Town of Hume

Allegany County, New York

TOWN OF HUME/TOWN OF CANEADEA

MUNICIPAL WASTEWATER TREATMENT SYSTEMS

CONSOLIDATION STUDY

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I. BACKGROUND

This study has been prepared for the benefit of the Towns of Hume and Caneadea in Allegany County, New York under the Shared Municipal Services Incentive Program administered by New York State Department of State. Both Towns have partnered with the State to investigate the consolidation of their wastewater treatment systems for the primary objectives of reducing total treatment costs and providing capacity for future growth.

No previous studies have been done on this specific consolidation objective. The Town of Hume has a history of consolidation. Several years ago the Village of Fillmore dissolved their municipal structure, and the Town of Hume now owns and operates the municipal water and sewerage systems.

More recently, the Town of Hume installed a water main to interconnect the Hume water system with the Caneadea water system. The Town of Hume has only one water well supply, so this metered interconnection provides a back up water source, should the Hume well be out of service. This is another example of the inter-municipal agreements and consolidation efforts that have occurred within these communities.

This study will provide detailed information on the sewerage systems in Hume and Caneadea in order to determine alternatives that would consolidate the two wastewater treatment facilities into one regional facility serving the needs of the Town of Hume and Caneadea.

II. OBJECTIVES OF STUDY

The primary objective of this study is to compile technical and cost data from the Hume and Caneadea Wastewater treatment facilities and to develop alternatives to consolidate these systems into one regional facility. Each year the municipalities are faced with increasing operating costs and environmental regulations. In an effort to control costs and provide capacity for future growth, it is imperative that these communities develop a long term plan to best address these challenges and to optimize the funds available for wastewater treatment.

This study will conclude with alternative(s) that address the concept of a regional wastewater facility for the Towns of Hume and Caneadea. It is most likely that one or the other facility will remain and be expanded to accommodate both municipal systems. In order to best determine the future design capacity for both sewerage systems, this study will also include a review of the sewer service areas and the projected growth areas. This future growth component will be included in the alternatives analysis and cost estimations.

III. THE TOWN OF HUME'S EXISTING WASTEWATER TREATMENT SYSTEM

A. SYSTEM DESCRIPTION AND TREATMENT CAPACITY

The Town of Hume's wastewater collection system was built as a grey water system, with each sanitary service having an individual septic tank. The purpose of each septic tank is to provide primary treatment, i.e. primary settling and anaerobic digestion of accumulated solids. The septic tanks discharge into small diameter (6-inch) interceptors which convey the primary treated wastewater to a pump station located in the northeastern part of Fillmore off Route 19A. The pump station transports the primary treated wastewater to the wastewater treatment facility (WWTP) located north of the Fillmore west of Route 19A. Refer to Figure III.1 for locations of the pump station and WWTP.

The WWTP consists of primary settling tank, dual siphon dosing tank, three (3) single pass intermittent sand filters, flow monitoring manhole, and a six inch diameter treated effluent pipe. Refer to Figure III.2 for a flow schematic of the existing WWTP.

This collection and treatment system built over twenty years ago was less costly than constructing a conventional wastewater treatment facility because the septic systems were already in place, and the WWTP organic loading is lower than for a conventional system.

However, the sand filter system has been plagued with operational challenges over the years. Sand filters are sensitive to solids overloading and required replacing the sand and re-building the beds on at least one occasion.

B. SLUDGE PROCESSING AND DISPOSAL

The sludge generated in the individual septic tanks is currently pumped and hauled to the Town of Caneadea WWTP for processing.

C. INDIVIDUAL CAPACITIES OF UNIT TREATMENT PROCESSES

The hydraulic and treatment capacities of the wastewater treatment unit processes at the Town of Hume's existing WWTP are summarized in Table III.1 below.

Table III.1: Hume WWTP - Summary Individual Hydraulic Capacities of Unit Treatment Processes

Unit Process	Estimated Capacity (gpd)	
	Peak Hourly	Average Daily
Interceptor Sewer	257,184	64,296
Pump Station	116,640	29,160
Forcemain	451,296	112,824
Dual Siphon Dosing Tank	1,123,200	280,800
Sand Filters	-	40,000
Flow Meter	636,480	159,120
Outfall Pipe	257,184	64,296

As shown in Table III.1, the overall hydraulic capacity of the existing WWTP is limited by the pump station and the sand filters. The sand filters capacities are restricted by the surface area of the sand and a loading rate of 5 gpd/sq. ft. Refer to Appendix A for background calculations.

D. CURRENT SERVICE AREA AND USER BASE

The current service area of the Town of Hume WWTP includes the hamlet of Fillmore. The current user base consists of 233 EDUs (equivalent dwelling units). This total includes Fillmore Central School which consists of 10 EDUs.

E. HISTORIC WASTEWATER FLOWS

The Town of Hume measures the flow at the WWTP via a 30 degree V-Notch weir located downstream of the intermittent sand filters. The depth of flow passing through the weir is measured and that depth corresponds to a specific flow rate. The flows recorded in the WWTP Discharge Monitoring Reports (DMRs) from January 2005 – October 2007 are summarized in Table III.2 below.

Table III.2: Hume WWTP - Summary of Historic Wastewater Flows

Year	Flow (gpd)	
	Average	Maximum
2005	30,000	40,000
2006	30,000	40,000
2007 (only through October)	29,000	39,000

The existing WWTP flows appear to be very consistent from the data reviewed for this study. The maximum flows indicated in Table III.2 above represent maximum average daily flows recorded, maximum or peak instantaneous flows are not recorded in the DMRs. The average to maximum average flows can vary by up to 10,000 gpd or approximately 22% of the current permitted flow of 45,000 gpd.

F. HISTORIC WASTEWATER QUALITY

The existing wastewater treated at the WWTP is primarily comprised of domestic waste and limited commercial waste. A summary of (biochemical oxygen demand) BOD_5 concentrations, total suspended solids (TSS) concentrations, pH, total Kjeidahl nitrogen (TKN) and temperature in the influent wastewater from January 2005-October 2007 is presented in Table III.3 below.

Table III.3: Hume WWTP - Influent Wastewater Quality

Parameter	Range		
	Average	Minimum	Maximum
BOD_5	97	52	143
TSS	46	12	103
pH (S.U.)	7.3	6.7	7.9
TKN	50	29	110
Temp (°C)	14.2	9.0	19.0

The sample data in Table III.3 above represent wastewater which has received primary treatment via the individual septic tanks. In comparison to typical domestic wastewater the Town of Hume's WWTP BOD_5 concentration is weak (typically 110 mg/l) to medium (typically 220 mg/l) strength in nature. The TSS concentrations are weak (typically 100 mg/l). The ph of the influent wastewater is generally neutral ranging between 6.7 and 7.9. The average TKN concentration present in the WWTP influent wastewater is considered medium strength when compared with typical TKN concentrations (20 mg/l to 85 mg/l). The temperature of the influent wastewater fluctuates based on seasonal weather conditions with the lowest influent temperatures occurring in the winter months and the highest temperatures occurring in the summer months.

G. CURRENT PERMITTED EFFLUENT LIMITS

The Town of Hume's existing WWTP currently operates under a State Pollutant

Discharge Elimination System (SPDES) permit number NY0203858 regulated by the New York State Department of Environmental Conservation (NYSDEC). The SPDES permit effluent limits are summarized in Table III.4 below.

Table III.4: Hume WWTP - SPDES Permit No. NY0203858 Effluent Limits

Parameter	Limits	
Flow	45,000 gpd*	
BOD_5	30 mg/l* 45 mg/l**	11.3 lbs/day* 16.9 lbs/day**
TSS	30 mg/l* 45 mg/l**	11.3 lbs/day* 16.9 lbs/day**
pH (S.U.)	6.0 to 9.0	
Settleable Solids	0.1 ml/l	
Dissolved Oxygen	5 mg/l***	
UOD	75 mg/l****	28.1 lbs/day****

* - 30 Day Arithmetic Mean

** - 7 Day Arithmetic Mean

*** - Daily Minimum

**** - June – October only

In addition the BOD_5 and TSS effluent values shall not exceed 15% of influent values.

H. HISTORIC PERFORMANCE OF WWTP

The performance of the existing WWTP can be gauged in terms of consistently meeting effluent limits and removal percentages as prescribed by the SPDES permit. To examine historic WWTP performance, effluent BOD_5, TSS, and ultimate oxygen demand (UOD) concentrations and loadings are compared to permitted limits, as well as associated required minimum removal percentages. The UOD in the effluent wastewater is a function of the sampled BOD_5 and TKN values, i.e. UOD = 1.5 x BOD_5 Concentration + 4.5 x TKN Concentration.

Table III.5 below illustrates the WWTP effluent quality and removal rates with respect to BOD_5, TSS, UOD, pH, and Temperature.

Table III.5: Hume WWTP - Effluent Wastewater Quality

Parameter	Range (mg/l)						Removal (%)		
	Ave (mg/l)	Ave (lbs/day)	Min (mg/l)	Min (lbs/day)	Max (mg/l)	Max (lbs/day)	Ave	Min	Max
BOD_5	7	1.6	0	0	28	7.7	97	86	100
TSS	4	0.9	0	0	15	3.4	98	93	100
UOD	28.6	7.0	4.9	1.3	79.1	19.8	-		
pH (S.U.)	7.1		6.5		7.9		-		
Temp (°C)	14.2		7.0		22.0		-		

According to the sample data on the influent and effluent reviewed for this study the WWTP has been performing well. During January 2005-October 2007 all effluent limit parameters and removal rates were meet or exceeded. A copy of the DMRs along with a summary is included in Appendix B.

I. Shortcomings of WWTP and Collection System

Upon review of historic wastewater loadings, plant performance the Town of Hume WWTP is consistently providing quality effluent. We note that there a few shortcomings associated with the existing WWTP site. These shortcomings include the following:

1. Treatment Capability

As previously noted, the existing WWTP was designed as a grey water system, as such; this limits the future wastewater strength which can be conveyed to the WWTP. Any future wastewater will have to be primary treated wastewater.

2. Collection System

The existing collection system was also designed for a grey water system, i.e. small diameter interceptors which are not capable of transporting raw wastewater. In order to convey raw wastewater the existing interceptors would have to be replaced with 8-inch minimum pipes to be in compliance with recommended standards.

3. Limited/Restrictive Capacity of Key Wastewater Treatment Processes

The existing sand filters represent the bottle neck of the WWTP. The sand filters system current capacity will only allow for 6,000 gpd of additional flow.

4. WWTP Site

The configuration of the sand filters and piping in relation to the existing WWTP property will make expansion of the site for additional treatment capacity very difficult and most likely require land purchase.

5. Effluent Limits

Since the WWTP currently discharges to an intermittent stream, more stringent effluent limits are imposed. Most notably, the current SPDES permit requires a seasonal nitrogen limit.

J. PROJECTED WASTEWATER LOADINGS

1. Projected Growth within the Existing Service Area

Projected flow data in the Town of Hume with respect to sanitary sewer is not anticipated to increase significantly over the next 20 years based on existing population growth trends. According to 1990 census data the population of the Town was 1,970 and in the year 2000 the population was 1,987.

2. Future Potential Service Areas & Wastewater Streams

In the short term the hamlet of Hume is the most obvious service area expansion given its location and proximity to existing sewer infrastructure. It is estimated that the hamlet of Hume consists of 80 residential homes. If 250 gpd per home is assumed then the total average daily flow from the Hume would be approximately 20,000 gpd.

In the long term, with respect to other communities in the Town which are not served by sanitary sewers, Wiscoy and Rossburg which could be potential future service areas to be connected to a regional facility in the Town of Hume or Caneadea. It is estimated that the there are 60 residential homes that exist in these two communities which equates to approximately 15,000 gpd.

3. Projected Future Wastewater Flow & Quality

The projected future wastewater flow and quality are presented in Table III.6 below.

Table III.6: Hume - Projected Future Wastewater Flow and Quality

Service Area	Parameter			
	Ave Flow (gpd)	Peak Hourly* Flow (gpd)	BOD_5** (lbs/day)	TSS*** (lbs/day)
hamlet of Hume	20,000	80,000	26	30
hamlet of Wiscoy and Rossburg	15,000	60,000	19	23

* - Based on a Peak Hourly Flow to Average Daily Flow factor of 4

** - Based on 0.17 lbs/day per capita

*** - Based on 0.20 lbs/day per capita

Based on the flow data reviewed, the existing WWTP has the ability to except approximately 60 additional residential customers (6,000 gpd or 100 gpd per capita) while maintaining an average daily flow of 36,000 gpd, 80% of the WWTP SPDES permit limited flow capacity. As you can see the existing WWTP located in Hume does not have capacity to accommodate the hamlet of Hume without expanding.

K. Improvements to the Hume Wastewater Treatment Facility

1. Wastewater System Improvements to Accommodate Town of Hume's Current Service Area

Improvements to the Town of Hume's current service area should include a detailed inspection of all of the existing treatment components, including the existing septic tanks, pump station and sand filters. As these components are over 20 years old they should be replaced or rehabilitated based on the findings of the inspections.

The existing septic tanks should be inspected by trained personal during scheduled pump outs. The tanks should be inspected for cracks, water tightness and overall structural integrity.

The existing pump station should be inspected, for items such as the wet well integrity and water tightness, pump operation and efficiency, pump removal system condition, and pump control circuitry condition.

In addition each existing sand filter should be replaced at least once every 5 years. We note that a layer of geotextile fabric, as originally installed within the sand filters, should not be placed between the bottom of the filter sand and the under drain stone. This fabric can develop a slime layer and cause premature filter plugging.

2. Wastewater System Improvements to Accommodate Town of Hume's Current Service Area & Future Additional Service Areas

Wastewater system improvements to accommodate the Town of Hume's current service area and future additional service area will require expansion of the existing wastewater treatment system. As previously mentioned the existing WWTP does not have the capacity for the short term service area expansion which includes the hamlet of Hume. In order to accommodate the hamlet of Hume there would need to be an additional 4,000 square feet of sand filter area constructed this assumes that the existing WWTP would continue to operate a grey water type system with single pass intermittent sand filters. Based on the current size and configuration of the WWTP site it does not seam feasible to expand this facility beyond its current capacity.

L. EXISTING SEWER BUDGET

According to the 2007 sewer budget the Town is currently spending upwards of $131,000 annually. This figure includes existing debt service totaling $22,500

($15,000 principal and $7,500 interest). It is estimated that if the Town of Hume's WWTP were decommissioned it could leverage up to $45,000 towards capital improvements if portions of labor were re-allocated to other funds. The 2007 sewer budget and potential savings if the WWTP were decommissioned is presented in Table III.7 below.

Table III.7: Hume – Existing and Projected Sewer Budget

Expenses	2007 Budget	Projected Budget with No WWTP
Insurance and Contingencies	$9,000	$9,000
Transportation	$5,000	$5,000
Home and Community Service	$5,175	$5,175
Sanitary Sewers	$14,500	$16,000*
Treatment and Disposal	$60,400	$24,000**
Employee Benefits	$14,455	$4,337***
Debt Service	$22,500	$22,500
Totals	**$131,030**	**$86,012**

* - The sanitary sewer expense will increase due to increased energy costs associated with a larger horsepower pump station required for the conveyance to Caneadea

** - Operator salaries will be reduced if allocated to other funds

*** - Based on a 30% reduction

When the existing debt service is paid off the Town could recognize a total of $67,500 annually towards capital improvements. These funds could produce roughly $2,000,000 over a 30 year period for capital improvements to allocate towards consolidation with Caneadea.

We note that if the reduced sewer budget of $86,000 plus additional operation and maintenance costs incurred after consolidation with Caneadea is greater than the existing sewer budget of roughly $131,000 then no long term financial benefit will exist for the Town of Hume. Operation and maintenance costs for the Hume and Caneadea consolidation will be discussed further in Section IV.

IV. THE TOWN OF CANEADEA'S EXISTING WASTEWATER TREATMENT SYSTEM

A. SYSTEM DESCRIPTION AND TREATMENT CAPACITY

The Town of Caneadea's WWTP provides secondary treatment of the influent wastewater stream. The WWTP is located on the west side of the Genesee River. Refer to Figure III.1 for the location of the Town's WWTP. Refer to Figure IV.1 for a schematic plan showing the various components of the WWTP.

Raw wastewater is conveyed to the WWTP via a 10-inch diameter sanitary sewer. Wastewater first passes through a Kennison Nozzle for flow measurement. In turn, wastewater passes through a comminutor, prior to entering a grit chamber. After flow passes through the grit chamber, wastewater enters a wet well. Four raw influent sewage pumps are located on the lowest floor of the Control Building and have suction lines into the wet well chamber. In addition to pumping raw wastewater flows, these pumps also accommodate internal recycle flows.

In turn, a combination of raw sewage and recycle flows is pumped to a single rectangular clarifier measuring 15.5-feet wide by 53-feet long. Primary effluent, in turn, is conveyed via gravity to an adjacent single circular trickling filter unit for biological treatment. Primary effluent is distributed over the trickling filter via a self-propelled rotating arm. Although this trickling filter originally utilized rock media, that media was replaced with a plastic media with a high surface area to volume ratio.
Flow from the trickling filter is conveyed via gravity to a circular secondary clarifier with an inside diameter of 22-feet. The recycle flow line begins at the bottom of this clarifier and conveys recycle flows to the wet well chamber.

