projections.
2. Evaluation of the existing facilities to meet the current and future demands.
3. Recommendation for the most cost effective plan to meet community needs.
4. Presentation of a preliminary design for the selected plan.

The treatment facility was upgraded from a stabilization pond system to an extended aeration facility with filtration and disinfection in 1993. The upgrade was designed for a population of approximately 4,026 in the year 2010. Recent and planned development indicates the service area will approach the design population by 2007.

1.2 Report Organization

To address the major areas evaluated, the report is organized into the following sections:

Section 1: Introduction

Section 2: Design Considerations and Parameters

Section 3: Evaluation of Existing Wastewater Treatment Facilities

Section 4: Wastewater Treatment Facility Improvements

Section 5: Construction Cost Estimate

Section 6: Recommendations and Implementation

2.0 DESIGN CONSIDERATIONS AND PARAMETERS

2.1 Planning Period

Wastewater treatment facilities are typically designed in accordance with a 20-year planning period. This minimizes changes in the treatment capacity resulting in additional capital costs prior to the end of the design period.

This evaluation is based on a design year of 2027. Population projection is the primary basis of the design year. Projected wastewater flows and loadings are determined using a combination of population trends and expected commercial and industrial growth. Currently, Watertown does not contain any significant industrial users (SIU). Therefore, all projections are made based on normal domestic strength wastewater.

2.2 Population Projections

There are a number of methods used to predict population trends for cities such as Watertown. Historical city and county population trends are reviewed. Mathematical projections can be performed including arithmetic, geometric and linear regression methods. Often, a combination of methods, along with local intuition and conversations with City staff are involved in any sort of future population estimate.

The following population projection for the City of Watertown was developed based on planned residential developments and community growth expectations. Planned residential development through the year 2010 account for 934 new connections. After this period, a reduced rate of growth was assumed, resulting in 3,740 residential connections by the year 2027 for an equivalent population of 9,915. Historical and projected populations for the City of Watertown are presented in Table 2.1 and Figure 2.1.

Table 2.1 Historical and Projected Population Data City of Watertown		
Year	Population	
	Equivalent Connections	Population
1980	686	1,818
1990	909	2,408
2000	1,143	3,029
2005	1,307	3,464
2012	2,755	7,300
2017	3,085	8,175
2022	3,410	9,035
2027	3,740	9,915

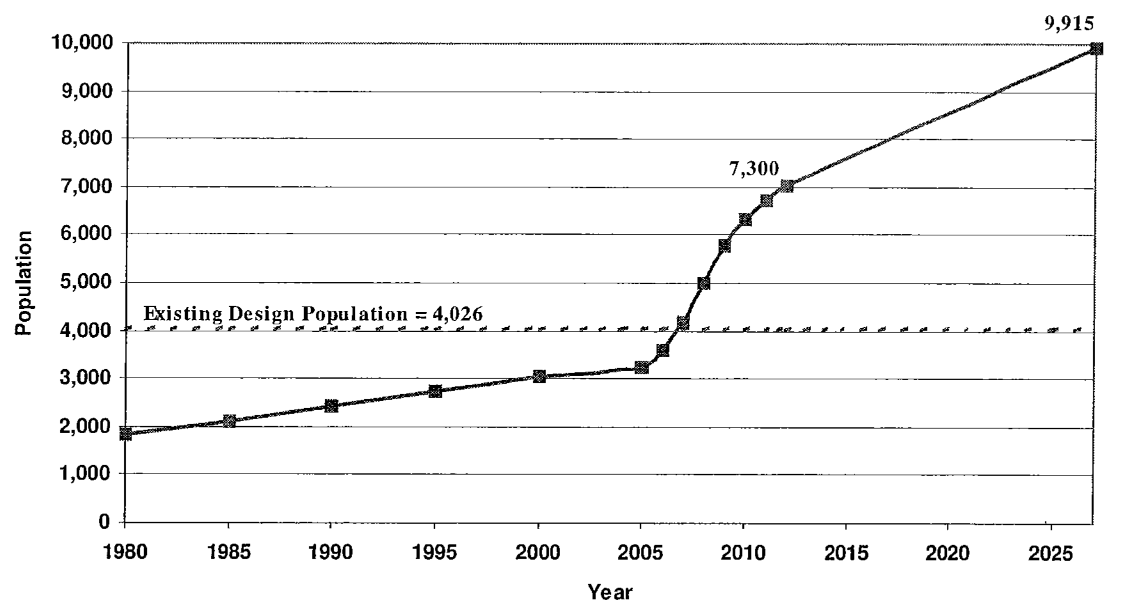

Figure 2.1 Historical and Projected Population

2.3 Wastewater Flows

A summary of the historical influent flow to the Watertown Treatment Facility for 2002 through 2005 is presented in Table 2.2 and Figure 2.2.

Table 2.2
Historical Wastewater Flows
2002 through 2005

Parameter	Flow (MGD)				
	2002	2003	2004	2005	Average
Average Daily Flow	0.362	0.286	0.317	0.340	0.326
Maximum Month	0.525	0.409	0.457	0.401	0.448
Maximum Day	1.000	0.614	0.669	0.699	0.746

