WATER POLLUTION CONTROL PLANT AND SAUQUOIT CREEK PUMP STATION EVALUATION

Prepared for

Oneida County Department of Water Quality & Water Pollution Control

Steven P. Devan, P.E., Commissioner
51 Leland Avenue
Utica, NY 13502

August 2012

Prepared by

Utica, NY | Cazenovia, NY | Liverpool, NY

ONEIDA COUNTY DEPARTMENT OF WATER QUALITY & WATER POLLUTION CONTROL
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Anthony J. Picente, Jr.
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August 27, 2012

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HAND DELIVERY

Koon Tang, P.E., Director
Bureau of Water Permits
Division of Water
NYS Department of Environmental Conservation
625 Broadway, 4th Floor
Albany, NY 12233

Re: Oneida County Sewer District
Water Pollution Control Plant and Sauquoit Creek Pump Station Evaluation

Consent Order No. R6-20060823-67

Dear Mr. Townsend and Mr. Tang:

We are pleased to submit herewith three (3) copies of the Water Pollution Control Plant (WPCP) and Sauquoit Creek Pump Station (SCPS) Evaluation for your Watertown office and one (1) additional copy for your Albany office. These reports are being submitted for your review and approval in accordance with NYSDEC Consent Order No. R620060823-67 (Schedule A, Paragraphs 4 and 5).

Based on preliminary estimates of required sanitary and combined sewer overflow (SSO and CSO) mitigation, the WPCP may have to increase its peak capacity from approximately 55 million gallons per day (mgd) to approximately 111 mgd. A critical component to evaluating alternatives for upgrading the WPCP to accept and treat additional flows will be the discharge limits the NYSDEC will impose on the County. To date, these limits are not defined. However, the NYSDEC has indicated that in the future, more stringent discharge limits for nitrogen and phosphorus may occur. There is concern regarding the financial commitment being made today by the County and the City of Utica when the future discharge permits are not known. After a review meeting with the NYSDEC on May 21st, the NYSDEC provided a written comment stating:

> *"The Department is currently developing new water quality criteria for nutrients which are expected to be finalized within the next few years and impact the SPDES permit. While these requirements are not yet known a preliminary assessment suggests that phosphorus reduction will likely be necessary and that total nitrogen reduction will likely not be necessary."*

The Engineering Team developed probable costs associated with upgrading the SCPS and WPCP to treat a peak flow of 111 mgd. The estimated project cost is $138,000,000 for work at the SCPS and WPCP alone.

The project represents a substantial financial burden on the rate payers in the Oneida County Sewer District. If the NYSDEC imposes enhanced nitrogen or phosphorus removal limits, the costs will increase dramatically. Further, the possibility exists that several components of the SCPS and WPCP upgrade could be constructed to meet the Consent Order but prior to the final determination of nitrogen and phosphorus limits, in which case the District would have to spend additional funds to upgrade recently constructed facilities that can no longer meet the more stringent permit requirements. Due to the financial burden associated with this work, the District will require funding assistance.

We respectfully request that the NYSDEC accelerate the determination of nitrogen and phosphorus discharge limits to the Mohawk River by confirming that total nitrogen reduction will not be necessary and determining the extent to which phosphorus reduction will be necessary. We also request determination of any seasonal ammonia limits that may be imposed. We feel if the limits are known prior to designing and constructing the pump station and WPCP upgrades, then the potential costly impacts of a secondary upgrade to the new facilities immediately after construction would be reduced. Better determination of regulations related to nitrogen and phosphorus limits at this time would be most beneficial to the overall project and would assist in the determination of funding requirements.

Please feel free to contact me should you have any questions or need additional information.