B. SLUDGE PROCESSING AND DISPOSAL

A primary anaerobic digester, in combination with a secondary anaerobic digester, is used to store and stabilize a mixture of primary and biological sludge. Methane, generated from the anaerobic digestion process, is utilized as a fuel source to heat the digester contents.

Digested liquid sludge is dewatered via paved drying beds. These drying beds have been roofed, to prevent precipitation from reaching the sludge to help maximize this dewatering/drying operation. Dewatered sludge is disposed at the Allegany County Landfill.

C. INDIVIDUAL CAPACITIES OF UNIT TREATMENT PROCESSES

The hydraulic and organic treatment capacities of the various wastewater treatment unit processes at the Town of Caneadea's existing WWTP are summarized in Table IV.1 and Table IV.2, respectively. As shown in Table IV.1, the overall hydraulic capacity of the existing WWTP is limited by the secondary clarifier. The organic treatment capacity in Table IV.2 is based on the trickling filter size and surface area/volume characteristics. Refer to Appendix C for background calculations.

Table IV.1: Caneadea WWTP - Summary Individual Hydraulic Capacities of Unit Treatment Processes

Unit Process	Estimated Capacity (gpd)	
	Peak Hourly	Average Daily
Influent Sewer	878,832	237,522
Kennison Nozzle	840,000	227,027
Comminutor	1,310,400	354,162
Grit Removal	733,824	198,331
Suction Lines	2,611,319	705,762
Wet Well	7,038,947	1,902,418
Raw Influent Pumping	700,080	189,211
Primary Settling	1,186,550	611,500
Primary Settling Weirs	910,000	245,945
Trickling Filter	-	280,000
Secondary Settling	456,120	123,275
Secondary Settling Weirs	1,240,000	335,135
Chlorine Contact Tank	419,904	113,487
Effluent Sewer	865,400	233,892

Table IV.2: Caneadea WWTP - Existing WWTP Organic Treatment Capacity - 1

Unit Process	Capacity (lb/BOD_5 /Day)
Trickling Filter	358

The capacities of the various unit sludge handling processes are presented in Table IV.3. These capacities are presented on the basis of the number of people which are able to be accommodated by the respective process.

Table IV.3: Caneadea WWTP - Existing WWTP Organic Treatment Capacity - 2

Unit Process	Estimated Capacity (Capita)
Anaerobic Digestion	5267
Paved Drying Beds	1727

D. Current Service Area and User Base

The current service area of the Town of Caneadea WWTP includes the hamlet of Houghton and Houghton College. The current user base consists of 350 EDUs (equivalent dwelling units).

E. Historic Wastewater Flows

The Town of Caneadea measures the flow at the WWTP via a 10-inch Kennison Nozzle located upstream of the comminutor and grit chamber. The flows recorded in the WWTP Discharge Monitoring Reports from January 2005 – October 2007 is summarized in Table IV.4 below.

Table IV.4: Caneadea WWTP - Summary of Historic Wastewater Flows

Year	Flow (mgd)	
	Average	Maximum
2005	0.1492	0.800
2006	0.1432	0.665
2007 (only through September)	0.1713	0.650

The existing WWTP flows appear to fluctuate during the snow melt and months with significant precipitation (wet season). Trends such as this typically indicate that inflow/infiltration is responsible for these flow variations measured at the WWTP. Based on the flow data reviewed, the wet season can increase the WWTP flow approximately 50,000 gpd, roughly 18% of the current permitted flow of 280,000 gpd.

F. Historic Wastewater Quality

The existing wastewater treated at the Town of Caneadea's WWTP is primarily comprised of domestic waste and limited commercial waste. A summary of

BOD_5 concentrations, total suspended (TSS) solids concentrations, pH, and temperature in the influent wastewater from January 2005-October 2007 is presented in Table IV.5 below.

Table IV.5: Caneadea WWTP - Influent Wastewater Quality

Parameter	Range		
	Average	**Minimum**	**Maximum**
BOD_5	235	56	680
TSS	264	116	1220
pH	7.7	7.5	8.0
Temp (°C)	15	11	19

In comparison to typical domestic wastewater the Town of Caneadea's WWTP average BOD_5 concentration is medium strength (typically 220 mg/l). The TSS concentrations are medium to strong in nature, typical medium to strong TSS concentrations range from 220 to 350 mg/l, respectively. The ph of the influent wastewater is generally neutral ranging between 7.5 and 8.0. The temperature of the influent wastewater fluctuates based on seasonal weather conditions with the lowest influent temperatures occurring in the winter months and the highest temperatures occurring in summer months.

G. CURRENT PERMITTED EFFLUENT LIMITS

The Town of Caneadea's existing WWTP currently operates under a State Pollutant Discharge Elimination System (SPDES) permit number NY0024431 regulated by the New York State Department of Environmental Conservation (NYSDEC). The SPDES permit effluent limits are summarized in Table IV.6 below.

Table IV.6: Caneadea WWTP - SPDES Permit No. NY0024431 Effluent Limits

Parameter	Limits	
Flow	280,000 gpd	
BOD_5	30 mg/l* 45 mg/l**	70 lbs/day* 105 lbs/day**
TSS	30 mg/l* 45 mg/l**	70 lbs/day* 105 lbs/day**
Fecal Coliform	200 No./100ml*	400 No./100ml**
pH	6.0 to 9.0	
Settleable Solids	0.3 ml/l	

* - 30 Day Arithmetic Mean

** - 7 Day Arithmetic Mean

In addition the BOD_5 and TSS effluent values shall not exceed 25% of influent values.

H. HISTORIC PERFORMANCE OF WWTP

The performance of the existing WWTP can be gauged in terms of consistently meeting effluent limits and removal percentages as prescribed by the SPDES permit. To examine historic WWTP performance, effluent BOD_5, and TSS, concentrations and loadings are compared to permitted limits, as well as associated required minimum removal percentages.

Table IV.7 below illustrates the WWTP effluent quality and removal rates with respect to BOD_5, TSS, pH, and Temperature.

Table IV.7: Caneadea WWTP - Effluent Wastewater Quality

Parameter	Range (mg/l)						Removal (%)		
	Ave (mg/l)	Ave (lbs/day)	Min (mg/l)	Min (lbs/day)	Max (mg/l)	Max (lbs/day)	Ave	Min	Max
BOD_5	14	20	6	4	25	61	93	79	99
TSS	10	14	5	3	24	49	95	80	99
pH (S.U.)	7.6		7.8		8.0		-		
Temp (°C)	15		8		20		-		

According to the sample data on the influent and effluent reviewed for this study the WWTP has been performing well. During January 2005-October 2007 all effluent limit parameters and removal rates were meet or exceeded. A copy of the DMRs along with a summary is included in Appendix D.

I. SHORTCOMINGS OF WWTP

Upon review of historic wastewater loadings, plant performance the Town of Caneadea WWTP is consistently providing quality effluent. We note that there a few shortcomings associated with the existing WWTP. These shortcomings include the following:

1. Recommended Standards Compliance

The WWTP currently does not comply with recommend standards relative to primary and secondary settling treatment units. Recommended standards indicate that multiple units capable of independent operation shall be provided in all WWTPs where the design average flows exceed 100,000 gpd. In addition WWTPs not having multiple units shall include other provisions to assure continuity of treatment.

2. Limited/Restrictive Capacity of Key Wastewater Treatment Processes

Various unit processes have limited capacities which act to restrict the overall WWTP capacity. Most notably, the capacities of the grit removal system, influent pumping system, secondary clarifier, and chlorine contact tank act to limit the overall WWTP capacity and represent a bottleneck in the overall wastewater treatment process.

3. Single Treatment Train

The current WWTP consists only of a single treatment train, as opposed to duel trains, which are commonly utilized. Accordingly, when a treatment unit (for example, a clarifier or trickling filter) is taken off line, partially treated wastewater would be discharged to the Genesee River.

4. Existing WWTP Site

The existing WWTP is located on the western shore of the Genesee River. This portion of the Genesee River due to a sharp curve has been prone to erosion to occur on the western bank. The erosion appears to be slowly eating away at the WWTP site. In the past, attempts have been made to stabilize the bank, however erosion of the bank continues.

5. Aging Treatment Equipment

Much of the original equipment from the late 1960s WWTP installation is still in use and is reaching the end of its intended design life. This equipment includes clarifier mechanisms, weirs, the trickling filter flow distributor mechanism, digester internal equipment and covers, and comminutor. Through routine maintenance efforts over the years, the Town has kept this equipment in working condition.

J. TOWN OF CANEADEA'S PROJECTED WASTEWATER LOADINGS

1. Projected Growth within the Existing Service Area

Based on Census data from 1990 – 2006 the Town of Caneadea population has increased from 2,551 to 2,746, or 7.1% in the last 16 years. Since the growth is specific to the Town of Caneadea it is difficult to quantify if this growth occurred in Houghton, which is the only service area currently connected to the WWTP. For the purposes of this study it will be assumed that growth in Houghton will remain constant.

The Town is in the design phase of building a new water treatment facility as necessary for iron/manganese/arsenic removal. The proposed removal process will generate an average of 49,000 gpd and a maximum of 81,740 gpd of filtrate which is currently planned to discharge to the WWTP.

2. Future Potential Service Areas & Wastewater Streams

There are three communities located the Town of Caneadea which are currently not served by sanitary sewers; Caneadea, Oramel and Rushford. Connection of these two communities should be considered long term goals at this point as the infrastructure required to convey sanitary sewer to a regional facility located in Hume or Caneadea would be very costly with minimal user cost shares. It is estimated that 120 residents exist between the two communities.

3. Projected Future Wastewater Loadings & Quality

The projected future wastewater flow and quality are presented in Table V.2 below.

Table IV.8: Hume - Projected Future Wastewater Flow and Quality

Service Area	Parameter			
	Ave Flow (gpd)	Peak Hourly* Flow (gpd)	BOD_5** (lbs/day)	TSS*** (lbs/day)
Caneadea Water Treatment Plant	49,000	-	-	-
hamlet of Caneadea, Oramel and Rushford	30,000	70,000	51	60

* - Based on a Peak Hourly Flow to Average Daily Flow factor of 4

** - Based on 0.17 lbs/day per capita

*** - Based on 0.20 lbs/day per capita

V. INTERCONNECTION EVALUATION

A. MIXED HUME/CANEADEA WASTEWATER STREAM

The mixed Hume/Caneadea wastewater stream would be a combination of primary treated wastewater (from Fillmore) and raw wastewater (from Houghton). Table V.1 below illustrates the combined wastewater flows and quality expected from the combined wastewater streams.

Table V.1: Mixed Hume/Caneadea Wastewater Streams

Service Area	Parameter			
	Ave Flow* (mgd)	Peak Hourly** Flow (mgd)	BOD_5*** (lbs/day)	TSS*** (lbs/day)
Town of Caneadea	0.204	0.609	304	341
Town of Hume	0.030	0.102	24	12
Totals	**0.234**	**0.711**	**328**	**353**

* - Based on average monthly flows from Jan. 2005-Oct. 2007. The Town of Caneadea average flow includes 49,000 gpd (average) from the Caneadea Water Treatment Plant

** - Based on a Peak Hourly Flow to Average Daily Flow factor of 3.4 according to the combined population between Hume and Caneadea. The Town of Caneadea Peak Hourly Flow is calculated as follows: (0.155 mgd x 3.4) + (81,740 gpd (maximum)/1,000,000) from the Town of Caneadea Water Treatment Plant.

*** - Based on average monthly influent values from Jan. 2005 – Oct 2007

B. IMPROVEMENTS REQUIRED TO ACCOMMODATE COMBINED FLOWS

Based on the shortcomings associated with the existing Town of Hume WWTP discovered in this study, the alternative associated with conveying the Town of Caneadea wastewater to the Town of Hume WWTP will be eliminated from further discussion.

If we explore the option of conveying the Town of Hume's existing primary treated wastewater to the Town of Caneadea WWTP the following improvements necessary for the conveyance to Caneadea would need to take place:

- Town of Hume pump station upgrade
- New Forcemain from the existing pump station to the Town of Caneadea WWTP
- Caneadea WWTP Improvements

1. Hume Pump Station

Since Hume already has a pump station, which receives all the wastewater flow generated by Hume, it would be cost effective to use the existing site for a new or rehabilitated pump station. It is recommended that the existing Hume pump station be replaced or rehabilitated as it is over 20 years old and the pump design conditions will be altered when transporting wastewater to Caneadea. In addition, a flow meter structure should be installed to monitor flows from the Town of Hume.

2. Forcemain Route

The most feasible forcemain route would be to travel south along Rte 19A until it reaches Rte 19. The forcemain would then continue along Rte 19 until it reaches an upstream manhole of the Caneadea WWTP. The actual placement of the forcemain would need to be looked at in more detail as the west side currently includes a water main and the Greenway Trail is on the east side. Refer to Figure V.1 for an illustration of the conveyance improvements required.

The conceptual costs for the interconnection between the Town and Hume and the Caneadea WWTP is summarized in the following Table V.2:

Table V.2: Interconnection Evaluation – Hume to Caneadea Conveyance Project Cost Estimate

Description	Quantity	Units	Cost/Unit	Total Cost
Abandon existing pump station	1	LS	$10,000	$10,000
Decommission existing WWTP	1	LS	$20,000	$20,000
New pump station wetwell	1	LS	$20,000	$20,000
New pumps, controls, and valves	1	LS	$150,000	$150,000
Meter pit and equipment	1	LS	$20,000	$20,000
4-inch forcemain	20,000	LF	$60	$1,200,000
Air/Vacuum release valve manholes (assumed)	5	EA	$8,000	$40,000
Stream Crossing	1	LS	$15,000	$15,000
Restoration	1	LS	$25,000	$25,000
Electric and controls	1	LS	$17,000	$17,000
Mobilization (3%)	1	LS	$45,000	$45,000
Maintenance & Protection of Traffic (3%)	1	LS	$45,000	$45,000
Construction Contingency (15%)	1	LS	$241,050	$241,050
Construction Subtotal				**$1,848,050**
Engineering, inspection, and bidding (25%)				$401,750
Soil borings/geotechnical evaluation				$10,000
Admin./Financial (3%)				$48,210
Legal (3%)				$48,210
Total Project Cost				**$2,356,220**

3. Caneadea WWTP Improvements

In order to accommodate the existing Town of Hume wastewater flows combined with the proposed flows from the new water treatment plant improvements to the following components at existing WWTP will be necessary:

- Influent pumping system
- Secondary clarifier

- Chlorine contact tank expansion (we note that disinfection is currently not required in the SPDES permit, hence improvements to the chlorine contact tank will not be included in the WWTP improvement alternatives)

Many alternatives exist for upgrading the Town of Caneadea WWTP as necessary to accommodate the future flow proposed from the Caneadea WTP and the Town of Hume wastewater flows. For the purposes of this study there will be four upgrade alternatives discussed. Each upgrade will recognize the following flow and organic loading capacities:

Average Daily Flow = 234,000 gpd
Peak hourly flow = 711,000 gpd
Average Daily Organic Loading = 328 lbs BOD_5/day

Two different biological wastewater treatment processes are evaluated in this study, trickling filters and sequencing batch reactors (SBRs). A general description of each is as follows.

Trickling Filters

A trickling filter consists of a bed of highly porous media to which wastewater is applied in a trickle fashion. On the surface of the media, a biofilm develops which effects the treatment of the wastewater. Trickling filters are a member of the fixed film family of processes. A variety of media is utilized, including rock and plastic media. Presently, Caneadea utilizes a trickling filter with plastic media at their existing WWTP. Refer to Figure V.2 for a schematic of a trickling filter.

Sequencing Batch Reactors

Sequencing batch reactors (SBRs) are a member of the activated sludge family of biological treatment processes. SBRs are unique from other activated sludge processes in that biological treatment (with aeration) and sedimentation occur in a single basin. With SBRs, wastewater is treated by means of the fill and draw principle. For purposes of this study, a variation of the pure SBR concept, known as the Intermittent Activated Sludge Extended Aeration System (ICEAS), is evaluated. Unlike the pure SBR cycle, the ICEAS process allows for continuous discharge of wastewater into the reactor basins throughout all phases of the treatment cycle. Refer to Figure V.3 for a schematic of the SBR process.

a. Alternative A1: This alternative is consistent with the original biological treatment process utilizing fixed film or trickling filters currently used at the WWTP. We note that this alternative will not be in compliance with the current recommended standards; however offers a solution to accommodate the hydraulic and organic capacities outlined above. The conceptual improvement components, associated with Alternative A1, include the following. Refer to Figure V.4 for a schematic plan of this alternative.