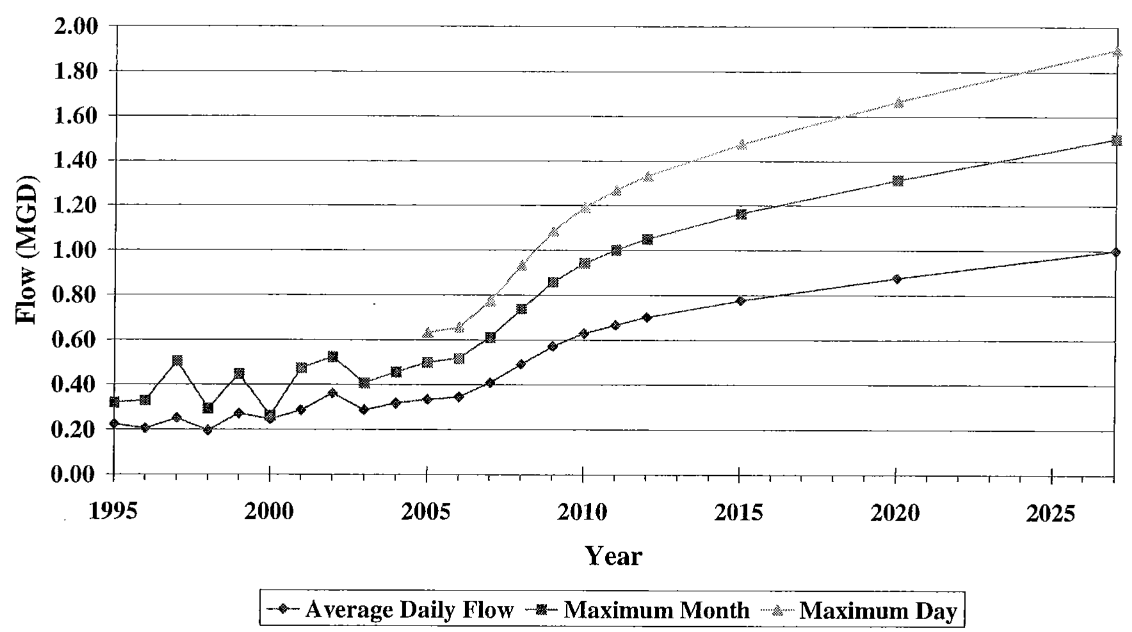

Figure 2.2 Historical and Projected Wastewater Flows

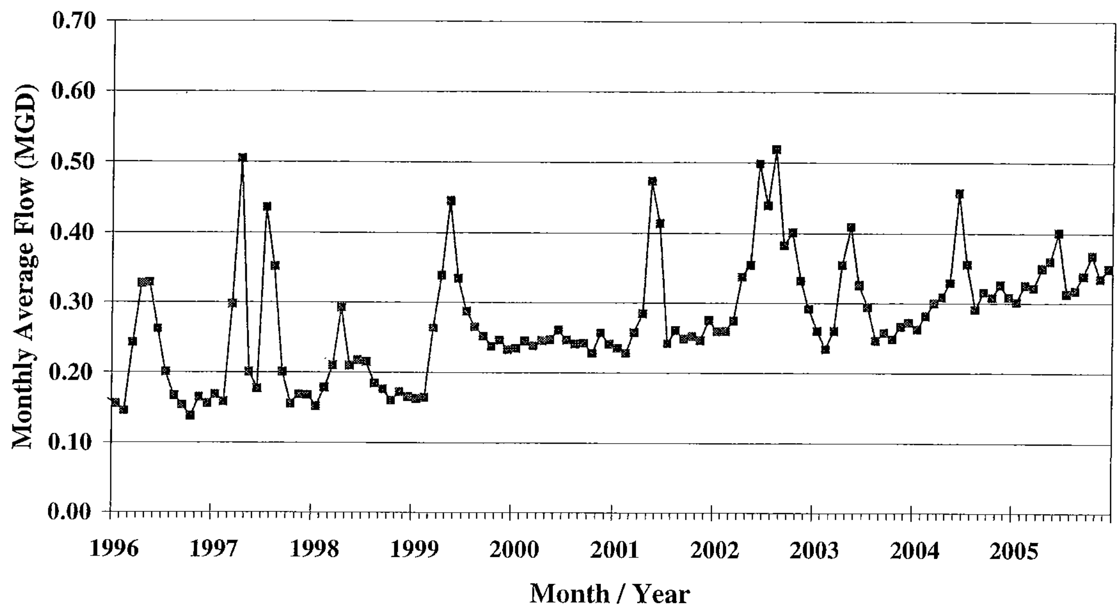

Figure 2.3 Historical Monthly Wastewater Flows

Minnesota Pollution Control Agency (MPCA) guidelines are used to determine design flows for the City of Watertown for the design year 2027 using MPCA's Determination of Design Flow worksheet. The completed worksheet is included as Table 2.3.

In completing the Determination of Design flows worksheet, wastewater flow increase due to population growth is estimated at 100 gallons per capita per day.

Table 2.3
Determination of Design Flows
City of Watertown, Minnesota – Year 2027

A.	**For Determination of Peak Hourly Wet Weather Design Flow (PHWW)**		**(MGD)**
1	Present peak hourly dry weather flow (Present ADW * 3.4 Peaking Factor)		1.100
2	Present peak hourly flow during high groundwater period (no runoff)		1.320
3	Present peak hourly dry weather flow (same as line 1)	-	1.100
4	Present peak hourly infiltration	=	0.220
5	Present hourly flow during high groundwater period and runoff		1.990
6	Present hourly flow during high groundwater (no runoff) at the same time of day as line 5	-	1.320
7	Present peak hourly inflow (2.2 inches/hour rain event)	=	0.670
8	Present peak hourly inflow adjusted for a 5-year 1-hour rainfall event (1.8 inches)		0.548
9	Present peak hourly infiltration (same as line 4)		0.220
10	Peak hourly infiltration cost effective to eliminate	-	0.000
11	Peak hourly infiltration after rehab (where cost effective)	=	0.220
12	Present peak hourly adjusted inflow (same as line 8)		0.548
13	Peak hourly inflow cost effective to eliminate	-	0.000
14	Peak hourly inflow after rehab (where cost effective)	=	0.548
15	Population increase: 6,695 at 100 gpcd times 3.1 peaking factor		2.075
16	Peak hourly flow from planned industrial increase		0.000
17	Estimated peak hourly flow from future unidentified industries		0.000
18	Peak hourly flow from other future increases		0.000
19	Peak hourly wet weather design flow (1+11+14+15+16+17+18)		**3.944**
B.	**For Determination of Peak Instantaneous Wet Weather Design Flow (PIWW)**		
20	Peak hourly wet weather design flow (same as line 19)		3.944
21	Present peak hourly inflow adjusted for a 5-year 1-hour rainfall event (same as line 8)	-	0.548
22	Present peak inflow adjusted for a 25-year 1-hour rainfall event (2.4 inches)	+	0.731
23	Peak instantaneous wet weather design flow	=	**4.126**
C.	**For Determination of Average Dry Weather Design Flow (ADW)**		
24	Present average dry weather flow: 3,220 at 100 gpcd		0.322
25	Population increase: 6,695 at 100 gpcd	+	0.670
26	Average flow from planned industrial increase	+	0.000
27	Estimated average flow from other future unidentified industries	+	0.000
28	Average flow from other future increases	+	0.000
29	Average dry weather design flow (24+25+26+27+28)		**0.992**
D.	**For Determination of 30-Day Average Wet Weather Design Flow (AWW)**		
30	Present average dry weather flow: 3,220 at 100 gpcd		0.322
31	Average infiltration after rehabilitation (where cost effective)	+	0.147
32	Average inflow after rehab (where cost effective)	+	0.365
33	Population increase: 6,695 at 100 gpcd	+	0.670
34	Average flow from planned industrial increase	+	0.000
35	Estimated average flow form other future unidentified sources: at gpcd	+	0.000
36	Average flow from other future increases	+	0.000
37	Average wet weather design flow (30+31+32+33+34+35+36)	=	**1.504**

Table 2.4 summarizes the projected design flows. Historical and projected flows are also presented in Figure 2.2.

Table 2.4 Projected Design Flows		
Parameter	**2027 Design Flow**	
	mgd	**gpm**
Average Daily Flow (ADW)	0.99	680
Average 30-Day Wet Weather Flow (AWW)	1.5	1,040
Maximum Daily Flow (MDF)	1.9	1,320
Peak Hourly Wet Weather Flow (PHWW)	3.9	2,780
Peak Instantaneous Wet Weather Flow (PIWW)	4.1	2,850

2.4 Wastewater Loadings

In addition to flow projections, pollutant loadings are required to size a wastewater treatment facility. This is often done by determining a per capita value for carbonaceous biochemical oxygen demand ($CBOD_5$), total suspended solids (TSS), total Kjeldahl nitrogen (TKN) and phosphorus (P). These per capita values are then multiplied by the projected population. Historical influent and per capita loadings are presented in Table 2.5.

Table 2.5 Summary of Historical Pollutant Loadings				
Parameter	**2002**	**2003**	**2004**	**2005**
Population	3,125	3,172	3,220	3,246
$CBOD_5$				
Influent Load (lb/day)	465	458	557	542
Per Capita Load (lb/day)	0.15	0.14	0.17	0.17
TSS				
Average Influent Load (lb/day)	419	410	469	475
Per Capita Load (lb/day)	0.13	0.13	0.15	0.15.
TKN[1]	---	---	---	---
P[1]	---	---	---	---

1: Influent Total Kjeldahl Nitrogen (TKN) and Phosphorus (P) data not available.