Sincerely,

**THE ONEIDA COUNTY DEPARTMENT OF
WATER QUALITY & WATER POLLUTION CONTROL**

Steven P. Devan, P.E.
Commissioner

cc: Anthony J. Picente, Jr. – Oneida County Executive
Robert Palmieri – Mayor, City of Utica
Deborah Day – City of Utica
Karl E. Schrantz, P.E. – Shumaker Engineering
Howard B. LaFever, P.E. – GHD
Mark Allenwood, P.E. – Brown & Caldwell
Peter M. Rayhill, Esq. – Martin and Rayhill
Judy Drabicki – NYSDEC
Joseph DiMura, P.E. – NYSDEC
Matthew Duffany, P.E. – NYSDEC
James Stearns, P.E. – NYSEFC

WATER POLLUTION CONTROL PLANT AND SAUQUOIT CREEK PUMP STATION EVALUATION

Prepared for

ONEIDA COUNTY, NEW YORK

AUGUST 2012

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WATER POLLUTION CONTROL PLANT AND SAUQUOIT CREEK PUMP STATION EVALUATION

Prepared for

ONEIDA COUNTY, NEW YORK

AUGUST 2012

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1.0 EXECUTIVE SUMMARY

The Oneida County Sewer District (District) is administered through the Oneida County Department of Water Quality and Water Pollution Control (WQ&WPC) which is responsible for the operation and management of the District's facilities and personnel. District facilities include 45 miles of interceptor sewers, the Sauquoit Creek and the Barnes Avenue Pumping Station, and the Water Pollution Control Plant (WPCP). The District services 15 municipalities including the City of Utica.

The WPCP is a regional facility that treats wastewater from the City of Utica, 14 municipalities, and the Oneida County Business Park. Wastewater from regions outside than the City of Utica includes only sanitary sewage. Wastewater from the City of Utica is combined sewage. The sewer systems outside the City of Utica are separate sanitary sewers. The WPCP is designed and operated to accept sanitary sewage, infiltration and inflow, and some combined sewer overflow (CSO) flows. It is standard practice to use available WPCP hydraulic capacity to treat extraneous infiltration and inflow and combined sewage. The WPCP staff currently adjusts operations to treat as much combined sewage from the City of Utica as possible. When the combined sewage from the City of Utica exceeds the available hydraulic capacity of the WPCP, some storage is provided in the interceptor before this excess flow is diverted to a permitted outfall.

The NYSDEC and Oneida County (County) entered into Consent Order No. R620060823-67 due to sanitary sewer overflow (SSO) at the Sauquoit Creek Pumping Station (SCPS). The Consent Order has an effective date of December 12, 2011 and requires mitigation of the SSO at the SCPS.

In addition to the Consent Order with the County, the NYSDEC has required a combined sewer overflow long term control plan (LTCP) as part of the City of Utica's SPDES permit. The LTCP requires the City to increase its percent capture of CSO flows during wet weather.

As a result of the County's Consent Order to mitigate SSO at the SCPS, and the City's LTCP to increase the capture of CSO flows, the WPCP will be required to accept and treat flows beyond its existing capacity. The WPCP can currently process a peak flow of approximately 55 million gallons per day (mgd). Based on preliminary CSO/SSO mitigation requirements as well as projected growth within the District, the WPCP may need to be expanded for a peak capacity of 111 mgd.

1.1 EVALUATION OF ALTERNATIVES

Several alternatives were evaluated to expand the WPCP to a capacity of 111 mgd. These alternatives included:

- Conventional WPCP expansion
- "Split Flow" wet weather operating strategy
- Aeration operation modifications
- Integrated Fixed Film Activated Sludge (IFAS)
- High rate ballasted flocculation
- Solids handling alternatives, including gravity thickening, and belt filter press and centrifuge dewatering
- Solids disposal alternatives, including incineration, anaerobic digestion, and lime stabilization

Based on a detailed evaluation of the alternatives, the most cost effective method for increasing the capacity of the WPCP is through the split flow operating strategy. In this scenario, improvements to the headworks of the WPCP will be made to maintain dedicated treatment trains for flows from the combined sewers in the City of Utica and the sanitary sewers in the rest of the district. During dry weather, all flows will receive screening, grit removal, primary sedimentation, secondary treatment, and disinfection. During wet weather, combined flows from the City of Utica will receive screening, grit removal, primary sedimentation, and high rate disinfection. Sanitary flows will continue receive secondary treatment.