<u>Wastewater Treatment Process Improvements</u>

- New influent pumps
- Upgrade existing primary clarifier equipment
- New distributor arm and media for existing trickling filter
- New trickling filter recycle system (existing secondary clarifier to be converted to a recycle pumping station)
- New secondary clarifier

<u>Sludge Treatment Process Improvements</u>

- New anaerobic digester covers and internal equipment
- New sludge feed pump

A conceptual cost estimate for Alternate A1 is summarized in Table V.3 below:

Table V.3: Estimated Costs of Caneadea WWTP Alternative A1

Description	Quantity	Units	Cost/Unit	Total Cost
Pavement	820	SY	$50	$41,000
Process piping and valving	1	LS	$25,000	$25,000
Fill	1	LS	$10,000	$10,000
Restoration	1	LS	$10,000	$10,000
Raw influent pumps	1	LS	$120,000	$120,000
Upgrade existing primary clarifier equipment	1	LS	$100,000	$100,000
Upgrade existing trickling filter	1	LS	$173,000	$173,000
Trickling filter recirc PS – convert secondary clarifier	1	LS	$150,000	$150,000
New secondary clarifier	1	LS	$162,000	$162,000
New sludge feed pump	1	LS	$50,000	$50,000
New anaerobic digester internal equipment	2	EA	$50,000	$100,000
New anaerobic digester covers	2	EA	$225,000	$450,000
Electric and Controls	1	LS	$49,300	$49,300
Improvements to existing control building	1	LS	$25,000	$25,000
Misc. valve replacement	1	LS	$20,000	$20,000
Mobilization (3%)	1	LS	$44,559	$44,559
Construction Contingency (15%)	1	LS	$229,479	$229,479
Construction Subtotal				**$1,759,338**
Engineering, inspection, and bidding (25%)				$382,465
Soil borings/geotechnical evaluation				$5,000
Admin./Financial (3%)				$45,896
Legal (3%)				$45,896
Total Project Cost				**$2,238,594**

b. Alternative A2: This alternative is consistent with the original biological treatment process utilizing fixed film or trickling filters currently used at the WWTP. This alternative will bring the existing WWTP in compliance with the current recommended standards, i.e. parallel treatment trains. The conceptual improvement components, associated with Alternative A2, include the following. Refer to Figure V.5 for a schematic plan of this alternative.

Wastewater Treatment Process Improvements

- New headworks facility (screening, grit removal, flow metering)
- New primary clarifiers
- New trickling filter unit in parallel with the existing trickling filter
- New distributor arm and media for existing trickling filter
- New influent pumps
- New trickling filter recycle system (existing secondary clarifier to be converted to a recycle pumping station)
- New secondary clarifiers

Sludge Treatment Process Improvements

- New anaerobic digester covers and internal equipment
- New sludge pump

A conceptual cost estimate for Alternate A2 is summarized in Table V.4 below:

Table V.4: Estimated Costs of Caneadea WWTP Alternative A2

Description	Quantity	Units	Cost/Unit	Total Cost
Pavement removal	820	SY	$4	$3,280
Pavement	820	SY	$50	$41,000
Process piping and valving	1	LS	$50,000	$50,000
Chainlink fence	400	LF	$70	$28,000
Fill	1	LS	$20,000	$20,000
Restoration	1	LS	$25,000	$25,000
Headworks facility	1	LS	$500,000	$500,000
Raw influent pumps	1	LS	$120,000	$120,000
New primary clarifier	2	EA	$150,000	$300,000
Upgrade existing trickling filter	1	LS	$173,000	$173,000
New trickling filter	1	LS	$230,000	$230,000
Trickling filter recirc PS – convert secondary clarifier	1	LS	$150,000	$150,000
New secondary clarifier	2	EA	$162,000	$324,000
Splitter boxes	2	EA	$15,000	$30,000
Convert exist. primary clarifier to a sludge thickener	1	LS	$100,000	$100,000
New sludge feed pump	1	LS	$50,000	$50,000
New anaerobic digester internal equipment	2	EA	50,000	$100,000
New anaerobic digester covers	2	EA	$225,000	$450,000
Electric and Controls	1	LS	$177,400	$177,400
HVAC	1	LS	$32,500	$$32,500
Plumbing	1	LS	$17,500	$17,500
Improvements to existing control building	1	LS	$25,000	$25,000
Misc. valve replacement	1	LS	$20,000	$20,000
Mobilization (3%)	1	LS	$89,000	$89,000
Construction Contingency (15%)	1	LS	$458,352	$458,352
			Construction Subtotal	**$3,514,032**
			Engineering, inspection, and bidding (25%)	$763,920
			Soil borings/geotechnical evaluation	$10,000
			Admin./Financial (3%)	$91,760
			Legal (3%)	$91,760
			Total Project Cost	**$4,471,293**

c. Alternative A3: This alternative explores the option of converting the existing fixed film WWTP to an activated sludge WWTP which utilizes an SBR. The conceptual improvement components, associated with Alternative A3, include the following. Refer to Figure V.6 for a schematic plan of this alternative.

Wastewater Treatment Process Improvements

- New headworks facility (screening, grit removal, flow metering)
- New influent pumps
- New SBRs
- Covert existing primary clarifier to a sludge thickener
- Abandon existing trickling filter
- Abandon existing secondary clarifier

Sludge Treatment Process Improvements

- New anaerobic digester covers and internal equipment
- New sludge pumps

A conceptual cost estimate for Alternate A3 is summarized in Table V.5 below:

Table V.5: Estimated Costs of Caneadea WWTP Alternative A3

Description	Quantity	Units	Cost/Unit	Total Cost
Pavement removal	820	SY	$4	$3,280
Existing secondary clarifier removal	1	LS	$50,000	$50,000
Existing trickling filter removal	1	LS	$45,000	$45,000
Existing chlorine contact tank removal	1	LS	$35,000	$35,000
Pavement	820	SY	$50	$41,000
Process piping and valving	1	LS	$50,000	$50,000
Effluent sewer	300	LF	$70	$21,000
Chainlink fence	400	LF	$70	$28,000
Fill	1	LS	$20,000	$20,000
Restoration	1	LS	$25,000	$25,000
Headworks facility	1	LS	$500,000	$500,000
Raw influent pumps	1	LS	$120,000	$120,000
Sequencing batch reactor	1	LS	$900,000	$900,000
Convert exist. primary clarifier to a sludge thickener	1	LS	$100,000	$100,000
New sludge feed pump	1	LS	$50,000	$50,000
New anaerobic digester internal equipment	2	EA	50,000	$100,000
New anaerobic digester covers	2	EA	$225,000	$450,000
Electric and Controls	1	LS	$167,000	$167,000
HVAC	1	LS	$25,000	$25,000
Plumbing	1	LS	$16,750	$16,750
Improvements to existing control building	1	LS	$25,000	$25,000
Misc. valve replacement	1	LS	$20,000	$20,000
Mobilization (3%)	1	LS	$83,761	$83,761
Construction Contingency (15%)	1	LS	$431,769	$431,769
Construction Subtotal				**$3,307,160**
Engineering, inspection, and bidding (25%)				$693,008
Soil borings/geotechnical evaluation				$10,000
Admin./Financial (3%)				$83,161
Legal (3%)				$83,161
Total Project Cost				**$4,176,489**

d. Alternative A4 – This alternative is similar to A3 with regard to the biological treatment process, only Alternative A4 investigates a relocated WWTP. This alternative will be more costly than Alternative A3, however given the shortcomings of the existing site with regard to size and erosion it may be more favorable. Refer to Figure V.7 for conceptual locations for a new WWTP facility. The conceptual improvement components, associated with Alternative A4, include the following. Refer to Figure V.8 for a schematic plan of this alternative.

<u>Wastewater Treatment Process Improvements</u>

- New headworks facility (screening, grit removal, flow metering)
- Convert existing influent pumps to a pump station
- New sequencing batch reactors (SBRs)
- Decommission the existing WWTP

<u>Sludge Treatment Process</u>

- New aerobic digesters
- New sludge dewatering facility
- New sludge storage area

A conceptual cost estimate for Alternate A4 is summarized in Table V.6 below:

Table V.6: Estimated Costs of Caneadea WWTP Alternative A4

Description	Quantity	Units	Cost/Unit	Total Cost
Decommission existing WWTP	1	LS	$50,000	$50,000
Forcemain (from Caneadea WWTP to New Site)	8,800	LF	$60	$528,000
Upgrade exist. influent pumps to convey to new WWTP	1	LS	$50,000	$50,000
Effluent sewer	400	LF	$80	$32,000
Greenway trail boring for forcemain	50	LF	$200	$10,000
Highway boring for forcemain	60	LF	$200	$12,000
Highway boring for effluent sewer	60	LF	$300	$18,000
Greenway trail boring for effluent sewer	50	LF	$300	$15,000
Headwall for effluent discharge	1	LS	$45,000	$45,000
Driveway	1	LS	$20,000	$20,000
Restoration (forcemain installation)	1	LS	$10,000	$10,000
Pavement	850	SY	$50	$42,500
Gates	1	EA	$4,000	$4,000
Process piping and valving	1	LS	$100,000	$100,000
Chainlink fence	1000	LF	$70	$70,000
Fill	1	LS	$20,000	$20,000
Restoration (WWTP Site)	1	LS	$45,000	$45,000
Headworks facility	1	LS	$500,000	$500,000
SBRs with attached aerobic digesters	1	LS	$1,100,000	$1,100,000
Sludge dewatering/blower facility	1	LS	$400,000	$400,000
Electric and Controls	1	LS	$200,000	$200,000
HVAC	1	LS	$45,000	$45,000
Plumbing	1	LS	$22,500	$22,500
Mobilization (3%)	1	LS	$67,050	$67,050
Maintenance and Protection of Traffic	1	LS	$19,950	$19,950
Construction Contingency (15%)	1	LS	$513,900	$513,900
Construction Subtotal				**$3,939,900**
Engineering, inspection, and bidding (25%)				$834,750
Property purchase (assumed)				$20,000
Soil borings/geotechnical evaluation				$20,000

Admin./Financial (3%)	$100,170
Legal (3%)	$100,170
Total Project Cost	**$5,014,990**

C. OVERALL PROJECT COST ESTIMATES

The conceptual cost for the interconnection between Hume and Caneadea is presented in Table V.7 below.

Table V.7: Summary of Conceptual Costs of the Hume-Caneadea Interconnection and Caneadea WWTP Improvement Concepts

Caneadea WWTP Improvement Concept	WWTP Improvement Cost	Hume to Caneadea Interconnection Cost
Alternative A1	$2,238,594	$2,356,220
Alternative A2	$4,471,293	$2,356,220
Alternative A3	$4,176,489	$2,356,220
Alternative A4	$5,014,990	$1,527,349

As shown in Table 19, the following points are apparent.

- To accommodate the flows from the current service area including the proposed Town of Caneadea water treatment filtrate along with the Town of Hume, Improvement Alternative A1 has the lowest estimated project cost, although some inherent issues exist with that alternative. Notably, this layout consists of a single wastewater treatment train. As such, when the existing trickling filter or primary/secondary clarifiers are taken off line for upgrades or regular scheduled maintenance activities, raw or partially treated wastewater would be discharged to the Genesee River.

- Alternative A3 which utilized SBRs has the lowest estimated project costs, when compared to Alternative A2, a duel treatment train layout.

- Based on the new shared WWTP site selected in this study, the Town of Hume can recognize a significant savings due to a shorter forcemain required.

VI. Cost Analysis

A. EDU Rate Structures for each Alternative

As previously noted the Town of Caneadea and the Town of Hume currently occupy 350 and 233 EDUs, respectively. If we assume that by negotiation the Towns agree to split the cost of the improvements in Caneadea by their percentages of final flow, the Town of Hume would contribute 13% and Town of Caneadea would contribute 87% for costs associated with WWTP improvements.

B. Operational Cost for each Alternative

In addition to the costs associated with each alternative described above there are cost for operating the WWTP and collection system. It is assumed that the Town of Hume and Caneadea will each cover 50% of the Operation and Maintenance costs associated with each alternative. The annual operation cost includes the following appropriations:

- Operator salaries and benefits
- Administration
- Estimated Maintenance Costs
- WWTP electric usage ($0.12/kilowatt)
- Town of Hume Pump Station Operation and Maintenance Costs
- Town of Caneadea Collection System Costs
- Sludge Disposal Costs

Table VI.1: Operation and Maintenance Costs for each Alternative

Improvement Alternative	Total Cost
Alternative A1	$170,796
Alternative A2	$171,293
Alternative A3	$172,567
Alternative A4	$186,325

As shown Table VI.1 Alternate 1, 2, and 3 all maintain very similar annual operation and maintenance costs. Alternate 4 operation and maintenance costs are higher due to the use of aerobic digester which require aeration resulting in increased energy costs.

The cost per EDU for Hume and Caneadea associated with each of the Alternatives including annual operation and maintenance costs is summarized in the Table VI.2 and Table VI.3 below. The annual debt service for capital improvements is based on a 30 year bond and a 5% interest rate.

Table VI.2: Hume EDU Structures for each Alternative

Improvement Alternative	Total Cost	Annual Debt Service Including O & M	Annual Cost Per EDU
Alternative A1	$2,643,219	$257,343	$1,104
Alternative A2	$2,929,463	$276,212	$1,185
Alternative A3	$2,891,667	$274,391	$1,178
Alternative A4	$2,170,297	$234,344	$1,006

Table VI.3: Caneadea EDU Structures for each Alternative

Improvement Alternative	Total Cost	Annual Debt Service Including O & M	Annual Cost Per EDU
Alternative A1	$1,951,595	$212,352	$607
Alternative A2	$3,898,051	$339,220	$969
Alternative A3	$3,641,042	$323,139	$923
Alternative A4	$4,372,043	$377,570	$1,079

C. NET FINANCIAL RESULT FOR EACH ALTERNATIVE

The Town of Hume current sewer district costs are currently $540 per EDU annually. The Town of Caneadea sewer district costs are currently $330. It is very apparent based on current sewer district costs that neither community will benefit financially from any of the alternatives without significant funding aid.

The Town of Hume will not experience long term financial benefits based on the on the estimated cost of operation and maintenance. If the Town of Hume decommissioned its WWTP the projected sewer budget would reduce from \$131,000 to \$86,000. If 50% of the annual operation and maintenance cost of the lowest cost alternative associated with the consolidating the Towns is added to this figure it would result in an annual sewer budget of \$171,500, approximately \$40,500 more than the 2007 budget.

VII. HAMLET OF HUME SERVICE AREA STUDY

A. COLLECTION SYSTEM ALTERNATIVES

As discussed earlier, the Town of Hume has three specific areas of medium density development where new sewer service is desired. The hamlets of Hume, Wiscoy & Rossburg each can be characterized as having underperforming individual treatment systems. Understanding that the cost of a trunk sewer / forcemain conveyance option from Hume to Caneadea is upwards of \$2M, the conveyance of wastewater from a common point serving the hamlets of Wiscoy and Rossburg to the hamlet of Fillmore could be similarly estimated.

Within the hamlets, varying factors will impact the selection of preferred collection system designs. One option is to expand the collection system to the original specifications of the greywater system in the hamlet of Fillmore. This would ensure uniformity between all service areas from an operations and maintenance perspective.

However, there are inherent flaws with greywater collection systems that were noted above which would limit the opportunity for the hamlets to accommodate any industrial or commercial growth and will be a long-term hindrance on development. The build-up of sulfuric acids in the effluent stream can become a significant issue to the stability of the concrete manholes in the collection system, resulting in long-term maintenance costs that exceed that of a conventional system.

Other collection systems have been developed to address the unique characteristics of small systems and each has their benefits and drawbacks. The depth to which construction can be done affordably will dictate the number of lift stations required and whether or not a full gravity fed system can be constructed.

To construct a conventional gravity sewer system within each hamlet would be a good long-term consideration; however this method will not be feasible when one takes into account the limitations of the existing WWTP facility which is designed to accept effluent only.

If a joint WWTP facility with Caneadea is provided in the future, then a conventional system should be constructed so long as conveyance within the collection system to the Fillmore pump station meets minimum grade and pipe diameters for conventional sewer systems.

B. Conveyance to Fillmore Collection System

1. Sewer Alignment

The Hamlet of Hume is split geographically in the middle by a deep gorge of Cold Creek and is reported to have high bedrock across the area. Gravity sewers may reach a depth of 12 to 15 feet along portions of the Claybed Road. Bennett Street is a short loop on the north side of Claybed which circles around the Hume Town Museum and is the lowest elevation on the west side of Cold Creek. It is suggested that a pump station be constructed at the north end of the Bennett Loop with a small diameter forcemain routed up to the County Road 23 bridge, then across Cold Creek and into a manhole on the east side of the creek.

The total estimated length of gravity sewer on the west side of Cold Creek is 4500 lineal feet serving approximately forty-five (45) parcels. The forcemain length would be approximately 850 feet.

On the west side of Cold Creek, conventional gravity sewers should be able to handle all flows without the need for individual lift stations and depths of 10-15 feet are anticipated. Again, the bedrock conditions will likely cause some difficulty at these depths and may result in the need to consider lift stations to eliminate conflicts.

On the east side of Cold Creek, the estimated length of gravity sewer would be 3500 lineal feet. The sewer in this area is estimated to add approximately thirty-five (35) parcels to the total service area in the hamlet.

From the common point of collection at the former intersection of Liberty Street and Route 19, an additional length of approximately 3-4,000 lineal feet is required to tie into the existing Fillmore system.