Domestic design loadings were calculated using the design year population equivalent of 9,915 persons and the following per capita loading rates. Design flows and loadings are summarized in Table 2.6.

$CBOD_5$	= 0.17	lb/capita/day
TSS	= 0.20	lb/capita/day
TKN	= 0.075	lb/capita/day
Phosphorus	= 0.006	lb/capita/day

Table 2.6
Design Wastewater Flows and Loadings

Parameter	Value	Unit
Design Year	2027	
Design Population	9,915	
Average Dry Weather Flow (ADW)	0.99	MGD
Average Wet Weather Flow (AWW)	1.5	MGD
Maximum Daily Flow	1.9	MGD
Peak Hourly Flow (PHWW)	3.9	MGD
Peak Instantaneous Flow (PIWW)	4.1	MGD
Carbonaceous Biochemical Oxygen Demand ($CBOD_5$)	1,685	lb/day
Total Suspended Solids (TSS)	1,983	lb/day
Total Kjeldahl Nitrogen (TKN)	427	lb/day
Total Phosphorus (P)[1]	59	lb/day

1: Phosphorus removal not currently required by effluent limitations.

2.5 Industrial Projections

The City of Watertown does not currently provide service for, nor do they anticipate serving a significant industrial user. Therefore, allocations for future industrial discharges are not included in the flow and loading projections. An industrial user agreement is required between the City and the user for all dischargers that:

- are greater than 25,000 gpd,
- are in excess of 5% of the plant flow or loading capacity,
- contain toxic pollutants, or
- result in a significant impact at the wastewater treatment facility.

2.6 Biosolids Production

Biosolids production rates are based on typical rates of 0.65-0.85 lb produced/lb $CBOD_5$ removed for extended aeration plus 15 lb produced/lb phosphorus removed by chemical precipitation. Table 2.7 summarizes the estimated biosolids production based on the design flows and loadings presented in Table 2.6 assuming 100% removal of the $CBOD_5$ and 85% removal of the influent phosphorus.

Table 2.7
Estimated Biosolids Production

Design Condition	Influent Plant Loading (lb/day)		Biosolids Production		Annual Volume (MGY)	
	$CBOD_5$	Phos.	(lb/day)	(gpd)[1]	3%	6%
Current Design	577	---	375	1,500	0.55	0.27
Proposed Design	1,685	59	1,850	7,400	2.70	1.35

1: Biosolids production based on an estimated 3.0 percent solids concentration.

2.7 Discharge Site and Effluent Limits

The existing discharge point for the facility is shown in Figure 2.4. The effluent discharges to the South Fork of the Crow River, a Class 2B, 3B, 4A, 4B, 5 and 6 receiving water. The discharge location will not change as a result of the proposed design modifications.

Effluent discharge limits were most recently issued by the Minnesota Pollution Control Agency (MPCA) in October 2004 and expire in September 2009. The current NPDES permit (Appendix A) indicates that future permit limits will enforce a mass loading limit, based on the design average wet weather flow of the Facility, as of January 1, 1988 and associated mass loading.

A request for permit discharge limits has been submitted to the MPCA for consideration; however, proposed limits have not been issued. This facility plan has been prepared based on the estimated limits presented in Table 2.8.

A $CBOD_5$ limit of 4.2 mg/L is expected based on the existing mass limit of 52.5 kg/day (23.8 lb/day). The analytical detection limit for $CBOD_5$ is 2 mg/L.

Table 2.8
Estimated Design Effluent Limits

Parameter	Current Limit		Estimated Limit	
Average Wet Weather Flow	1.26 MGD		1.50 MGD	
$CBOD_5$	5.0 mg/L	23.9 kg/day	4.2 mg/L	23.9 kg/day
TSS	30 mg/L	143 kg/day	25.2 mg/L	143 kg/day
Ammonia Nitrogen: June-Sept.	1.4 mg/L	6.7 kg/day	1.2 mg/L	6.7 kg/day
Ammonia Nitrogen: Oct.-Nov.	5.1 mg/L	24.3 kg/day	4.3 mg/L	24.3 kg/day
Ammonia Nitrogen: Dec.-March	7.7 mg/L	36.7 kg/day	6.5 mg/L	36.7 kg/day
Ammonia Nitrogen: April-May	24.0 mg/L	114.5 kg/day	20.2 mg/L	114.5 kg/day
Phosphorus	Monitor Only		1.0 mg/L	5.7 kg/day
pH	6.0-9.0 SU		6.0-9.0 SU	
Fecal Coliform (May 1–Oct. 31)	200 org/100 mL		200 org/100 mL	
Total Residual Chlorine	<0.038 mg/L		<0.038 mg/L	
Dissolved Oxygen	Monitor Only		>6.0 mg/L	
Mercury	No Limit		10 ng/L	

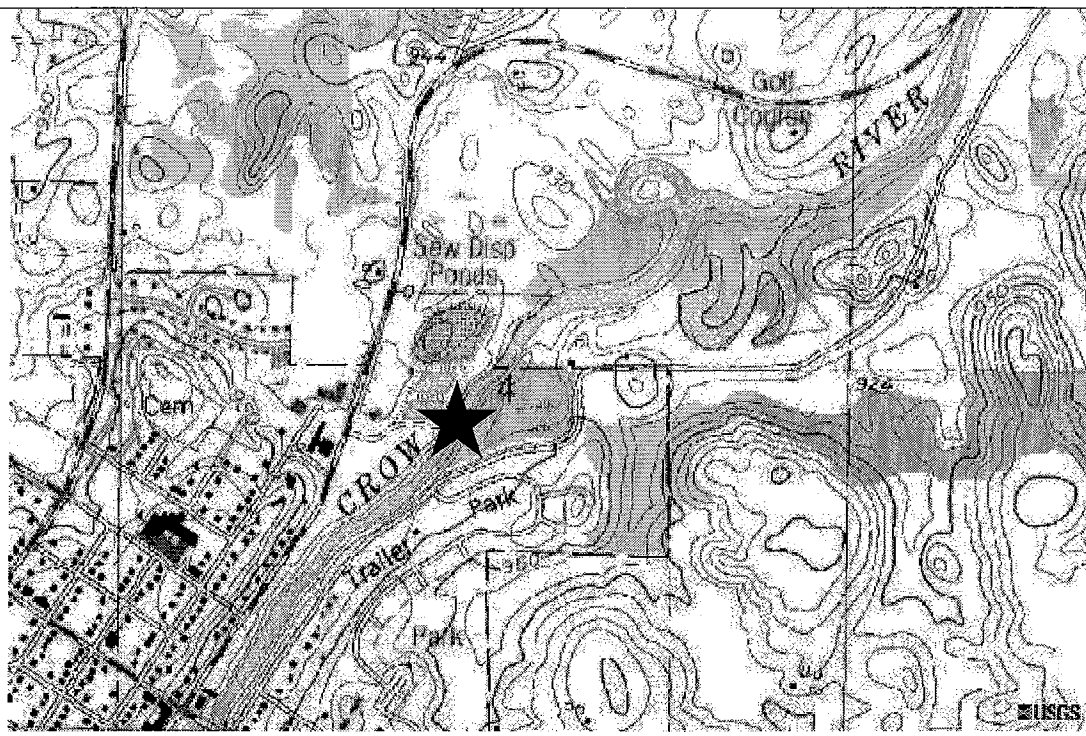

Figure 2.4 Facility Discharge Location

A TSS limit of 25 mg/L is expected based on maintaining the existing mass discharge limit of 143 kg/day (315 lb/day). This limit is easily achieved based on effluent requirements for $CBOD_5$ and phosphorus.