2. Upgrades within the Fillmore Collection System

There is approximately 4,500 lineal feet of collection system between the anticipated point of connection between Hume and the Fillmore collection system and further study would be needed to assess the impact of adding approximately 80 units into the Fillmore system at Rte 19. At an estimated 250 gallons per day per unit, it is approximated that 20,000 gallon average daily flow would be generated. It should be noted that the average daily flow per unit in the existing Fillmore is approximately 130 gallons per day, so these estimates are conservative.

C. OVERALL PROJECT COST ESTIMATE

The anticipated cost to construct a complete collection system in Hume is $2.0 million, or roughly $1,630 per unit in construction cost alone. Additional costs would be allocated to the District for its share in the cost of operation and maintenance for the collection system, biennial pumping of septic tanks, and WWTP.

The NYS Audit & Control threshold for sewer districts constructed in 2008 is $667 per unit, the maximum that a District could be established without special consideration by the State Comptroller's office. At this level, the project would require funding assistance in the amount of over $1.2 million.

In terms of consolidation, the share of operation and maintenance cost would be estimated as a 25% / 75% split between the new service area and existing with the additional cost of pumping septic tanks added to the budget. That being understood, it should be expected that no cost savings would be available to the parent district by expanding the size of the service area.

There remains some existing debt on the WWTP and collection system of roughly $22,500 annually. The parent district would expect to receive connection charges to apply against current debt. Otherwise, there are no distinct benefits to the parent district from a consolidation of services.

VIII. SUMMARY

The key findings can be summarized as follows:

- The Town of Hume has operated the Fillmore sewer collection system and effluent sewer treatment plant following dissolution of the Village of Fillmore in the 1990s. The sewer district serving Fillmore is the only public sewer system in the Town.

- There are three other medium density hamlets in the Town of Hume where public sewer service is desired. Within 1 mile of Fillmore is the hamlet of Hume and it may be economically feasible to serve this hamlet by conveyance to the Fillmore collection system. Within 3 ½ miles of Fillmore are the hamlets of Wiscoy and Rossburg, which are less than 1 mile apart from each other. It is not economically feasible to provide public sewer serve Wiscoy and Rossburg at this time.

- Hume has recognized limitations to its growth due in part to having grey water sewer treatment system. Caneadea is located 3 ½ miles away and has a traditional sewer collection and treatment system. Consolidation of the treatment works could provide Hume with an option to upgrade to a full system in the future.

- A pump station / forcemain project from Hume to Caneadea will not provide a positive cost benefit. Significant subsidies would be required to achieve a zero cost impact.

- The Hume WWTP will require extensive modifications and improvements to reduce operating costs and to accommodate future flows. It appears that on a per unit basis, these improvements may be quite costly and possibly cost prohibitive.

IX. RECOMMENDATIONS

Any move to consolidate the sanitary sewer systems of Hume and Caneadea will be met with significant fiscal challenges. If the Towns choose to pursue consolidation, the move should be planned to coincide with a significant expansion of service area or of plant capacity to share costs of development over a larger area and with subsidies from federal or state sources. As this report determined, however, even the anticipation of plant expansion in Caneadea and future expansion of service area does not appear to offer favorable economics without significant subsidies.

It is our recommendation that each town continue to serve their interests and to not pursue consolidation of the sanitary sewer systems at this time.

X. REFERENCES

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2. Federal Emergency Management Agency, Flood Insurance Study, Town of Caneadea, Allegany County, Effective Date August 20, 1982

3. Metcalf and Eddy, Inc., WASTEWATER ENGINEERING: Treatment, Disposal, and Reuse, Third Edition

4. United States Geologic Survey, Caneadea and Hume, NY Quadrangles

5. Wastewater Facility Operational Records for the Town of Hume's Wastewater Treatment Plant, SPDES Permit No. NY0203858

6. Wastewater Facility Operational Records for the Town of Caneadea's Wastewater Treatment Plant, SPDES Permit No. NY0024431

7. Great Lakes-Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers, Recommended Standards for Wastewater Facilities, 2004 Edition

8. NYSDEC, Design Standards for Wastewater Treatment Works, 1988

9. Fagan Engineers – Engineering drawings for the Town of Hume's Wastewater Treatment Facility, circa 1987

10. Fagan Engineers –Town of Hume's Wastewater Treatment Facility Operation and Maintenance Manual, September 1987

11. Woodward Engineers – Engineering drawings for the Town of Caneadea's Wastewater Treatment Facility, circa 1967

APPENDIX A

TOWN OF HUME WWTP – ESTIMATION OF INDIVIDUAL CAPACITIES OF UNIT TREATMENT PROCESSES

APPENDIX A

ESTIMATION OF INDIVIDUAL CAPACITIES
OF
UNIT TREATMENT PROCESSES

The purpose of the following calculations is to develop an estimate of the hydraulic and organic capacities of the unit processes utilized within the Town of Hume existing WWTP. The Great Lakes – Upper Mississippi River Board of State Public Health and Environmental Manager's Recommended Standards for Wastewater Facilities, 2004 Edition, (Ten States' Standards) is utilized as the basis for the estimation of these various unit capacities. Other reference sources, including equipment manufacturer's literature, are also used and noted accordingly.

WASTEWATER TREATMENT & CONVEYANCE PROCESSES

I. RESIDENTIAL/COMMERCIAL SEPTIC TANKS

The existing wastewater collection system in the Village of Fillmore consists of individual septic systems at each residential or commercial facility. To determine the septic capacities for this exercise a three bedroom house will be assumed. According to the Design Standards for Wastewater Treatment Works , NYSDEC Publication, 1988 the average daily flow for a three bedroom house is 400 gpd. Therefore based on the previously mentioned Design Standards the septic tank capacity can be calculated as follows:

> For a daily flow under 5000 gpd the septic tank size shall be 1.5 times the daily flow or 600 gallons. The Design Standard also states that no septic tanks shall be less than 1,000 gallons.

According to the Village of Fillmore WWTP Operation and Maintenance Manual, prepared in February 1987 by Fagan Engineers the residential and commercial facilities were provided with at least 1,000 gallon capacity septic tanks.

II. INTERCEPTOR SEWER

Manning's Equation is utilized to calculate the hydraulic capacity of the existing 6-inch diameter interceptor immediately before the pump station. This sewer is polyvinyl chloride (PVC) and at an assumed minimum slope of 0.5 percent. Manning's Equation is as follows.

$$Q = 1.49/n \times A \times R^{2/3} \times S^{1/2}$$

Q	- Flow rate (cfs)
n	- Roughness coefficient
A	- Cross-sectional area of sewer (sq. ft.)
R	- Hydraulic radius of sewer (ft.)

S - Longitudinal sewer slope (ft./ft.)

$Q = 1.49/n \times A \times R^{2/3} \times S^{1/2}$

$Q = 1.49/0.013 \times \pi \times (3/12)^2 \times (3/(2(12)))^{2/3} \times (0.005)^{1/2}$

$Q = 0.398$ CFS = 178.6 GPM = 257,184 GPD

Peak Hourly Capacity = 257,184 GPD

Utilizing a Peak Hourly/Design Average factor of 4 the average daily capacity can be computed as follows:

Average Daily Capacity = 257,184 GPD/4.0 = 64,296 GPD

III. PUMP STATION

According to the Village of Fillmore WWTP Operation and Maintenance prepared by Fagan Engineers in February 1987 the pumps were designed to operate at 81 gpm at 88 feet of total dynamic head. This flow rate will be used in computing the peak hourly and average daily flows as shown below:

Peak Hourly Capacity = 81 gpm x 24 hours/day x 60 minutes/hour = 116,640

Utilizing a Peak Hourly/Design Average factor of 4 the average daily capacity can be computed as follows:

Average Daily Capacity = 116,640 GPD/4.0 = 29,160 GPD

IV. FORCEMAIN

A sanitary forcemains capacity is a function of the velocity times the cross-sectional area flowing through the pipe. The maximum recommended velocity for a sanitary forcemain for this exercise is assumed to be 8 fps. The peak hourly flow (maximum flow) for a 4 inch forcemain based on a maximum velocity of 8 fps can be computed as follows:

$Q = VA$

Q - Flow rate (cfs)
V - Velocity (fps)
A - Cross-sectional area of sewer (sq. ft.)

$Q = 8 \text{ fps} \times (\pi \times (2/12)^2 = 0.698 \text{ cfs}$

Q = 0.698 cfs x 448.831 gpm/cfs = 313.4 gpm

Peak Hourly Capacity = 451,296 gpd

Utilizing a Peak Hourly/Design Average factor of 4 the average daily capacity can be computed as follows:

Average Daily Capacity = 451,296 GPD/4.0 = 112,824 GPD

V. DOSING TANK SIPHONS

The dosing tank (12' (w) x 12' (l) x 6' (h) located upstream of the sand filters contains two 6 inch diameter siphons which alternate after each dosing cycle. The actual operating volume of the tank is approximately 4.5' or 4,850 gallons per dose. According to the Village of Fillmore WWTP Operation and Maintenance Manual the manufacturer of the siphons is Fluid Dynamics, Inc. According to Fluid Dynamics, Inc. published data on a 6 inch siphon with 4.5 feet maximum water depth the siphon has a maximum capacity of approximately 780 gpm. Therefore the peak hourly capacity and the average daily capacities can be computed as follows.

Peak Hourly Capacity = 780 gpm x 24 hours/day x 60 minutes/hour = 1,123,200 gpd

Average Daily Capacity = 1,123,200 gpd x 4 = 280,800 gpd

According to the Design Standards for Wastewater Treatment Works, 1988 NYSDEC Publication the siphons must have 100% excess capacity for inflow to the dosing tank. In this case the inflow to the dosing tank is equal to the pump rate in part III above or 81 gpm. Therefore 100% excess capacity of 81 gpm is equal to 162 gpm.

VI. INTERMITTENT SAND FILTERS

The WWTP currently has three single pass open intermittent sand filters. Two filters operate at any given time while the third rests. The sand filters are 50 feet (w) x 80 feet (l). According to the Design Standards for Wastewater Treatment Works, 1988 NYSDEC Publication the loading rate for this type of filter shall not exceed 5 gpd/sq. ft. for secondary treatment from septic tanks. Based on this loading rate the maximum capacity can be computed as follows:

Maximum Capacity = (50' x 80' x 5 gpd/sq. ft.) = 20,000 gpd per filter or 40,000 gpd

VII. FLOW MONITORING MANHOLE

Sand filter effluent travels to a 6 foot inside diameter manhole containing a 30 degree V-Notch weir. The V-Notch weir functions as a method of measuring flow from the WWTP. The depth of flow passing through the weir corresponds to a specific flow rate. The flow equation for a 30 degree V-Notch relative to flow depth is as follows:

$$Q = 300.72 \text{ x } H^{5/2}$$

Q	- Flow rate (gpm)
H	- Velocity (ft)

According to the Record Drawings date 10/26/87 prepared by Fagan Engineers the maximum measurable flow depth through the weir is 14 inches or 1.167 feet. Therefore the peak hourly capacity can be computed as follows:

$$Q = 300.72 \text{ x } (0.167^{5/2}) = 442 \text{ gpm}$$

Peak Hourly Capacity = 442 gpm x 24 hours/day x 60 minutes/hour = 636,480 gpd

Utilizing a Peak Hourly/Design Average factor of 4 the average daily capacity can be computed as follows:

Average Daily Capacity = 636,480 GPD/4.0 = 159,120 GPD, which corresponds to a flow depth of:

$H = (Q / 300.72)^{2/5} = (159{,}120 \text{ gpd} / (24 \text{ hours/day x } 60 \text{ minutes/hour})) / 300.72))^{2/5} =$ 0.67 feet or approximately 8 inches.

VIII. WWTP OUTFALL PIPE

Manning's Equation is utilized to calculate the hydraulic capacity of the existing 6-inch diameter WWTP outfall immediately downstream of the flow meter structure. The outfall pipe is polyvinyl chloride (PVC) and at a minimum slope of 0.5 percent based on the Record Drawings dated 10/26/87 prepared by Fagan Engineers. Manning's Equation is as follows.

$$Q = 1.49/n \text{ x } A \text{ x } R^{2/3} \text{ x } S^{1/2}$$

Q	- Flow rate (cfs)
n	- Roughness coefficient
A	- Cross-sectional area of sewer (sq. ft.)
R	- Hydraulic radius of sewer (ft.)
S	- Longitudinal sewer slope (ft./ft.)

$$Q = 1.49/n \text{ x } A \text{ x } R^{2/3} \text{ x } S^{1/2}$$

$$Q = 1.49/0.013 \text{ x } \pi \text{ x } (3/12)^2 \text{ x } (3/(2(12)))^{2/3} \text{ x } (0.005)^{1/2}$$

$$Q = 0.398 \text{ CFS} = 178.6 \text{ GPM} = 257{,}184 \text{ GPD}$$

Peak Hourly Capacity = 257,184 GPD

Utilizing a Peak Hourly/Design Average factor of 4 the average daily capacity can be computed as follows:

Average Daily Capacity = 257,184 GPD/4.0 = 64,296 GPD

SUMMARY

TABLE 1: SUMMARY OF ESTIMATED CAPACITIES OF UNIT PROCESSES		
UNIT PROCESS	ESTIMATED CAPACITIES (GPD)	
	PEAK HOURLY	AVE. DAILY
INTERCEPTOR SEWER	257,184	64,296
PUMP STATION	116,640	29,160
FORCEMAIN	451,296	112,824
DUAL SIPHON DOSING TANK	1,123,200	280,800
SAND FILTERS	-	40,000
FLOW METER	636,480	159,120
OUTFALL PIPE	257,184	64,296

REFERENCES

1. Great Lakes – Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers, Recommended Standards for Wastewater Facilities, 2004 Edition

2. Design Standards for Wastewater Treatment Works, NYSDEC Publication, 1988

3. Fagan Engineers – Consulting Engineers, Village of Fillmore WWTP Operation and Maintenance Manual, February 1987

4. Fagan Engineers – Consulting Engineers, Record Drawings for the Village of Fillmore WWTP, 10/26/087

5. Lindeburg, Michael R., Civil Engineering Reference Manual, Professional Publications Inc., 9th Edition

6. Isco Open Channel Flow Measurement Handbook, Third Edition

7. Fluid Dynamics, Inc., Performance Data for the Doing Tank Siphons

APPENDIX B

TOWN OF HUME WWTP DMR SUMMARY

Wastewater Facility Operation Report Averages for the Months of 2005-2007 (through September only)