The ammonia-nitrogen limit for June through September may remain the same at 1.4 mg/L based on reasonable expectations of available technology. The limit for the remainder of the year is expected to drop in relation to the current permitted mass discharge.

A phosphorus limit will be imposed with the new permit. The limit may be set based on maintaining the current mass discharge or the 1.0 mg/L which is a fairly typical limit in Minnesota. Based on a sample 12 month data period on record at the MPCA, a mass limit of 9.5 lb/day was calculated (0.327 MGD * 3.5 mg/L * 8.34 lb L/mg MG). This results in a concentration limit of 0.76 mg/L at the design average wet weather flow of 1.5 MGD. This anticipated limit may vary, but will not exceed 1.0 mg/L.

The residual chlorine, fecal coliform and pH limits are not expected to change. An effluent dissolved oxygen limit will be added.

A mercury limit has also been added to most new or reissued permits. The limit of 10 ng/L quarterly average and 17 ng/L daily maximum are used for most facilities regardless of flow.

The final $CBOD_5$ limit may have the greatest impact on the evaluation of treatment alternatives. A limit of less than 5 mg/L requires more stringent filtration than the existing sand filtration process. Although the existing limit is 5.0 mg/L, which would be reduced in order to maintain a constant mass discharge limit, few facilities have received $CBOD_5$ limits less than 5.0 mg/L on a monthly average.

In addition, the existing NPDES permit requires a non-degradation review for an increase in design average wet weather flow of greater than 200,000 gpd and an increased mass loading rate for one or more of the pollutants.

3.0 EVALUATION OF EXISTING WASTEWATER TREATMENT FACILITIES

3.1 General

This section evaluates the existing wastewater treatment facility and its ability to treat the projected design flows and loadings. The overall wastewater treatment facility is best evaluated by analyzing the main unit processes and their respective treatment capacities. The current treatment facility includes influent pumping, flow equalization, screening, extended aeration activated sludge, secondary clarifiers, filtration, chlorine disinfection, anaerobic digestion and biosolids storage. A site layout for the existing wastewater treatment facilities is shown in Figure 3.1.

3.2 Evaluation of Treatment Units

3.2.1. Influent Pumping

The influent pump station was recently upgraded to 1,760 gpm (2.53 mgd). The lift station includes two submersible centrifugal pumps that discharge up to 555 gpm (0.80 mgd) to the WWTF. One pump is designed to meet the peak demand and the second pump is provided for backup. Excess flow routed to the flow equalization basin. The influent lift station will require additional improvements to meet the proposed peak instantaneous wet weather flow (PIWW) of 4.1 mgd (5,900 gpm).

Table 3.1
Influent Lift Station Capacity Evaluation

Design Criteria	Available Capacity
Peak Pumping Rate	2.53 MGD

3.2.2. Flow Equalization

The current facility utilizes an estimated 3.5 acre (4.5 MG) flow equalization basin to reduce the peak flow to the WWTF. The existing facility is designed with an average flow to treatment of 0.80 mgd.

3.2.3. Pretreatment Facilities

Existing pretreatment process consists of a mechanical bar screen located at the influent of the aeration basins. The mechanically cleaned barscreen is used to remove large objects that may damage downstream pumps and process equipment. A manually cleaned screen is also provided as a backup in case of mechanical breakdown.

The treatment capacity of this process is controlled by peak influent flow rates. Preliminary treatment capacity is based on hydraulic loading and is independent of the pollutant, or organic loading rates.

The existing mechanical screen is constructed with 1/4" bars and 3/4" clear space between bars. It is sized for a normal flow range of 0.4 to 0.8 MGD with a peak capacity of 1.9 MGD.

Due to an increased peak design flow rate, the existing screen is not adequately sized for the proposed design flow. The existing facility also does not provide grit removal that reduces sand and grit from accumulating in process basins and may damage process equipment.

3.2.4. Extended Aeration Activated Sludge Process

The aeration basins are currently operating as an extended aeration activated sludge process. In this process, the waste is biologically treated by a biomass, or "activated sludge", which is present in the aeration basin. The activated sludge is captured in the secondary clarifiers, and a portion of this returned to the aeration basin to keep the amount of biomass in the basin high. Oxygen is supplied through an aeration system to increase the rate of biological treatment. The air used to supply the oxygen also provides mixing within the aeration basin to prevent solids from settling.

The capacity of the aeration basin is determined by a combination of organic and hydraulic loadings. According to the Great Lakes-Upper Mississippi River Board of State Public Health and Environmental Managers Recommended Standards for Wastewater Facilities (Ten States Standards), the maximum recommended organic loading for extended aeration activated sludge systems is 15 lbs of $CBOD_5$/day/1,000 cubic feet of aeration basin volume.

The maximum recommended hydraulic loading of the aeration basin is approximately 18 hours. For extended aeration facilities that are required to remove ammonia, such as Watertown, the minimum hydraulic residence time is recommended to be between 18 and 36 hours (Metcalf & Eddy).

The recommended minimum aeration capacity for extended aeration treatment is 2,050 standard cubic feet (SCF) / lb $CBOD_5$.

Table 3.2 summarizes the existing aeration system capacity based on the existing 0.38 MG aeration basins and three 750 SCFM centrifugal blowers.

Based on the age of the existing fine bubble membrane diffusers, it is recommended that all existing diffusers also be replaced. Over time, the material properties of the membrane change resulting in larger bubbles and reduced oxygen transfer efficiency.

Table 3.2
Extended Aeration Capacity Evaluation

Design Criteria	Available Capacity
Organic Loading: 15 lb $CBOD_5$/1000 ft^3	780 lb $CBOD_5$/day
Hydraulic Loading: 18 hour HRT	0.52 MGD
Aeration Capacity: 2,050 SCF/lb $CBOD_5$	1,054 lb $CBOD_5$/day

3.2.5. Secondary Clarification and Return Activated Sludge

Secondary clarifiers allow the biomass and other remaining suspended solids to settle through physical separation. The activated sludge is removed from the bottom of the clarifier and either returned to the aeration basin (return activated sludge - RAS) or wasted to the anaerobic digester (waste activated sludge - WAS). The clear liquid, or clarifier effluent, is discharged to the next unit process.

The secondary clarifier capacity is determined by three factors; 1) surface overflow rate, 2) weir loading rate and 3) solids loading rate. All factors are primarily related to hydraulic loading, however, the solids loading rate is influenced by the maximum mixed liquor solids concentration in the aeration basin and the maximum return activated sludge (RAS) rate.

The surface overflow rate is limited to 900 gpd/sf at peak influent flow conditions. Only influent flow, not including RAS flow, is used in this evaluation.

The weir loading rate is also based on the peak influent flow and is limited to 20,000 gpd/ft of weir length.

The solids loading is based on the maximum estimated daily flow, maximum MLSS concentration and maximum RAS flow rate. This evaluation is based on a 150 percent RAS rate and a MLSS concentration of 3,500 mg/L.