Month	Volume of Sewage (mgd)			Temp. (Celsius)		pH (S.U.)				Settl. Sol. (ml/l)		B.O.D. (mg/l)				Susp. Sol. (mg/l)				B.O.D (lbs/Day)			Susp. Sol. (lbs/Day)			DO (mg/l)		TKN (mg/l)		UOD	
	Monthly Ave	Flow for BOD and TSS	Max (Ave)	Inf	Eff	Inf (min)	Inf. (max)	Eff (min)	Eff (max)	Inf (max)	Eff (max)	Inf (typ)	Inf Type	Eff Type	% Rem.	Inf (typ)	Inf Type	Eff Type	% Rem.	Inf	Eff	#-Rem.	Inf	Eff	#-Rem.	Inf (min)	Eff (min)	Inf	Eff	Eff (mg/l)	Eff (lbs/day)
Jan-05	0.029	0.029	0.036	9.3	9.0	6.9	7.4	6.6	7.2	0	0	200	140	19	91%	200	63	7	97%	33.9	4.6	29.3	15.2	1.7	13.5	0.8	5.3	-	-	-	-
Feb-05	0.029	0.029	0.032	9.3	8.0	6.9	7.3	6.7	7.0	0	0	200	110	9	96%	200	29	3	99%	26.6	2.2	24.4	7.0	0.7	6.3	0.8	5.1	-	-	-	-
Mar-05	0.029	0.030	0.036	10.0	9.0	7.0	7.3	6.7	7.0	0	0	200	120	12	94%	200	31	7	97%	30.0	3.0	27.0	7.8	1.8	6.0	0.8	5.6	-	-	-	-
Apr-05	0.030	0.032	0.034	9.0	11.0	7.0	7.4	6.8	7.0	0	0	200	82	5	98%	200	69	4	98%	21.9	1.3	20.5	18.4	1.1	17.3	0.7	5.2	-	-	-	-
May-05	0.029	0.028	0.033	11.0	13.0	7.0	7.3	6.8	7.0	0	0	200	120	0	100%	200	14	3	99%	28.0	0.0	28.0	3.3	0.7	2.6	0.8	5.2	-	-	-	-
Jun-05	0.031	0.030	0.040	15.0	17.0	7.0	7.3	6.7	7.0	0	0	200	52	4	98%	200	24	4	98%	13.0	1.0	12.0	5.9	1.1	4.8	0.8	5.3	110.0	5.0	28.5	7.1
Jul-05	0.029	0.026	0.034	18.0	19.0	6.9	7.3	6.7	7.0	0	0	200	82	3	99%	200	76	2	99%	17.8	0.7	17.1	16.5	0.4	16.0	0.8	5.3	39.2	5.7	29.9	6.5
Aug-05	0.031	0.030	0.037	19.0	21.0	7.0	7.3	6.8	7.0	0	0	200	73	0	100%	200	33	2	99%	18.3	0.0	18.3	8.3	0.5	7.8	0.8	5.3	33.7	2.0	8.9	2.2
Sep-05	0.029	0.028	0.034	17.0	19.0	7.0	7.3	6.7	7.0	0	0	200	110	9	96%	200	69	4	98%	25.7	2.1	23.6	16.1	0.9	15.2	0.7	5.0	44.6	9.0	54.1	12.6
Oct-05	0.030	0.034	0.036	15.0	17.0	6.7	7.0	7.0	7.3	0	0	200	87	8	96%	200	35	0	100%	24.7	2.3	22.4	9.9	0.0	9.9	0.8	5.1	52.0	8.4	49.9	14.1
Nov-05	0.030	0.030	0.036	14.5	12.9	7.1	7.7	6.8	7.5	0	0	200	74	7	97%	200	36	0	100%	18.5	1.8	16.8	9.0	0.0	9.0	0.9	5.1	-	-	-	-
Dec-05	0.031	0.029	0.037	12.6	9.6	7.2	7.6	6.8	7.2	0	0	200	110	10	95%	200	43	0	100%	26.6	2.4	24.2	10.4	0.0	10.4	0.9	5.1	-	-	-	-
Avg-05	**0.030**	**0.030**	**0.035**	**13.3**	**13.8**	**7.0**	**7.4**	**6.8**	**7.1**	**0**	**0**	**-**	**97**	**7**	**96%**	**-**	**43**	**3**	**98%**	**23.7**	**1.8**	**22.0**	**10.7**	**0.7**	**9.9**	**0.8**	**5.2**	**55.9**	**6.0**	**34.3**	**8.5**
Jan-06	0.033	0.034	0.038	13.0	10.0	7.2	7.6	6.9	7.4	0	0	200	130	8	96%	200	35	3	99%	36.9	2.3	34.6	9.9	0.9	9.1	0.9	5.1	-	-	-	-
Feb-06	0.029	0.029	0.033	10.0	7.0	7.2	7.6	6.8	7.2	0	0	200	140	7	97%	200	53	3	99%	33.9	1.7	32.2	12.8	0.7	12.1	0.8	5.1	-	-	-	-
Mar-06	0.031	0.029	0.036	10.0	9.0	7.2	7.5	6.9	7.2	0	0	200	68	11	95%	200	23	4	98%	16.4	2.7	13.8	5.6	1.0	4.6	0.8	5.2	-	-	-	-
Apr-06	0.031	0.032	0.038	12.0	14.0	7.3	7.6	6.8	7.1	0	0	200	130	0	100%	200	46	3	99%	34.7	0.0	34.7	12.3	0.8	11.5	0.7	5.1	-	-	-	-
May-06	0.031	0.030	0.036	12.3	15.6	7.3	7.6	6.8	7.1	0	0	200	73	8	96%	200	61	5	98%	18.3	2.0	16.3	15.3	1.3	14.0	0.8	5.0	47.9	14.9	79.1	19.8
Jun-06	0.030	0.029	0.038	14.0	16.0	7.3	7.6	6.8	7.2	0	0	200	120	0	100%	200	18	0	100%	29.0	0.0	29.0	4.4	0.0	4.4	0.8	5.2	33.2	4.4	19.9	4.8
Jul-06	0.030	0.029	0.038	15.0	17.0	7.3	7.6	6.7	7.2	0	0	200	90	7	97%	200	58	13	94%	21.8	1.7	20.1	14.0	3.1	10.9	0.8	5.1	48.4	7.2	42.9	10.4
Aug-06	0.030	0.029	0.037	16.0	18.0	7.3	7.6	6.8	7.1	0	0	200	59	0	100%	200	48	0	100%	14.3	0.0	14.3	11.6	0.0	11.6	0.6	5.3	28.6	2.5	11.4	2.8
Sep-06	0.030	0.029	0.040	16.0	18.0	7.3	7.6	6.8	7.0	0	0	200	100	0	100%	200	103	0	100%	24.2	0.0	24.2	24.9	0.0	24.9	0.8	5.1	56.4	5.7	25.8	6.2
Oct-06	0.029	0.030	0.036	16.0	15.0	7.2	7.6	6.8	7.2	0	0	200	93	0	100%	200	44	0	100%	23.3	0.0	23.3	11.0	0.0	11.0	0.7	5.2	50.2	4.8	21.8	5.4
Nov-06	0.029	0.028	0.039	16.0	12.0	7.2	7.6	7.5	7.9	0	0	200	140	11	95%	200	45	5	98%	32.7	2.6	30.1	10.5	1.2	9.3	0.3	5.1				
Dec-06	0.026	0.025	0.031	15.0	11.0	6.9	7.6	7.3	7.9	0	0	200	62	0	100%	200	37	3	99%	12.9	0.0	12.9	7.7	0.6	7.1	0.3	5.1				
Avg-06	**0.030**	**0.029**	**0.037**	**13.8**	**13.6**	**7.2**	**7.6**	**6.9**	**7.3**	**0**	**0**	**-**	**100**	**4**	**98%**	**-**	**48**	**3**	**98%**	**24.9**	**1.1**	**23.8**	**11.7**	**0.8**	**10.9**	**0.7**	**5.1**	**44.1**	**6.6**	**33.5**	**8.2**
Jan-07	0.030	0.029	0.034	13.0	10.0	7.4	7.9	6.5	7.3	0	0	200	86	21	90%	200	73	8	96%	20.8	5.1	15.7	17.7	1.9	15.7	0.6	5.0	-	-	-	-
Feb-07	0.031	0.033	0.036	12.0	9.0	7.6	7.9	6.9	7.4	0	0	200	65	28	86%	200	12	9	96%	17.9	7.7	10.2	3.3	2.5	0.8	0.7	5.1	-	-	-	-
Mar-07	0.029	0.027	0.033	13.0	10.0	7.3	7.8	6.8	7.3	0	0	200	78	24	88%	200	29	15	93%	17.6	5.4	12.2	6.5	3.4	3.2	0.8	5.1	-	-	-	-
Apr-07	0.028	0.030	0.033	14.0	11.0	7.2	7.6	6.8	7.2	0	0	200	69	0	100%	200	16	0	100%	17.3	0.0	17.3	4.0	0.0	4.0	0.8	5.2	-	-	-	-
May-07	0.029	0.026	0.034	14.0	16.0	7.0	7.6	6.8	7.2	0	0	200	143	0	100%	200	40	0	100%	31.0	0.0	31.0	8.7	0.0	8.7	0.8	5.2	47.2	5.1	23.0	5.0
Jun-07	0.030	0.032	0.034	17.0	18.5	7.0	7.4	6.7	7.1	0	0	200	87	0	100%	200	60	0	100%	23.2	0.0	23.2	16.0	0.0	16.0	0.7	5.2	60.8	1.1	4.9	1.3
Jul-07	0.028	0.027	0.033	18.0	20.0	6.9	7.5	6.6	7.2	0	0	200	86	0	100%	200	100	5	98%	19.4	0.0	19.4	22.5	1.1	21.4	0.7	5.2	49.8	11.2	50.4	11.3
Aug-07	0.029	0.033	0.033	19.0	22.0	7.0	7.4	6.7	7.0	0	0	200	107	0	100%	200	36	0	100%	29.4	0.0	29.4	9.9	0.0	9.9	0.7	5.3	57.4	2.4	10.8	3.0
Sep-07	0.029	0.027	0.034	17.0	19.0	7.0	7.4	6.7	7.2	0	0	200	112	7	97%	200	44	6	97%	25.2	1.6	23.6	9.9	1.4	8.6	1.0	5.3	-	0.0	10.5	2.4
Oct-07	0.030	0.032	0.039	17.0	16.0	7.0	7.4	6.8	7.1	0	0	200	92	2	99%	200	72	5	98%	24.6	0.5	24.0	19.2	1.3	17.9	0.7	5.1	39.1	1.1	8.1	1.8
Avg-07	**0.029**	**0.030**	**0.034**	**15.4**	**15.2**	**7.1**	**7.6**	**6.7**	**7.2**	**0**	**0**	**-**	**93**	**8**	**96%**	**-**	**48**	**5**	**98%**	**22.6**	**2.0**	**20.6**	**11.8**	**1.2**	**10.6**	**0.8**	**5.2**	**50.9**	**3.5**	**17.9**	**4.1**
Avg-05, 06, 07	**0.030**	**0.030**	**0.035**	**14.2**	**14.2**	**7.1**	**7.5**	**6.8**	**7.2**	**0**	**0**	**-**	**97**	**7**	**97%**	**-**	**46**	**4**	**98%**	**23.7**	**1.6**	**22.1**	**11.4**	**0.9**	**10.5**	**0.7**	**5.2**	**50.3**	**5.4**	**28.6**	**7.0**
Min-05, 06, 07	**0.026**	**0.025**	**0.031**	**9.0**	**7.0**	**6.7**	**7.0**	**6.5**	**7.0**	**0**	**0**	**-**	**52**	**0**	**86%**	**-**	**12**	**0**	**93%**	**12.9**	**0.0**	**10.2**	**3.3**	**0.0**	**0.8**	**0.3**	**5.0**	**28.6**	**0.0**	**4.9**	**1.3**
Max-05, 06, 07	**0.033**	**0.034**	**0.040**	**19.0**	**22.0**	**7.6**	**7.9**	**7.5**	**7.9**	**0**	**0**	**-**	**143**	**28**	**100%**	**-**	**103**	**15**	**100%**	**36.9**	**7.7**	**34.7**	**24.9**	**3.4**	**24.9**	**1.0**	**5.6**	**110.0**	**14.9**	**79.1**	**19.8**

APPENDIX C

TOWN OF CANEADEA WWTP – ESTIMATION OF INDIVIDUAL CAPACITIES OF UNIT TREATMENT PROCESSES

APPENDIX C

ESTIMATION OF INDIVIDUAL CAPACITIES OF UNIT TREATMENT PROCESSES

The purpose of the following calculations is to develop an estimate of the hydraulic and organic capacities of the various unit processes utilized within the Town of Caneadea's existing WWTP. The Great Lakes – Upper Mississippi River Board of State Public Health and Environmental Manager's Recommended Standards for Wastewater Facilities, 2004 Edition, (Ten States' Standards) is utilized as the basis for the estimation of these various unit capacities. Other reference sources, including equipment manufacturer's literature, are also used and noted accordingly.

WASTEWATER TREATMENT & CONVEYANCE PROCESSES

I. INFLUENT SEWER

Manning's Equation is utilized to calculate the hydraulic capacity of the existing 12-inch diameter sewer section immediately before the WWTP. This sewer is cast-iron pipe (CIP) and has a slope of 0.28 percent. Manning's Equation is as follows.

$$Q = 1.49/n \times A \times R^{2/3} \times S^{1/2}$$

Q	- Flow rate (CFS)
n	- Roughness coefficient
A	- Cross-sectional area of sewer (sq. ft.)
R	- Hydraulic radius of sewer (ft.)
S	- Longitudinal sewer slope (ft./ft.)

$$Q = 1.49/n \times A \times R^{2/3} \times S^{1/2}$$

$$Q = 1.49/0.018 \times \pi \times (6/12)^2 \times (6/(2(12)))^{2/3} \times (0.0028)^{1/2}$$

$$Q = 1.36 \text{ CFS} = 610.3 \text{ GPM} = 878{,}832 \text{ GPD}$$

Peak Hourly Capacity = 878,832 GPD

Average Daily Capacity = 878,832 GPD/3.7 = 237,522 GPD

II. KENNISON NOZZEL

Raw influent wastewater flow rates are measured via a 10-inch diameter Kennison Nozzel (Reference 4). As per Reference 9, the maximum capacity of this measuring device is 840,000 GPD.

Peak Hourly Capacity = 840,000 GPD

Average Daily Capacity = 840,000 GPD/3.7 = 227,027 GPD

III. COMMINUTOR

A single comminutor is utilized to treat raw wastewater prior to the grit removal process (Reference 4). As per Section 62.32 of Reference 1, "Comminutor capacity shall be adequate to handle design peak hourly flow." The existing comminutor is a CC-18 Sewer Chewer from the Yeomans Chicago Corporation. Utilizing Reference 10, the hydraulic capacity of this unit is estimated as follows.

Max. Downstream water depth = 14.5 inches (Refer to Section IV of this appendix).

Max. Upstream water depth = 21 inches

Allowable headloss across unit = 21 – 14.5 = 6.5 inches

Peak Hourly Capacity = 910 GPM = 1,310,400 GPD (as per Reference 10)

Average Daily Capacity = 1,310,400 GPD/3.7 = 354,162 GPD

IV. GRIT REMOVAL

Grit removal is accomplished via two parallel horizontal flow grit chambers at the Town of Caneadea's WWTP (Reference 4). Each of these chambers is 18 feet long x 1 foot wide. Water depth within these chambers is controlled via proportional (Sutro) weirs at the tail end of each of the chambers.

The discharge from one Sutro weir at a water elevation of 1178.21, the design maximum water elevation as per Reference 4, is calculated as follows. Note dimensional and elevation data for the Sutro weirs are obtained from Sheet 14 of Reference 4.

$$Q = C_d b(2ga(h_1 – a/3))^{0.5}$$

$$Q = (0.608)(0.79)(2 \times 32.2 \times 0.104 \times (0.88 - .104/3))^{0.5}$$

$$Q = 1.14 \text{ CFS}$$

The associated depth of flow is 1.21 feet, (1178.21 – 1177.00). The average velocity through the grit channel is calculated as follows.

Velocity = 1.14 CFS/(1 feet wide x 1.21 feet deep) = 0.94 fps

As per Table 9.3 of Reference 1, horizontal velocities in a grit chamber range from 0.8 to 1.3 fps. As such, the current design of the grit chamber would accommodate recommended flow velocities.

A typical settling velocity for 65-mesh grit is 3.8 ft/min. The theoretical detention time to allow this particle size to settle is calculated as follows.

Detention Time = 1.21 feet/3.8 ft/min = 0.32 min

The effective volume of a single grit chamber is calculated as follows. As per Reference 1, allowance should be made for inlet and outlet turbulence. At least, a 50 percent increase in the theoretical channel length is recommended.

Effective Volume = 0.5 x 18 feet x 1.21 feet x 1 feet = 10.9 cubic feet

The associated treatment capacity for two grit chambers is, in turn, calculated as follows.

Peak Hourly Capacity = 2 x 10.9 cubic feet x 7.481/0.32 min
= 509.6 GPM = 733,824 GPD

Average Daily Capacity = 733,824/3.7 = 198,331 GPD

The peak hourly and average daily capacities for the grit chambers would be reduced to 482,779 GPD and 130,481 GPD, respectively, if 100-mesh grit were to be removed. 100-mesh grit has a settling velocity of 2.5 ft./min.

V. SUCTION PIPING INLETS FOR INFLUENT PUMPS

5.1 SUCTION LINES

As per Reference 4, a submergence above each of the four 6-inch diameter suction lines from the wet well through the wall to the pump area of 2.88 feet can be provided. The hydraulic capacity of this suction piping is based on the amount of submergence and the associated occurrence of vortices. The following relationship is used to estimate the maximum acceptable velocity through a suction inlet (as per Reference 2).

$$S/D = 1.0 + 2.3\ F_D$$

Where;

F_D	-	Froude number = $V/(gD)^{0.5}$
D	-	outlet fitting diameter
V	-	outlet fitting velocity
G	-	acceleration of gravity
S	-	submergence

$$2.88/.5 = 1.0 + 2.3F_D$$

$$F_D = 2.06$$

$$2.06 = V/(32.2 \times 0.5)^{0.5}$$

V = 8.26 FPS (As per Table 9.8.3 of Reference 2, an acceptable velocity range of 2 to 9 FPS is noted.)

$$Q/\text{inlet} = (\pi \times (3/12)^2) \times 8.26\ \text{FPS} = 1.62\ \text{CFS} = 727\ \text{GPM}$$

Assuming three pumps functioning (i.e. one pump out of service), the associated pumping capacity of the suction piping inlets is calculated as follows.

Pumping Capacity = 3 x 727 x 24 x 60 = 3,140,640 GPD

Assuming that the recycle flow is 75 percent the average flow rate (as per Reference 4) and a peaking factor of 3.7, the average daily and peak hourly capacities of the suction lines is calculated as follows.

3,140,640 GPD = 3.7Q + 0.75Q = 4.45Q

Q = Average Daily Capacity = 705,762 GPD

Peak Hourly Capacity = 3.7 x 705,762 GPD = 2,611,319 GPD

5.2 TRENCH-TYPE WET WELL

The wet well of the existing WWTP is a trench-type. As per Figure 9.8.14 of Reference 2, the maximum recommended velocity above the rectangular trench is 1 FPS. At a water surface elevation of 1176.50 feet in the wet well, the associated cross-sectional flow area in the wet well is 13.1 sq. ft. Given the maximum velocity of 1 FPS, the associated capacity is calculated as follows.

Flow Capacity = 13.1 sq. ft. x 1 FPS = 13.1 CFS = 5,879 GPM

= 8,465,760 GPD

Assuming that the recycle flow is 75 percent of the average flow rate (as per Reference 4) and a peaking factor of 3.7, the average daily and peak hourly capacities of the trench-type wet well is calculated as follows.

8,465,760 GPD = 3.7Q + 0.75Q = 4.45Q

Q = Average Daily Capacity = 1,902,418 GPD

Peak Hourly Capacity = 3.7 x 1,902,418 GPD = 7,038,947 GPD

VI. RAW INFLUENT PUMPING

Four raw influent pumps are located within the lower level of the Control Building. In addition to pumping raw influent wastewater flows, these pumps also receive the recycle stream for the trickling filter (a maximum flow of roughly 210,000 GPD, as per Reference 4). As per Reference 4, the intended design discharge rates of these pumps, with the largest unit out of service, is 632 GPM.

As per Section 42.31 of Ten States' Standards, "Multiple pumps shall be provided. When only two pumps are provided, they shall be the same size. Units shall have the capacity such that, with any unit out of service, the remaining units will have capacity to handle the design peak hourly flow."

Peak Hourly Capacity = (632 GPM x 60 x 24) – 210,000 GPD = 700,080 GPD

Average Daily Capacity = 700,080 GPD/3.7 = 189,211 GPD

VII. PRIMARY SETTLING (INCLUDING OVERFLOW WEIRS)

Primary settling is provided by a single rectangular settling tank, which has plan dimensions of 15.5 feet x 53 feet feet. Therefore, the surface area of this unit is 821.5 sq. ft. In addition to receiving raw wastewater, the primary settling unit also receives the recycle stream for the trickling filter (a maximum flow of roughly 210,000 GPD, as per Reference 4). As per Section 72.21 of Ten States' Standards, the following design surface overflow rates are recommended for primary settling tanks which do not receive activated sludge.

Design Average - 1,000 GPD/sq.ft.

Peak Hourly - 1,500 to 2,000 GPD/sq. ft. (1,700 GPD/sq. ft. is used for this analysis)

Peak Hourly Capacity = (1,700 GPD/sq. ft. x 821.5 sq. ft.) – 210,000 GPD
= 1,186,550 GPD

Average Daily Capacity = (1,000 GPD/sq. ft. x 821.5 sq. ft.) – 210,000 GPD
= 611,500 GPD

An overflow weir is located near the end of the primary settling tank. As per Section 72.43 of Reference 1, the maximum allowable weir loading rate at design peak hourly flow is 20,000 GPD/LF of weir (for plants with an average capacity of 1 MGD or less). As per Reference 4, the length of weir is 45.5 feet. The associated capacity of the weir is calculated as follows.

Peak Hourly Capacity = 45.5 x 20,000 = 910,000 GPD

Average Daily Capacity = 910,000 GPD/3.7 = 245,945 GPD

VIII. TRICKLING FILTER

To estimate the treatment capacity of the existing trickling filter, the Modified Velz Equation shall be utilized. The existing trickling filter used to have rock media, which has been changed to plastic media from Munters.

$$(K_{20}A_sD\theta^{(T-20)})/Q^n) = \ln(S_i/S_e)$$

Where;

S_i -	soluble influent BOD (mg/l)
Se -	soluble effluent BOD (mg/l)
θ -	temperature correction coefficient
T -	wastewater temperature
N -	flow exponent
As -	Media specific surface area (sq. ft./cu. ft.)
D -	Media depth
Q -	Trickling filter feed flux (GPM/sq. ft.)

Q_n = 280,000 GPD/(24 x 60)/($\pi(20.5)^2$) = 0.147 GPM/sq. ft.

Se = 15 mg/l (Assume that effluent soluble BOD_5 is ½ of the permitted limit of 30 mg/l)

K_{20} = 0.0024

A_s = 42 sq. ft./cubic feet

D = 6 feet

θ = 1.035

T = 10 ^{0}C (Winter conditions)

Substituting these values into the above equation, an associated influent soluble BOD_5, S_i, is calculated to be 46 mg/l. Assuming that the soluble portion of the influent BOD is 35 percent, the associated organic loading capacity of the existing trickling filter is calculated as follows.

Organic Loading Capacity = 46/0.35 x 0.280 x 8.34 = 358 lbs BOD_5/day

IX. SECONDARY SETTLING (INCLUDING OVERFLOW WEIRS)

Secondary settling is provided by a single 22 foot diameter unit. Therefore, the surface area of the secondary settling tank is 380.1 sq. ft. As per Section 72.231 of Ten States' Standards, "Surface overflow rates for settling tanks following trickling filters or rotating biological contactors shall not exceed 1,200 GPD/sq. ft., based on design peak hourly flow."

Peak Hourly Capacity = 1,200 GPD/sq. ft. x 380.1 sq. ft. = 456,120 GPD

Average Daily Capacity = 456,120 GPD/3.7 = 123,275 GPD

An overflow weir is located near the end of the primary settling tank. As per Section 72.43 of Reference 1, the maximum allowable weir loading rate at design peak hourly flow is 20,000 GPD/LF of weir (for plants with an average capacity of 1 MGD or less). As per Reference 4, the length of weir is 62 feet. The associated capacity of the weir is calculated as follows.

Peak Hourly Capacity = 62 x 20,000 = 1,240,000 GPD

Average Daily Capacity = 1,240,000 GPD/3.7 = 335,135 GPD

X. CHLORINE CONTACT TANK

The existing chlorine contact basin has a volume of approximately 7290 gallons (13 feet x 15 feet x 5 feet deep). As per Section 102.44 of Ten States' Standards, "For a chlorination system, a minimum contact period of 15 minutes at design peak hourly flow or maximum rate of pumpage shall be provided after thorough mixing." As per Table C-5 of Reference 3, the baffle classification within the chlorine contact tank can be described as average to superior. An associated T_{10}/T factor of 0.6 is used for this evaluation.

Peak Hourly Capacity = 7290 gallons x 0.6/15 minutes = 291.6 GPM = 419,904 GPD

Average Daily Capacity = 419,904 GPD/3.7 = 113,487 GPD

XI. EFFLUENT SEWER

As per Section 51.2 of Ten States' Standards, treatment plants should remain fully operational and accessible during the 25-year flood event. Treated effluent is conveyed to the Genesee River via an 8-inch diameter steel outfall sewer, which begins in the chlorine contact tank.

The approximate length of the effluent sewer is 495 LF. This sewer is 12-inch diameter CIP. The design flood elevation of the Genesee River is approximately 1194 feet (Reference 4). The elevation of the top of the baffles of the chlorine contact tank is approximately 1195.33 feet (Reference 4). The estimated capacity of the existing effluent outfall sewer is as follows. Refer to the attached calculations.

Manning's Equation is utilized to calculate the hydraulic capacity of the existing 12-inch diameter sewer section immediately before the WWTP. This sewer is cast-iron pipe (CIP). Manning's Equation is as follows.

$$Q = 1.49/n \times A \times R^{2/3} \times S^{1/2}$$

Q	- Flow rate (CFS)
n	- Roughness coefficient
A	- Cross-sectional area of sewer (sq. ft.)
R	- Hydraulic radius of sewer (ft.)
S	- Hydraulic grade slope (ft./ft.)

$$Q = 1.49/n \times A \times R^{2/3} \times S^{1/2}$$

$$Q = 1.49/0.018 \times \pi \times (6/12)^2 \times (6/(2(12)))^{2/3} \times ((1195.33 - 1194)/495)^{1/2}$$

$$Q = 1.34 \text{ CFS} = 601 \text{ GPM} = 865{,}440 \text{ GPD}$$

Peak Hourly Capacity = 865,440 GPD

Average Daily Capacity = 865,400 GPD/3.7 = 233,892 GPD

SLUDGE TREATMENT AND PROCESSING

I. ANAEROBIC DIGESTERS

Anaerobic digestion is utilized at the Town of Caneadea's WWTP for the stabilization of sludge. The anaerobic digestion system is a two-stage process, consisting of a primary digester unit for active sludge digestion followed by a secondary unit for the storage of sludge and gas. Sludge mixing is accomplished by external recirculation pumps. As per Reference 4, the volume of the primary digester unit is 15,670 cu. ft.

As per Section 84.323 of Reference 1, "For digestion systems utilizing two stages (primary and secondary units), the first stage (primary) may be either completely mixed or moderately mixed and loaded in accordance with Paragraphs 84.321 and 84.322. The second stage (secondary) is to be designed for sludge storage, concentration, and gas collection and shall not be credited in the calculations for volumes required for sludge digestion."

As per Section 84.322 of Reference 1, "For digestion systems where mixing is accomplished only by circulating sludge through an external heat exchanger, the system may be loaded up to 40 lbs of volatile solids per 1000 cubic feet of volume per day in the active digestion units. This loading may be modified upward or downward depending upon the degree of mixing provided."

Allowable volatile solids loading = 15,670 cu. ft./1000 x 40 = 626.8 lbs VSS per day

Assuming that the raw sludge consists of 70 percent volatile solids, the associated allowable total solids loading to the primary digester unit is calculated as follows.

Total solids loading = 626.8/0.7 = 895.4 lbs TSS per day

As per Section 11.253(a) of Reference 1, domestic wastewater design shall be based on at least 0.17 lbs of BOD_5 per day per capita.

Effective population accommodated = 895.4/0.17 = 5,267 capita

II. PAVED DRYING BEDS

Paved drying beds are currently utilized for sludge dewatering at the Town of Caneadea's WWTP. The WWTP has three beds, each with rough plan dimensions of 25 feet x 25 feet. These existing beds are covered by a roof system. As per Section 77.12 of Reference 5, "In the absence of rational design, the size of the sand drying bed may be estimated on the basis of 1.25 to 1.75 sq. ft./capita for primary and humus digested sludge, when drying beds are the primary method of dewatering." In comparison, as per Reference 8, in regards to paved drying beds, 2.5 sq. ft./capita should be utilized for sizing. The following is an estimate of the number of capita which can be accommodated by the existing drying beds.

Existing bed area = 6 x 27'-4" x 26'-4" = 4,317 sq. ft.

Population accommodated by existing beds = 4,317 sq. ft./2.5 sq. ft./capita

= 1,727 capita

SUMMARY

TABLE 1: SUMMARY OF ESTIMATED CAPACITIES OF UNIT PROCESSES

UNIT PROCESS	ESTIMATED CAPACITIES (GPD)	
	PEAK HOURLY	AVE. DAILY
INFLUENT SEWER	878,832	237,522
KENNISON NOZZEL	840,000	227,027
COMMINUTOR	1,310,400	354,162
GRIT REMOVAL	733,824	198,331
SUCTION LINES	2,611,319	705,762
WET WELL	7,038,947	1,902,418
RAW INFLUENT PUMPING	700,080	189,211
PRIMARY SETTLING	1,186,550	611,500
PRIMARY SETTLING WEIRS	910,000	245,945
TRICKLING FILTER		280,000
SECONDARY SETTLING	456,120	123,275
SECONDARY SETTLING WEIRS	1,240,000	335,135
CHLORINE CONTACTTANK	419,904	113,487
EFFLUENT SEWER	865,400	233,892

TABLE 2: EXISTING WWTP ORGANIC TREATMENT CAPACITY

TRICKLING FILTER PROCESS	358 LB BOD_5 PER DAY

TABLE 3: EXISTING SLUDGE PROCESSING CAPACITY

UNIT PROCESS	ESTIMATED CAPACITY (CAPITA)
ANAEROBIC DIGESTION	5267
PAVED DRYING BEDS	1727

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8. Minnesota Pollution Control Agency, Wastewater Technical Review and Guidance, wq-wwtp5-74

9. Isco Open Channel Flow Measurement Handbook, Third Edition

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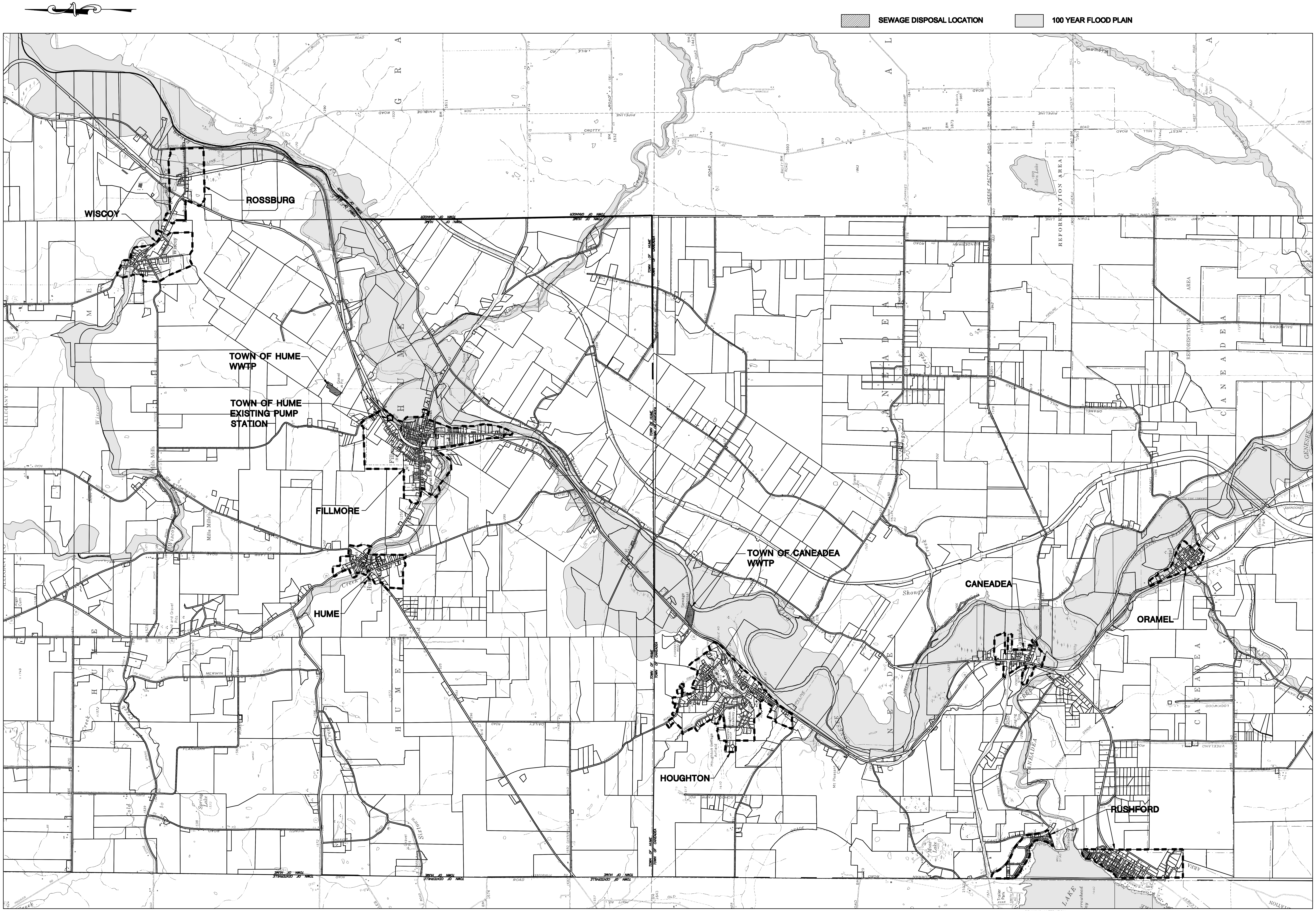
APPENDIX D

TOWN OF CANEADEA WWTP
DMR SUMMARY

Wastewater Facility Operation Report Averages for the Months of 2005-2007 (through October only)