Table 3.3 summarizes the existing secondary clarifier capacity based on two 34-ft diameter clarifiers with 28-ft 8-in diameter peripheral 90° v-notch weirs. This results in a combined surface area of 1,800 sf and a weir length of 155 ft.

Table 3.3
Clarifier Capacity Evaluation

Design Criteria	Available Capacity
Surface Overflow: 900 gpd/ft^2 (one out of service)	0.82 MGD
Weir Loading: 20,000 gpd/ft	3.10 MGD
Solids Loading: 35 $lb/day/ft^2$	64,000 lb/day (1.9 MGD)

3.2.6. Filtration

Effluent sand filters are provided to remove suspended solids from the wastewater that were not removed by the secondary clarifiers. This results in reduced TSS levels as well as a reduction in $CBOD_5$ and other pollutants. The existing filters are sized based on a design loading rate of 2 gpm/ft^2 to meet the existing $CBOD_5$ limit of 5 mg/L.

Table 3.4 summarizes the existing filtration capacity based on two 219 ft^2 traveling bridge sand filters.

Table 3.4 Sand Filter Capacity Evaluation	
Design Criteria	**Available Capacity**
Hydraulic Loading: 2 gpm/ft^2	1.26 MGD

3.2.7. Disinfection

Prior to final discharge, the treated wastewater must be disinfected to kill any remaining pathogens. Disinfection occurs with chlorine in the chlorine contact basin. Because chlorine is toxic to aquatic life, the treated water is dechlorinated prior to discharge from the facility.

The chlorine contact basin is controlled strictly by hydraulic loading. A minimum detention time of 15 minutes is required at the peak flow rate, or the maximum pumping rate to the WWTF.

Table 3.5 summarizes the existing disinfection capacity based on one, 24,000-gallon chlorine contact basin. Although the minimum contact time is 15 minutes, the contact time is reduced from 43 minutes at the current design average flow of 0.80 MGD to 18 minutes at the proposed design average flow of 1.9 MGD.

Table 3.5 Disinfection Capacity Evaluation	
Design Criteria	**Available Capacity**
Chlorine Contact Time: 15 min.	2.29 MGD
Dechlorination Contact Time: 30 sec.	2.76 MGD

3.2.8. Post Aeration

Post-aeration is provided to increase the dissolved oxygen concentration in the effluent waste stream. Fine bubble diffusers are used, similar to the aeration system, in a small detention channel/tank.

3.2.9. Biosolids Digestion and Storage

Solids wasted from the secondary clarifiers are transferred to the anaerobic digester. The digestion process reduces both the total sludge volume to be disposed of, and the amount of pathogens and vectors found in the biosolids, resulting in biosolids of less quantity and better quality. After digestion, the biosolids are transferred to an unheated storage tank where it is thickened to further reduce the volume and stored until it can be disposed of.

The anaerobic digester capacities are determined by volatile solids loadings and minimum detention time and temperature requirements. For complete mix digesters, a loading rate of 80 lb volatile suspended solids (VSS)/1000 ft^3 is allowed; however, to meet the Class "B" requirements, a minimum detention time of 15 days is also required. The biosolids are then land applied to agricultural fields and injected or disked into the soils to reduce potential vector attraction. Due to Minnesota's climate, biosolids may be applied to agricultural fields during limited time periods. Therefore, a minimum 180 days of storage is required.

Table 3.6 summarizes the existing biosolids treatment and storage capacity based on one 50,000-gallon anaerobic digester and one 140,000-gallon storage tank. The estimated solids concentration entering the digester is 3.0 percent and the solids concentration in storage is 6.0 percent. The design volatile solids concentration is 50 percent of the total solids concentration.

Table 3.6
Biosolids Capacity Evaluation

Design Criteria	Available Capacity		
	Biosolids Production		Treatment
Digestion: 80 lb VSS/1000 ft^3/day	1069 lb /day	4,275 gpd	970 lb $CBOD_5$/day
Time/Temperature: 15 days at 35-55°C	834 lb/day	3,333 gpd	960 lb $CBOD_5$/day
Storage: 180 days	460 lb/day	1,833 gpd	<420 lb [1] $CBOD_5$/day

1: Assumes 50,000 gallons at 3 percent and 140,000 gallons at 6 percent for 180 days.
2: Equivalent treatment capacity including chemical phosphorus removal. Approximate treatment capacity of 708 lb. $CBOD_5$/day without chemical phosphorus removal (0.65 lb solids/lb $CBOD_5$ removed).

The relationship between biosolids capacity and the equivalent $CBOD_5$ treatment capacity also includes biosolids production based on chemical precipitation of phosphorus. Without chemical phosphorus removal, the treatment capacity per pound of biosolids is increased by approximately 70 percent.

3.3 Hydraulics and Process Control

The existing treatment facility is designed for a hydraulic flow of 0.8 MGD. It is anticipated that site piping revisions will be required to meet the proposed peak treatment capacity of 1.9 MGD.

3.4 General Plant Conditions

Overall, the structures within the facility are structurally sound. The process and mechanical equipment within the facility are generally in good condition.

3.5 Summary

Review of the existing wastewater facility indicates the existing facility is not adequately sized for the current and projected flows and loadings. The limiting design parameter for each unit process is summarized in Table 3.7.

Table 3.7
Wastewater Treatment Facility Capacity Evaluation

Unit Process	Limiting Capacity	Capacity
Influent Pumping	Peak Instantaneous Flow Rate	2.53 MGD
Preliminary Treatment	Equipment Replacement	0.80 MGD
Aeration	Organic ($CBOD_5$) Loading	780 lb $CBOD_5$/day
Clarifiers	Surface Overflow, Solids Loading	0.82 MGD
Filter	Hydraulic Loading	1.26 MGD
Chlorination	Detention Time	2.29 MGD
Anaerobic Digestion	Volatile Solids Loading	760 lb $CBOD_5$/day
Biosolids Storage	Detention Time	420 lb $CBOD_5$/day

1: Chlorination capacity meets minimum requirements at reduced peak flow rates.

4.0 WASTEWATER TREATMENT FACILITY IMPROVEMENTS

4.1 General

In the following paragraphs, several categories of alternatives are considered. Generally, alternative solutions include: 1) optimum operation and/or expansion of existing facilities; 2) regionalization; 3) construction of a new facility. Only potentially feasible alternatives will receive further evaluation.

4.1.1. Optimum Operation/Expansion of Existing Facilities

This category continues to utilize the existing facility. Once the hydraulic and organic treatment capacity has been exceeded, facility operations are modified and/or expanded to meet the projected treatment levels required. Factors such as the age and condition of the existing facilities and available land for expansion influence the feasibility of this category of alternatives.

There are several unit processes in good condition and available space for facility expansion. Therefore, expansion of the existing facilities is considered a feasible alternative.

4.1.2. Regionalization

This category includes the possibility of diverting wastewater to a wastewater treatment facility of a nearby community.

Due to the distance associated with regionalizing in comparison to the magnitude of the required plant expansion, regionalization was not considered feasible at this time.

4.1.3. Construction of a New Facility

The current facility is a mechanical treatment facility with continuous discharge including a main lift station, mechanical screen, activated sludge, filtration, chlorination, and biosolids facilities.

At this time, it is not considered feasible to abandon the existing treatment facility and construct a new facility as several of the unit processes are in good condition and can be utilized to meet the proposed design flows and loadings. The recommended improvements discussed are then based on optimization and expansion of the existing facility.

4.2 Wastewater Improvements

Based on the design flows and loadings presented in Section 2.0. A proposed design flow of 1.9 MGD is recommended. This recommendation requires the continued use of the equalization basin for flow in excess of 1.9 MGD. The availability of the equalization basin is also used to meet the required reliability criteria. Therefore, the flow to the WWTF must be limited in order to take a basin, clarifier, filter or other unit process out-of-service.

Wastewater and biosolids improvements required to meet the design flow and loadings are discussed below.

The following proposed improvements are shown in Figures 4.1 and 4.2.

4.2.1. Influent Lift Station Upgrade/Meter Manhole Relocation

The influent lift station capacity must be upgraded to meet the peak instantaneous wet weather flow rate of 4.1 MGD (5,900 gpm). Due to the increase in the flow rate and other recommended improvements, the influent flow meter must also be relocated. The recommended location is in the proposed preliminary treatment building.

The lift station capacity was recently upgraded to 2.53 MGD. Additional improvements at the lift station and forcemain should be scheduled as the peak flow increases. Concurrent projects to reduce infiltration and inflow should extend the useful life of the recent upgrade.

4.2.2. Screening and Grit Removal

Screening and grit removal may be provided either prior to flow equalization with a design flow of 4.0 MGD or prior to aeration with a design flow of 1.9 MGD. A new preliminary treatment building with a mechanical fine screen, vortex grit removal and a grit classifier is recommended prior to flow equalization. Removal of solids prior to equalization reduces the potential for odors in the equalization basin. The existing mechanical screen will be removed from service.

4.2.3. Extended Aeration

The extended aeration system includes the basins, blowers and diffusers. To expand the aeration capacity, an additional three aeration basins are proposed with replacement of the existing blowers and diffusers. Three additional aeration basins will expand the aeration volume to 1.1 MG, resulting in an 18-hour detention time at the average wet weather design flow of 1.5 MGD. An increased blower capacity for the entire facility results in three new blowers at 1,200 SCFM each. In addition to the basin expansion and blower replacement, fine bubble diffusers are required for the new and existing basins.

4.2.4. Clarifier

The existing clarifier capacity is limited by the design surface overflow rate of 900 gpd/ft^2. One additional 52-ft diameter clarifier is proposed. This increases the peak treatment capacity to 1.9 MGD based on the solids loading with 3,500 mg/L and 150 percent RAS. The estimated peak hydraulic capacity based on a surface overflow rate of 900 gpd/ft^2 and a weir loading rate of 20,000 gpd/ft are 5.78 MGD and 3.55 MGD, respectively.

4.2.5. Filtration

The existing traveling bridge sand filters may not reliably meet the reduced $CBOD_5$ effluent limit of 4.2 mg/L. In addition, expansion of the sand filtration system at the current loading rate (gal/sf) requires a building expansion, resulting in relocation of the chlorine contact basin.

Other alternatives include cloth filters and membrane (micro or ultra) filters. Both cloth and membrane systems result in higher treatment rates; however, cloth filters also may not reliably meet the low $CBOD_5$ effluent limit. Installation of a cloth or membrane filter system is planned for the existing filter building with renovations to the existing treatment channels.

Membrane filtration is recommended based on the reliability to meet $CBOD_5$, TSS and phosphorus limits.

4.2.6. Disinfection

The current design of the chlorine contact basin is based on a minimum detention time of 15 minutes at peak flow. The existing basin provides 15 minutes at a flow rate of 2.29 MGD. This design is adequate for the design average flow rate of 1.9 MGD, resulting in a detention time of 18 minutes.

4.2.7. Nutrient Removal

Nutrient removal applies to the reduction in nitrogen and phosphorus in the facility effluent. Biological and chemical/physical removal methods are proposed to meet the stringent ammonia, TKN and phosphorus limits.

Addition of an anaerobic selector basin prior to the extended aeration process increases the biological nutrient uptake. The selector basin encourages natural selection for organisms capable of an increased phosphorus uptake. Under ideal conditions, the biomass may accumulate 4 to 12 percent phosphorus as opposed to the 1.5 to 2.0 percent that is required for cell growth

A selector basin is recommended prior to the aeration basins. An anaerobic basin with a design detention time of 3 hours is provided for the influent flow and return activated sludge. The selector basin is provided with complete mechanical mixing.

The design recommendations described above result in a 0.24 million gallon anaerobic basin and an estimated two submersible mixers.

A new control structure for flow distribution to the existing and new aeration basins will be incorporated into the selector basins.

Biological phosphorus removal is often not reliable and/or able to achieve the proposed low, less than 1.0 mg/L, discharge limit. Therefore, a chemical (ferric chloride or alum) removal system is also required. This system includes a chemical storage tank and feed system. The chemical is introduced into the wastewater prior to clarification. The phosphorus in the wastewater is precipitated with ferric chloride or alum and removed in the waste activated sludge.

4.3 Biosolids Improvements

Based on the design flows and loadings presented in Section 2.0, a projected design biosolids production rate in 2027 is 1,850 lb/day. This design production rate is based on extended aeration and chemical phosphorus removal. Alternatives for treatment and disposal of the biosolids are discussed below.

4.3.1. Biosolids Regulations and Classification

Biosolids consist of solids removed from raw wastewater and biological solids generated by the treatment process. Proper handling and disposal is an important aspect of wastewater treatment. There are many options available for biosolids treatment and disposal. Treatment is typically a two-part process consisting of treatment to stabilize and/or thicken the biosolids concentration and storage. Land application is the most common disposal alternative. Land application is governed by both federal and state regulations.

Federal Regulations

Federal regulations governing land application of biosolids are addressed in the Code of Federal Regulations (40 CFR Part 503). The 503 Regulations divide biosolids use and disposal practices into five subparts:

- General Provisions
- Land Application
- Surface Disposal
- Pathogen and Vector Attraction Reduction
- Incineration

The categories dealing with the application of biosolids to agricultural or non-agricultural lands and pathogen and vector attraction reduction are the main concerns for municipal biosolids generators. Standards for numerical pollutant limits and the level of management control are defined in these categories.

Land utilized for biosolids application is classified as agricultural or non-agricultural. Agricultural land is defined when biosolids are applied for the purpose of using the nutrients and soil conditioning properties for crops grown for either direct or indirect human consumption or animal feed or grazing. Non-agricultural land is defined as land where no food, feed crops, or grazing occurs.

The 503 Regulations impose a maximum annual application rate of 22.5 tons dry solids/acre/year to agricultural land regardless of the biosolids quality.

Individual pollutants or annual nitrogen loading rates may further limit the maximum allowable application rate. Regulations limit the allowable concentration for ten heavy metals. Annual nitrogen loading levels are limited by the type of cover crop and the crop yield. Monitoring, record keeping and reporting are required to ensure maximum limits are not exceeded.

The Regulations also include requirements to reduce pathogenic bacteria and require municipalities to practice vector attraction reduction methods, such as immediate incorporation into the soil.

State Regulations

Minnesota's regulations governing the disposal and utilization of sewage biosolids are addressed in Minnesota Rules Chapter 7041 and regulated by the Minnesota Pollution Control Agency (MPCA). The Minnesota Rules have adopted the Federal 40 CFR Part 503 Regulations. The following is a summary of the monitoring, application and record keeping requirements included in the Federal and State biosolids regulations.