Month	Volume of Sewage (mgd)			Temp. (Celsius)		pH (S.U.)		Settl. Sol. (ml/l)		B.O.D. (mg/l)			Susp. Sol. (mg/l)			B.O.D (lbs/Day)			Susp. Sol. (lbs/Day)		
	Monthly Ave	Flow for BOD and TSS	Max (day)	Inf	Eff	Inf (max)	Eff (max)	Inf (max)	Eff (max)	Inf Type	Eff Type	% Rem.	Inf Type	Eff Type	% Rem	Inf	Eff	#-Rem	Inf	Eff	#-Rem
Jan-05	0.1515	0.1401	0.420	12	9	8.0	8.0	8	0	378	10	97%	392	8	98%	442	12	430	458	9	449
Feb-05	0.1927	0.1892	0.340	13	10	7.8	7.7	7	0	327	25	92%	288	5	98%	516	39	477	454	8	447
Mar-05	0.2173	0.1867	0.570	11	10	8.0	7.8	8	0	236	15	94%	380	10	97%	367	23	344	592	16	576
Apr-05	0.2010	0.2445	0.800	12	11	7.8	7.8	8	0	147	19	87%	122	24	80%	300	39	261	249	49	200
May-05	0.1357	0.1150	0.800	13	13	7.8	7.8	10	0	210	21	90%	180	5	97%	201	20	181	173	5	168
Jun-05	0.1276	0.1016	0.580	15	19	7.6	7.8	8	0	105	10	90%	1220	16	99%	89	8	80	1034	14	1020
Jul-05	0.1130	0.1378	0.340	17	20	7.6	7.8	6	0	167	7	96%	130	6	95%	192	8	184	149	7	143
Aug-05	0.0777	0.0835	0.350	19	20	7.5	7.7	10	0	631	7	99%	513	5	99%	439	5	435	357	3	354
Sep-05	0.1598	0.1560	0.440	19	20	7.7	7.8	8	0	343	25	93%	198	22	89%	446	33	414	258	29	229
Oct-05	0.1557	0.1503	0.350	18	17	7.7	7.6	9	0	208	17	92%	202	15	93%	261	21	239	253	19	234
Nov-05	0.1214	0.1411	0.460	17	14	7.9	7.9	10	0	213	17	92%	204	24	88%	251	20	231	240	28	212
Dec-05	0.1373	0.1760	0.310	14	10	8.0	7.9	10	0	232	20	91%	174	6	97%	341	29	311	255	9	247
Avg-05	**0.1492**	**0.1518**	**0.480**	**15**	**14**	**7.8**	**7.8**	**9**	**0**	**266**	**16**	**93%**	**334**	**12**	**94%**	**320**	**21**	**299**	**373**	**16**	**356**
Jan-06	0.1351	0.1261	0.330	13	11	7.8	7.8	11	0	56	12	79%	204	5	98%	59	13	46	215	5	209
Feb-06	0.1699	0.1614	0.380	12	10	7.7	7.7	6	0	153	19	88%	348	5	99%	206	26	180	468	7	462
Mar-06	0.1550	0.1114	0.410	11	10	7.8	7.6	8	0	527	8	98%	468	5	99%	490	7	482	435	5	430
Apr-06	0.1278	0.0979	0.320	13	14	8.0	7.6	10	0	234	7	97%	200	5	98%	191	6	185	163	4	159
May-06	0.1410	0.1519	0.320	14	15	7.6	7.8	12	0	148	16	89%	116	5	96%	187	20	167	147	6	141
Jun-06	0.1130	0.0894	0.370	15	18	7.9	7.8	8	0	192	6	97%	262	10	96%	143	4	139	195	7	188
Jul-06	0.1264	0.1843	0.480	18	20	7.8	7.9	12	0	233	14	94%	352	18	95%	358	22	337	541	28	513
Aug-06	0.1250	0.1250	0.200	19	20	7.5	7.8	10	0	159	6	96%	226	14	94%	166	6	160	236	15	221
Sep-06	0.0979	0.1516	0.310	19	19	7.6	7.8	9	0	165	23	86%	121	19	84%	209	29	180	153	24	129
Oct-06	0.1874	0.2354	0.575	18	16	7.5	7.6	8	0	200	25	88%	224	14	94%	393	49	344	440	27	412
Nov-06	0.1856	0.1958	0.640	16	15	7.7	7.6	12	0	172	10	94%	168	7	96%	281	16	265	274	11	263
Dec-06	0.1537	0.1853	0.665	14	12	7.6	7.6	12	0	228	19	92%	210	7	97%	352	29	323	325	11	314
Avg-06	**0.1432**	**0.1513**	**0.417**	**15**	**15**	**7.7**	**7.7**	**10**	**0**	**206**	**14**	**91%**	**242**	**10**	**95%**	**253**	**19**	**234**	**299**	**13**	**287**
Jan-07	0.1763	0.1866	0.650	13	10	7.9	7.9	8	0	169	15	91%	152	6	96%	263	23	240	237	9	227
Feb-07	0.1688	0.1346	0.360	11	8	7.5	7.9	11	0	228	18	92%	264	17	94%	256	20	236	296	19	277
Mar-07	0.2007	0.1426	0.580	11	10	7.5	7.6	10	0	159	19	88%	194	12	94%	189	23	166	231	14	216
Apr-07	0.1563	0.1496	0.390	12	11	7.6	7.6	10	0	182	15	92%	158	5	97%	227	19	208	197	6	191
May-07	0.1629	0.1665	0.330	13	14	7.6	7.6	8	0	170	6	96%	206	5	98%	236	8	228	286	7	279
Jun-07	0.1429	0.1248	0.320	14	17	7.6	7.6	10	0	132	6	95%	166	5	97%	137	6	131	173	5	168
Jul-07	0.1664	0.1694	0.440	17	19	7.6	7.8	8	0	680	6	99%	440	5	99%	961	8	952	622	7	615
Aug-07	0.1571	0.1412	0.360	17	20	7.6	7.8	7	0	118	6	95%	142	20	86%	139	7	132	167	24	144
Sep-07	0.1959	0.2050	0.440	19	19	7.8	7.8	12	0	233	14	94%	216	10	95%	398	24	374	369	17	352
Oct-07	0.1861	0.3500	0.520	18	17	7.6	8.0	10	0	259	21	92%	234	8	97%	756	61	695	683	23	660
Avg-07	**0.1713**	**0.1770**	**0.439**	**15**	**15**	**7.6**	**7.8**	**9**	**0**	**233**	**13**	**93%**	**217**	**9**	**95%**	**356**	**20**	**336**	**326**	**13**	**313**
Avg-05, 06, 07	**0.155**	**0.160**	**0.445**	**15**	**15**	**7.7**	**7.8**	**9**	**0**	**235**	**14**	**93%**	**264**	**10**	**95%**	**310**	**20**	**290**	**333**	**14**	**319**
Min-05, 06, 07	**0.078**	**0.0835**	**0.200**	**11**	**8**	**7.5**	**7.6**	**6**	**0**	**56**	**6**	**79%**	**116**	**5**	**80%**	**59**	**4**	**46**	**147**	**3**	**129**
Max-05, 06, 07	**0.217**	**0.3500**	**0.800**	**19**	**20**	**8.0**	**8.0**	**12**	**0**	**680**	**25**	**99%**	**1220**	**24**	**99%**	**961**	**61**	**952**	**1034**	**49**	**1020**



DRAWING ALTERATION
THE FOLLOWING IS AN EXCERPT FROM THE NEW YORK EDUCATION LAW ARTICLE 145 SECTION 7209 AND APPLIES TO THIS DRAWING.
"IT IS A VIOLATION OF THIS LAW FOR ANY PERSON UNLESS HE IS ACTING UNDER THE DIRECTION OF A LICENSED PROFESSIONAL ENGINEER OR LAND SURVEYOR TO ALTER AN ITEM IN ANY WAY. IF AN ITEM BEARING THE SEAL OF AN ENGINEER OR LAND SURVEYOR IS ALTERED, THE ALTERING ENGINEER OR LAND SURVEYOR SHALL AFFIX TO THE ITEM HIS SEAL AND THE NOTATION "ALTERED BY" FOLLOWED BY HIS SIGNATURE AND THE DATE OF SUCH ALTERATION AND A SPECIFIC DESCRIPTION OF THE ALTERATION".

No.	Revisions and Descriptions	By	Date

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Checked By:
Scale: 1" = 2000'
Date: 7/08

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TOWN OF CANEADEA & TOWN OF HUME
ALLEGANY COUNTY, NEW YORK

Drawing Title:
EXISTING WWTP LOCATIONS

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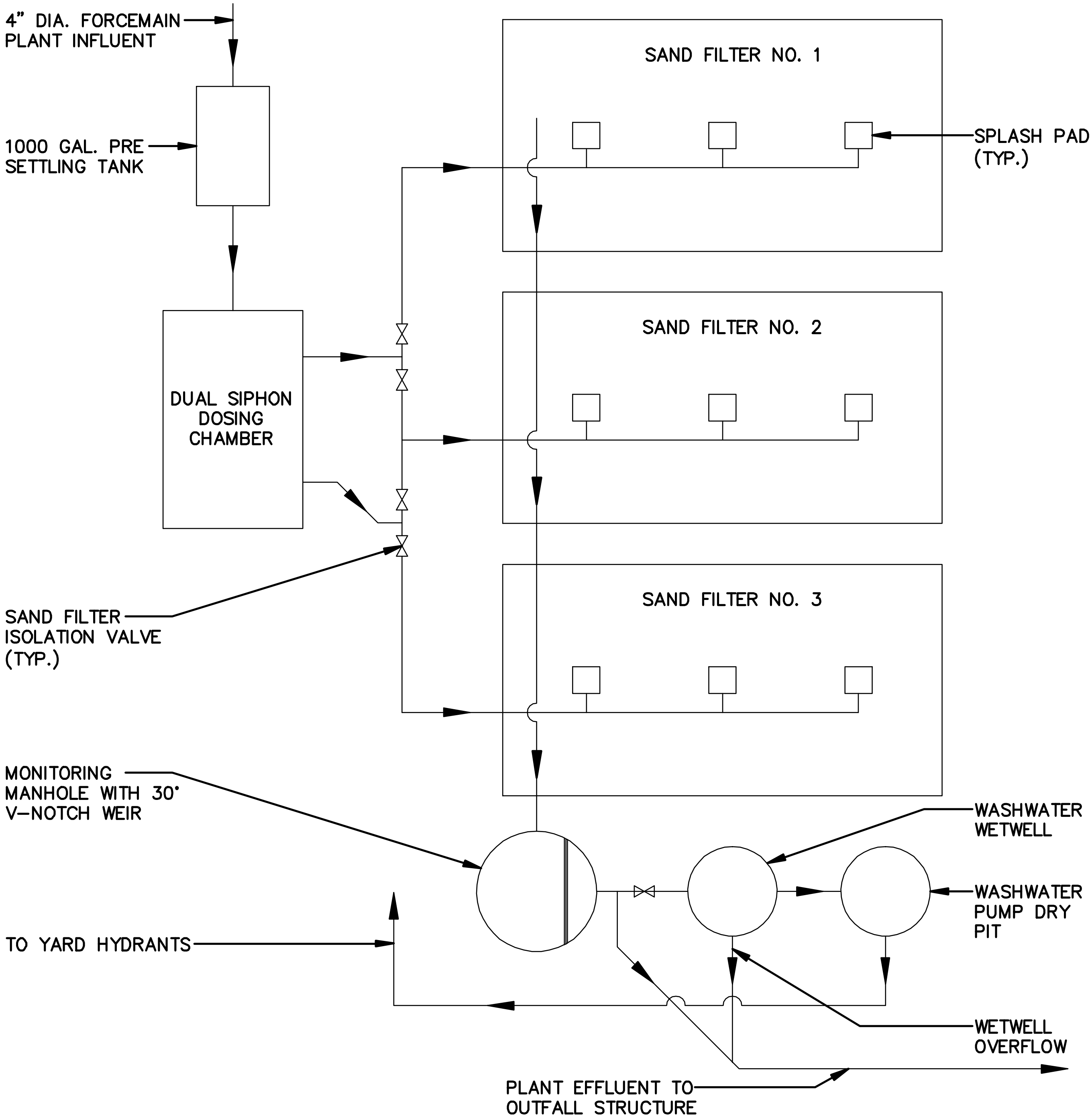
Figure No.
III.1

Project No.
080911



TOWN OF HUME WWTP SCHEMATIC

Project Name: **HUME-CANEADEA SEWER CONSOLIDATION STUDY**
TOWN OF CANEADEA & TOWN OF HUME
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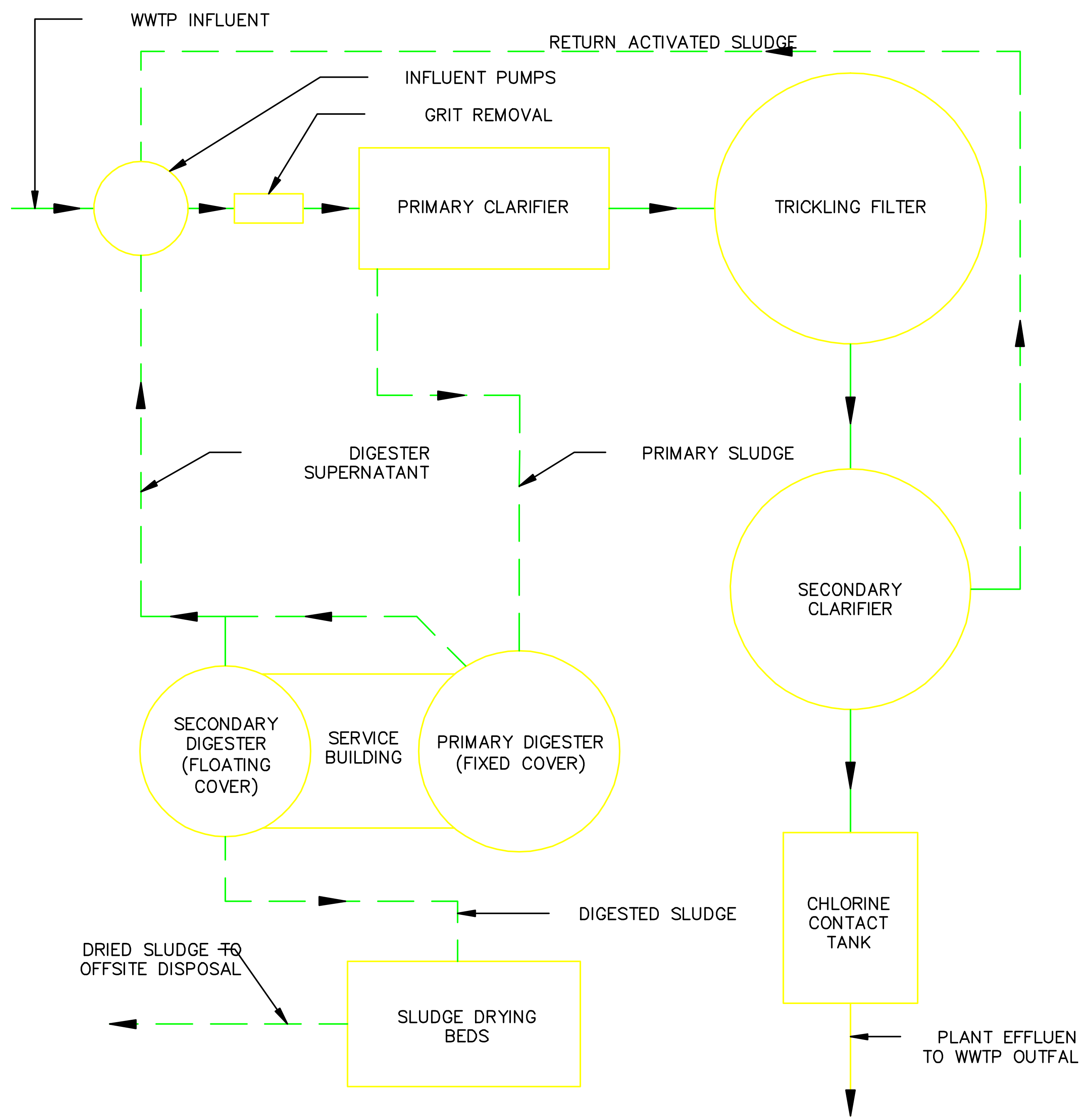
Figure No. **III.2**

Project No. **080911**



TOWN OF CANEADEA WWTP SCHEMATIC

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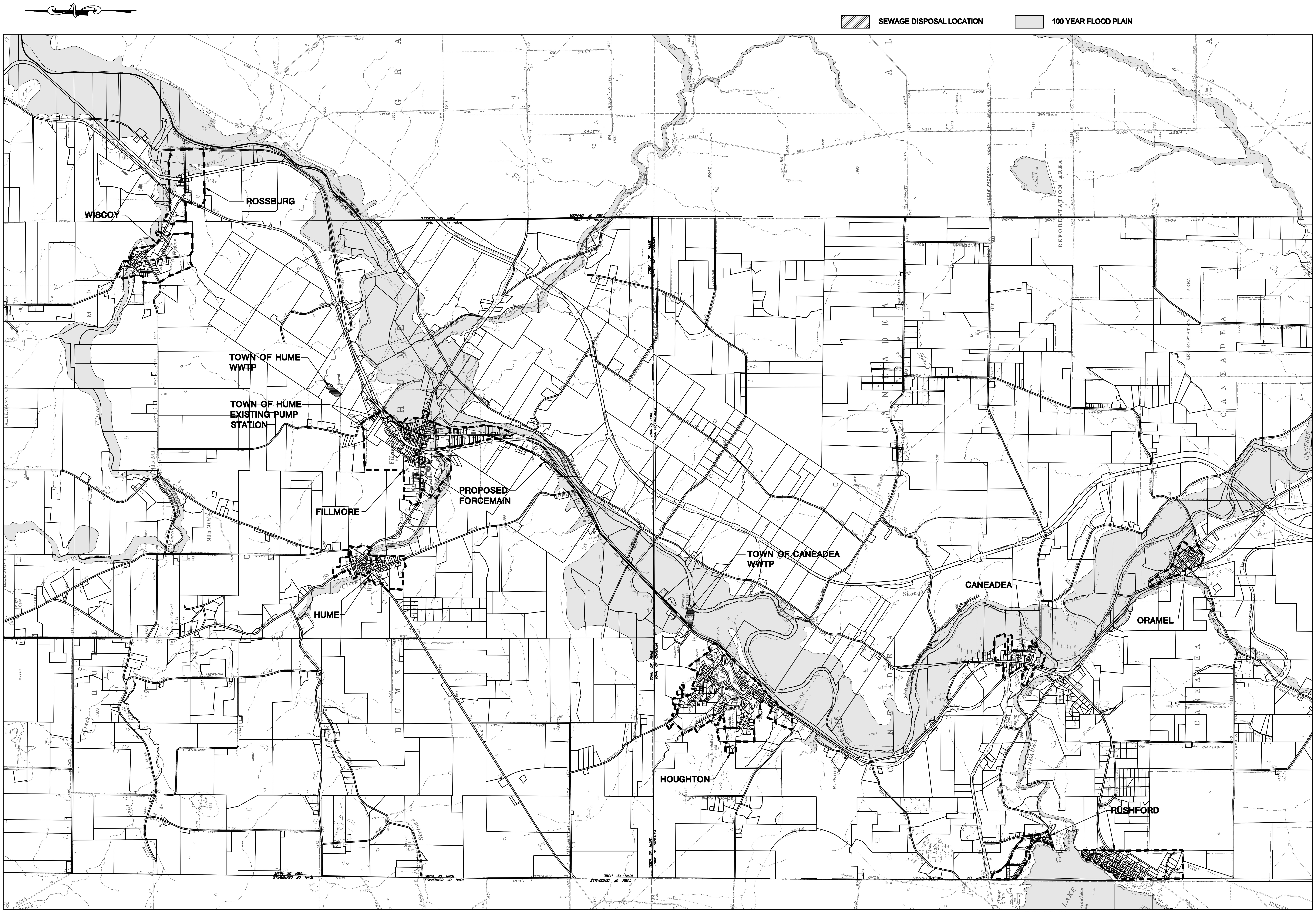
Figure No. **IV.1**