Biosolids to be land applied must be analyzed for the following parameters:

- pH
- Total Solids (TS)

- Total Volatile Solids (as a percent of TS)
- Nitrogen (Ammonia, Nitrate and Total Kjeldahl Nitrogen)
- Heavy Metals (Zn, Cu, Pb, Cd, Hg and Cr)
- PCB's

Application rates are limited to the annual nutrient loading and cumulative heavy metal loading. Annual nitrogen loading is calculated based on the cover crop and crop yield. The cumulative heavy metal loading is monitored from year to year and varies based on the soil cation exchange capacity.

Biosolids processing must include a Process to Significantly Reduce Pathogens (PSRP). Vector attraction reduction methods must also be provided. The application site must be evaluated based on the soil properties, distance from waterways, private wells and residences.

Monitoring and record keeping requirements include annual biosolids sampling, soil, vegetation and groundwater analysis and annual reporting to the MPCA.

<u>Biosolids Classification</u>

Biosolids treatment methods fall into two categories based on the resulting classification of the material. Class "A" biosolids have received treatment to reduce the pathogen and vector attraction. Fecal coliform limits for Class "A" biosolids are less than 1000. Class "B" biosolids also receive treatment, however, the fecal coliform limit is 2 million. Based on the different levels of treatment, Class "A" biosolids have fewer restrictions on the final disposal options. Class "B" biosolids can be land applied only to agricultural lands, whereas Class "A" biosolids can also be used on non-agricultural land where human contact may occur.

Class "A" treatment alternatives include Autothermal Thermophilic Aerobic Digestion (ATAD) and heat drying. Class "B" treatment alternatives include anaerobic and aerobic digestion. The existing treatment process produces Class "B" biosolids with mesophilic anaerobic digestion.

4.3.2. Anaerobic Digestion (Mesophilic)

Thickening and Digestion

The existing anaerobic digester provides the time and temperature requirements to meet Class B biosolids up to a wasting rate of 3,760 gallons/day. This is approximately one-half of the 2027 design biosolids wasting rate of 7,410 gallons/day at 3% solids following the thickener. Therefore, a second 20-ft diameter anaerobic digester and associated equipment, including an additional heat exchanger, are recommended.

Biosolids Storage and Disposal

The existing biosolids storage provides 140,000 gallons of storage. Based on an estimated 6% solids concentration, an additional 700,000 gallons of storage is required to meet the 180-day storage requirement for the 2027 total production rate of 1,850 lb/day.

4.3.3. Autothermal Thermophilic Aerobic Digestion (ATAD)

Conventional aerobic digestion is a process that utilizes biochemical oxidation of cell biomass to produce carbon dioxide, water and energy. The Autothermal Thermophilic Aerobic Digestion Process (ATAD) is a refinement of conventional aerobic digestion in which insulated reactor tanks are used to retain the energy released during oxidation, thereby allowing the temperature of the system to increase until a thermal balance occurs. With proper feed-solids characteristics, oxygen input and mixing, the amount of energy released is sufficient to reach and maintain thermophilic temperatures without the addition of supplemental heat.

When operated in batch-feed mode, the high temperatures generated within the ATAD system result in disinfection and stabilization required to meet the Class "A" requirements of 40 CFR Part 503. The biosolids are heated to over 120°F and processed for approximately 10 days. The aeration equipment and mixing process allows the microorganisms to generate the heat required for the process. Very little, if any, external heating is required.

There are two major manufacturers of the ATAD process and equipment. Major components of an ATAD facility include:

- Pre-ATAD Holding
- Biosolids Thickening
- ATAD Reactors
- Post ATAD Storage and Disposal

Pre ATAD Holding

The pre-ATAD holding tank allows for liquid biosolids to be stored for a short period of time, usually less than 30 days. This provides for the ATAD to be fed in a batch mode, as well as providing operations flexibility. Since anaerobic digestion is not required with an ATAD facility, the existing anaerobic digester and storage tank are available for pre-ATAD storage and thickening.

The existing 50,000-gallon digester and 140,000-gallon storage tank will provide an estimated 25 days of storage at the 2025 design biosolids production rate of 7,410 gallons per day (estimated at 3 percent solids).

Thickening

Biosolids must be thickened to approximately 4 percent solids prior to treatment. The thicker concentration is required for the autothermal digestion process to generate adequate heat.

Digestion

The ATAD process requires oxygen, mixing and foam control equipment. Oxygen is provided by either blowers or spiral type aerators. The mixing is provided by either circulation aerators or jet mechanical type mixers. Foam cutters are required to maintain foam levels resulting from the aerobic operation at high temperatures.

The digesters are designed for a 10-day detention time resulting in two, 22-ft diameter by 15-ft deep digesters. The working depth of each digester is 10-ft. This is based on the 2025 design production rate of 5,560 gallons per day at 4 percent solids concentration.

The thickened biosolids are fed to the ATAD tanks on a batch basis, typically once every day. Each batch is separated by a minimum of 23 hours allowing one-hour for fill and discharge. Biosolids are mixed and aerated throughout the remaining 23-hour cycle. The ATAD process is automatically monitored and controlled by a Program Logic Control System (PLC) to minimize operator requirements. The ATAD reactors are enclosed in a steel-framed, pre-engineered building to reduce energy costs, provide better access for maintenance and better control for potential odors.

Biosolids Storage and Disposal

Following treatment by the ATAD process, the biosolids are transferred to a storage facility. The Class "A" treated biosolids are not restricted to agricultural lands. However, since the biosolids concentration is approximately 4 percent solids, the most feasible disposal option is land application to agricultural lands as a fertilizer supplement.

Similar to aerobic or anaerobic digestion, approximately 1.00 to 1.35 million gallons of storage is required for 180 days at 3-4 percent solids concentration.

4.3.4. Heat Drying

Heat drying technology originated with direct heat systems that required large quantities of air to convey the heat and transport biosolids through the dryer. These direct heat systems resulted in large capital and operating costs.

Recent developments with indirect heat dryers have lowered capital and operating costs associated with heat drying. In addition, indirect heat dryers operate at lower temperatures than direct heat dryers, which reduce the risk of fires and explosions associated with operation of the dryers.

Indirect heat drying places dewatered biosolids into a specially designed heat drying/mixing chamber where the material is heated and the liquid is evaporated. The heat and evaporation kill pathogens and produce a Class "A" exceptional quality (E.Q.) product.

Although anaerobic digestion of the biosolids prior to heat drying is not required to meet the Class "A" classification, continued use of the digester will allow the City to utilize the methane gas as a supplemental fuel source and the digester will reduce the volatile solids, which reduces the overall volume of biosolids to heat drying.

An additional 20-ft diameter anaerobic digester is required to provide 15-days of detention time at the projected biosolids production rate.

There are two major manufacturers of the indirect heat drying process and equipment. Major components of a drying facility include:

- Pre-Treatment Storage
- Biosolids Thickening
- Belt Drying
- Post Drying Storage and Disposal

<u>Pre Treatment Storage</u>

Following digestion, the biosolids are transferred to a pre-drying storage tank prior to thickening and heat drying. The existing 140,000-gallon storage tank provides approximately 25 days of storage at the projected biosolids production rate and an estimated 3 percent solids.