Project No. **080911**



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Project Title:
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Drawing Title:
CONCEPTUAL INTERCONNECTION

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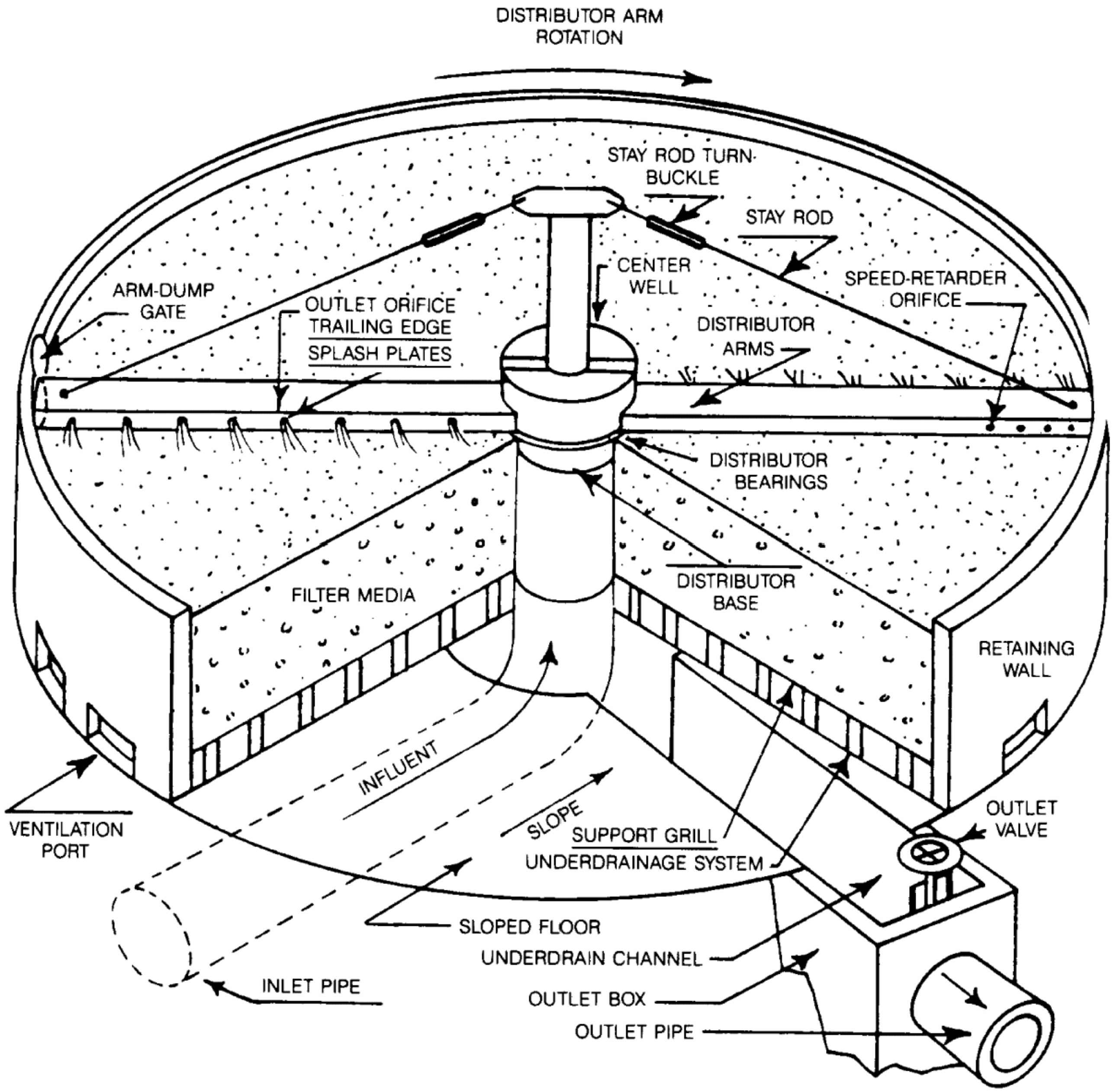
Figure No.
V.1

Project No.
080911



TRICKLING FILTER SCHEMATIC

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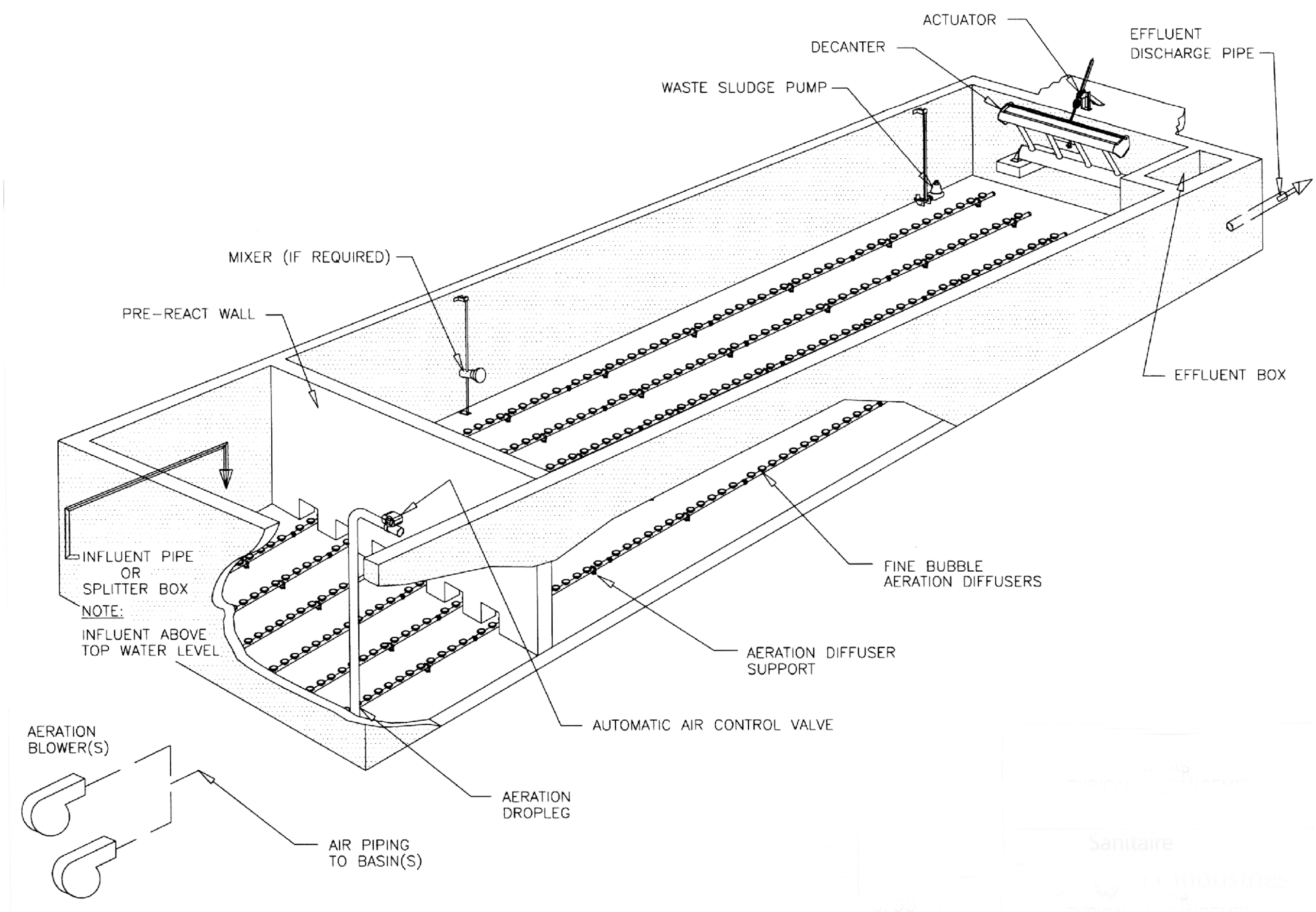
Date: 6/08	Distribution:				



SBR SCHEMATIC

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Figure No. **V.3**

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Drawing Title:
CANEADEA WWTP IMPROVEMENTS - ALT A1

Drawn By: JJT
Checked By:
Scale: 1" = 60'
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Figure No.
V.4

Project No.
080911

Project No. 08091

V.5

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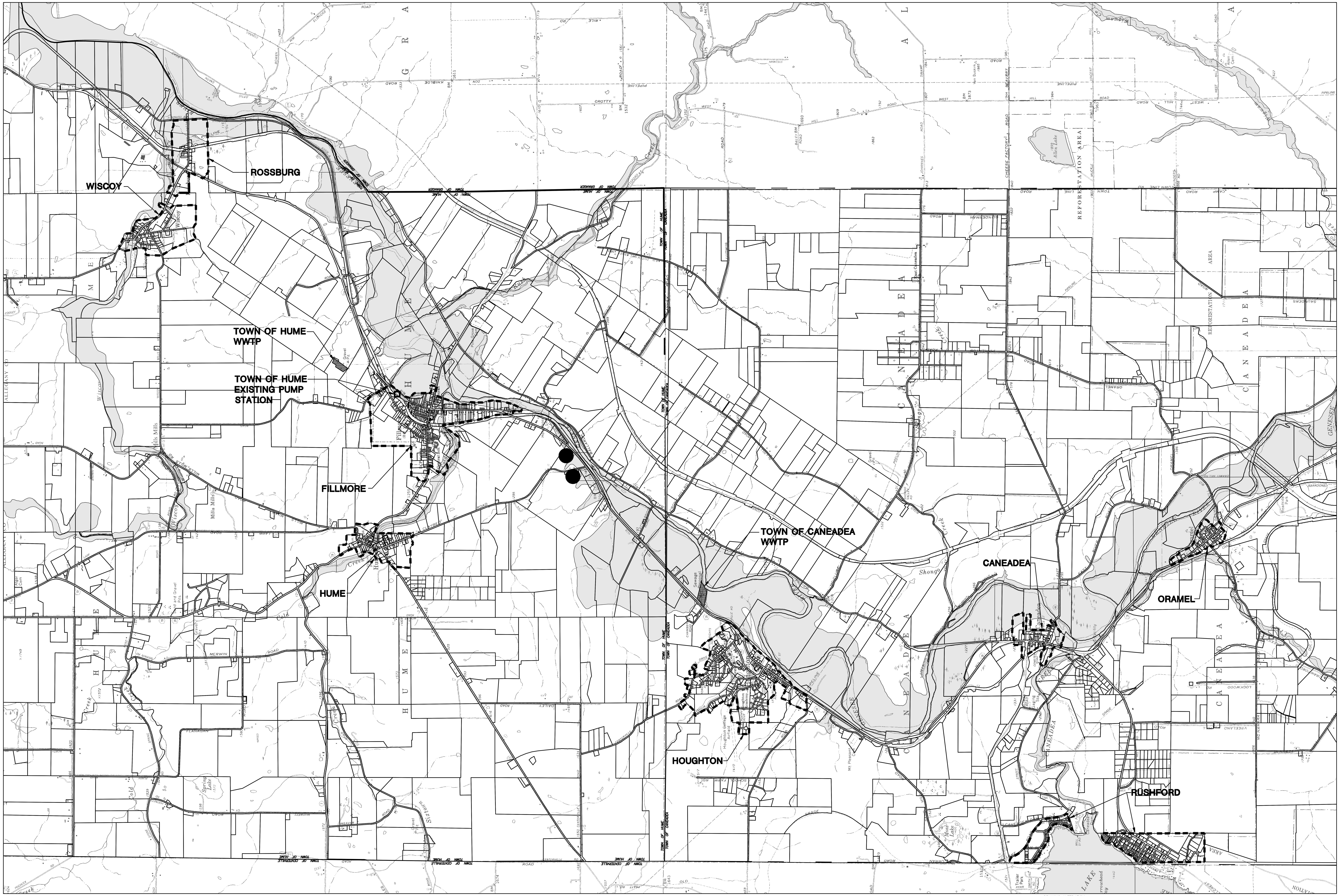
Figure No. **V.6**
Project No. **08091**

EXISTING SEWAGE DISPOSAL LOCATION

100 YEAR FLOOD PLAIN

CONCEPTUAL WWTP LOCATION



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Drawing Title:
CONCEPTUAL WWTP LOCATIONS

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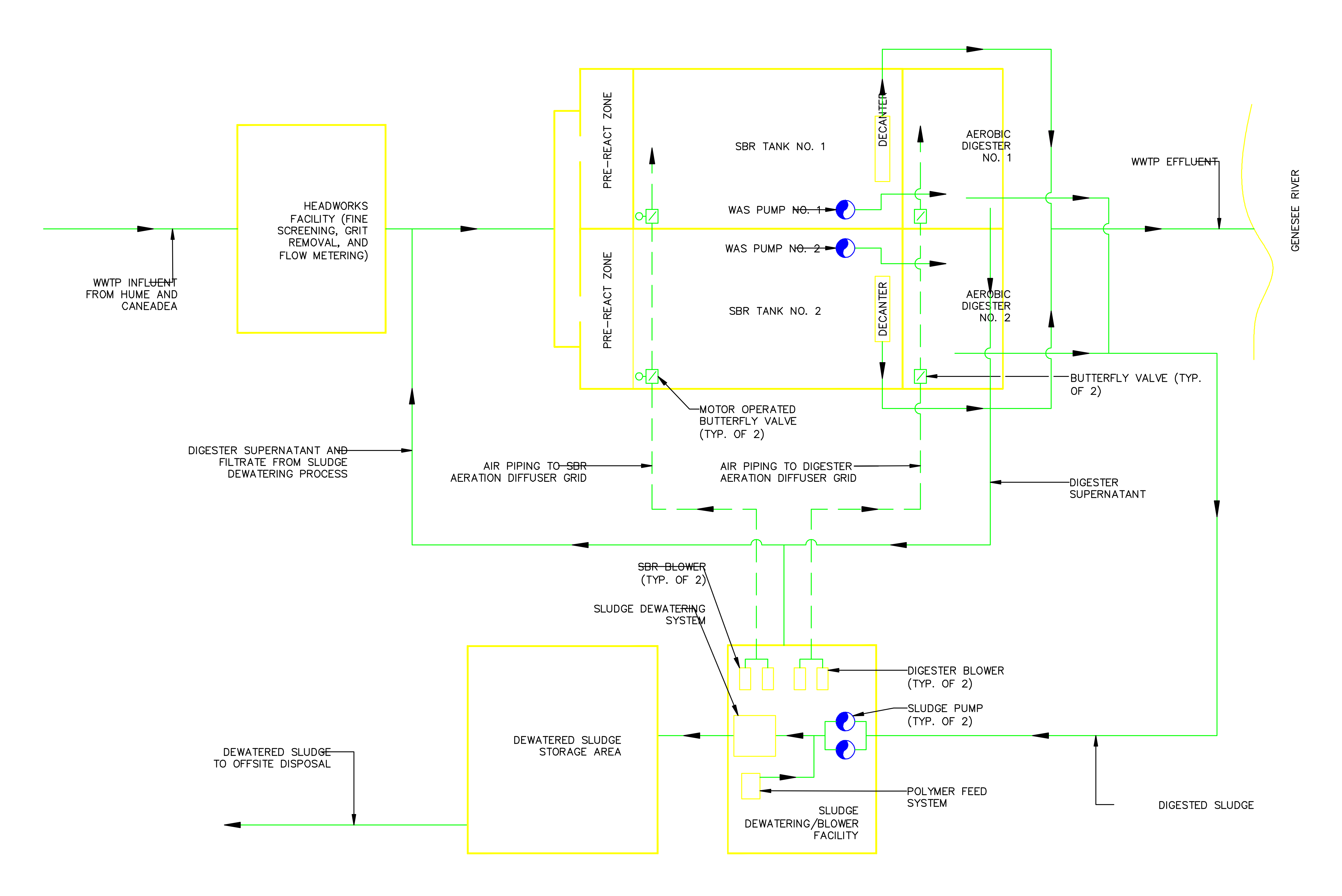
Figure No.
V.7

Project No.
080911



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Drawing Title:
CANEADEA WWTP IMPROVEMENTS - ALT A4

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Figure No.
V.8

Project No.
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