<u>Thickening/Dewatering</u>

Prior to heat drying, the biosolids must be dewatered to approximately 18-20 percent solids to reduce the volume of water that must be evaporated. Alternatives for mechanical dewatering include a belt press or a centrifuge. Both alternatives require chemical conditioning with a polymer prior to dewatering. The estimated solids concentration from a belt press is approximately 15-20 percent. This value is slightly higher with a centrifuge. Capital costs for a belt press are similar to a centrifuge; however, operation and maintenance costs are higher for a centrifuge. Either technology may be used with the heat drying alternative.

Drying

The drying process is a continuous feed process where dewatered solids are slowly conveyed on a belt while heated air is circulated through the belt. The hot air is supplied indirectly from a natural gas boiler and heat exchanger. The heat exchanger eliminates the chance of a spark or hot flue gas from igniting the dry biosolids. The process is highly automated and is typically operated 24 hours per day to maximize process efficiency; however, staff is required only during normal working hours.

The process is operated under a vacuum at all times, preventing the release of odors or dust. Estimated final solids concentration is in excess of 95 percent.

The Andritz belt Drying System (BDS) uses back mixing of the dried solids with the influent biosolids. The back mixing creates a dryer product and a dense final product with similar size and density to commercial fertilizers. The BDS has a built-in belt washer that automatically cleans the belt.

The Kruger BioCon system uses a progressive cavity pump to transport the biosolids without back-mixing from the pre-drying storage hopper to a series of depositors. The depositors have ½" rubber nozzles that place thin strands of biosolids on a traveling stainless steel belt.

Biosolids Storage and Disposal

Dry Class "A" biosolids disposal options include 1) fertilizer supplement to agricultural lands, 2) fertilizer supplement to non-agricultural lands, 3) added supplement to commercial fertilizer process facilities, 4) fuel source for energy incineration facilities, and 5) supplement for composting materials.

The storage requirement for dry biosolids is dependent on the final disposal alternative. If long term storage (greater than 60 days) is required, the biosolids can be stored in 1000 lb. bags in a covered building. The dryer can be designed with an automatic bagger or a loadout facility dependent on the method of disposal.

4.4 Facility Classification

Based on the proposed improvements, the Watertown WWTF will become a Class A treatment facility. A completed facility classification worksheet is included in Appendix B.

The current facility is classified as Class B. Additional requirements for a Class A facility include a Class A operator, an additional 8 hours of operator training every three years, and more frequent analytical sampling and analysis. In addition, a Type IV certified operator is required for land application of biosolids.

5.0 CONSTRUCTION COST ESTIMATE

5.1 General

In order to prepare the preliminary construction cost estimates presented herein, various material and equipment manufacturers and suppliers were contacted. Published and unpublished data on costs for similar kinds of construction were also utilized.

The estimated costs are for January 2006. Increases in construction costs due to inflation are not taken into account. The cost estimates presented here are intended for use as a guideline in the decision process. Once preparation of final drawings and specifications is underway, more refined cost estimates will become available.

5.2 Capital Cost

The estimated capital costs for the wastewater liquid stream treatment alternatives and the biosolids heat drying alternative presented in Section 4 are summarized in Table 5.1.

Table 5.1
Estimated Construction Cost for Wastewater Improvements

	Item	Cost
1.	Mobilization, Bonds and Insurance	$400,000
2.	Site Work / Excavation	150,000
3.	Site Piping	300,000
4.	Preliminary Treatment	560,000
5.	Aeration Basins, Blowers and Diffusers	1,040,000
6.	Chemical Feed	70,000
7.	Clarification	500,000
8.	Ultrafiltration	1,570,000
9.	Biosolids Thickening & Heat Drying	2,520,000
10.	Mechanical & Electrical	1,350,000
	SUBTOTAL	**$8,500,000**
11.	Engineering, Legal, Administration	900,000
	TOTAL ESTIMATED CONSTRUCTION COST	**$9,400,000**

5.3 Operation and Maintenance Cost

The existing operations and maintenance budget for the sewer fund (collection and treatment) is approximately $610,000. Table 5.2 summarizes the existing expenses and identifies proposed changes based on the proposed facilities. The O&M expenses following the plant upgrade are estimated at $650,000.

Table 5.2
Estimated Annual Operations & Maintenance Expenses

	Item	Cost
1.	2006 Operating Budget	$610,000
2.	Additional Employee	*
3.	Increased Sampling & Analysis	*
4.	Increased Electric	20,000
5.	Increased Natural Gas	20,000
6.	Additional Chemical Costs	23,000
7.	Biosolids Disposal	-25,000
8.	Additional Replacement Costs	2,000
	TOTAL ESTIMATED ANNUAL OPERATIONS & MAINTENANCE EXPENSES	**$650,000**

* Additional employee and increased sampling already included in the 2006 operating budget.

5.4 Annual Equivalent Cost

Annual sewer fund expenses are summarized in Table 5.3. The existing debt service expires in 2014 and the proposed debt service is based on the $9.4 million cost estimate financed for 20 years at 2 percent interest.

Table 5.3
Annual Sewer Fund Expenses

Item	Expense
Existing Debt Service	$348,000
Proposed Debt Service	$575,000
Operations & Maintenance	$650,000

6.0 RECOMMENDATIONS AND IMPLEMENTATION

6.1 Recommendations and Implementation

Due to the age of the existing facility, the recommended improvements are based on expansion of the existing facility. The existing pretreatment, screening and grit removal, should be replaced due to the condition of the existing equipment. The secondary treatment processes, aeration and clarification may be expanded to meet the projected loadings. Effluent filtration recommendations include replacement of the existing technology with membrane filtration to meet the increased capacity and effluent limitations. Recommended biosolids improvements include thickening, and heat drying to achieve a much smaller volume of Class A biosolids.

6.2 Project Funding

The City of Watertown will construct and operate the proposed treatment facility improvements and may request loan and grant participation from various funding agencies. A common funding plan is administered through the Public Facility Authority (PFA). This program establishes low interest loans and grants for implementation of wastewater treatment projects. The Public Facility Authority is the State's administrator for these funds. Typically, low interest loans are available at 2-3 percent interest for 20 years.

6.3 User Fees and Allocations

Wastewater user fees are typically designed to support the wastewater collection and treatment system without support from local taxes or operating budgets. In addition, user rates should be designed such that each user pays a representative share for both the capital and O,M&R expenses reflective of the actual flow and loading discharged to the system.

The existing user rates include a sewer access charge of $4,000, an $8.00 monthly base charge and a usage charge of $4.22/1000 gallons. An area charge is under consideration by the City as an additional revenue source.

6.4 Implementation Schedule

The proposed implementation schedule for the wastewater treatment facility improvements is presented in Table 6.1.

Table 6.1
Project Implementation Schedule
City of Watertown Wastewater Treatment Facility Improvements

Item	Date
• City Council Approves Wastewater Treatment and Biosolids Alternatives	January 2006
• Council Passes Resolution Adopting the Facility Plan	February 2006
• Public Hearing	March 2006
• Submit Facility Plan to MPCA	March 2006
• MPCA Reviews and Approves Facility Plan	April-May 2006
• Prepare Design Report for City Review	April-May 2006
• Prepare Plans and Specifications	June-August 2006
• MPCA Reviews and Approves Plans and Specifications	September-December 2006
• Advertise, Bid and Award Construction Contract	January-February 2007
• Initiate Construction	June 2007
• Complete Construction	June 2009