The split flow alternative requires the construction of a new screening facility and pump station for sanitary flows. Combined flows will be conveyed through the existing screening facility and raw waste pump station. New vortex grit facilities will be constructed for all flows, and rectangular primary clarifiers will replace the existing circular units. A new high-rate disinfection facility will be constructed for combined flows.

The existing sludge processing facilities would also require modifications to process peak sludge flows and loads. The most economical approach for upgrading new sludge facilities includes placing a currently idle thickener back into service. Consideration can be given to replacing existing belt filter presses with centrifuges to produce a dryer sludge cake.

The WPCP, which currently incinerates its sludge in two (2) operational fluidized bed incinerators, is subject to regulations recently issued by the USEPA for sanitary sewer incinerators (SSI). Continued operation of the incinerators with modifications for SSI emissions limits was evaluated versus converting from incineration to anaerobic digestion or lime stabilization. On a net present worth basis, the most cost effective approach for solids disposal includes rehabilitating two (2) of the existing incinerators for compliance with SSI regulations, and installing a backup lime stabilization system in place of the third incinerator.

In addition to the improvements necessary to accept and treat future peak flows and loads, several upgrades are required at the WPCP to ensure long-term viability. Based on a physical condition assessment of the WPCP, numerous improvements are necessary to the processes which will remain in service after the WPCP expansion. These improvements are related to the existing structural and architectural condition of buildings and tanks, the condition of existing operating equipment, and the condition of the overall site. In addition, upgrades to the existing WPCP electrical distribution system and emergency power capacity will be required to replace aged equipment and to support electrical loads associated with the WPCP expansion.

Besides the expansion at the WPCP, the SCPS will need to be expanded to pump flows associated with SSO mitigation. Expansion at the SCPS would include new screens, and a new discharge forcemain to the WPCP.

1.2 COST OF RECOMMENDED ALTERNATIVE

The probable project cost, in 2012 dollars, to expand the WPCP to a peak capacity of 111 mgd, upgrade the physical condition of the WPCP, and upgrade the SCPS and provide a new discharge forcemain, is $138,000,000. This cost includes construction costs, as well as fiscal, legal, administrative, engineering, and contingencies. There are six (6) major components to this overall project cost as shown in Table 1-1.

TABLE 1-1

ENGINEER'S OPINION OF PROBABLE PROJECT COST

PROJECT COMPONENT	PROBABLE COST (1) (2)
New Primary Settling Tanks	$22,000,000
Increase Capacity of WPCP and Split Flow	$26,000,000
SCPS Upgrades and Forcemain	$22,000,000
WPCP Physical Condition Upgrades (Architectural, Structural, Mechanical, Civil/Site)	$34,000,000
Solids Handling and Incinerator Upgrades	$26,000,000
Electrical Upgrades	$8,000,000
Total (Rounded)	**$138,000,000**

(1) Year 2012 dollars, rounded

(2) Includes 20% contingency, and 20% engineering, administrative, and legal costs

1.3 SCHEDULE

Based on the Consent Order, the WPCP expansion is required to be operational by December 31, 2021. By this date, all new facilities must be constructed including the SCPS modifications and the new forcemain.

2.0 BACKGROUND AND PURPOSE

2.1 ONEIDA COUNTY SEWER DISTRICT

The Oneida County Sewer District (District) was formed in 1965 through an act by the former Oneida County Board of Supervisors. It is administered by Oneida County through the Oneida County Department of Water Quality and Water Pollution Control (WQ&WPC) which is responsible for the operation and management of the District's facilities and personnel. District facilities include 45 miles of interceptor sewers, the Sauquoit Creek and the Barnes Avenue Pumping Station, and the Water Pollution Control Plant (WPCP). The District services 15 municipalities. These municipalities own and operate their own collection systems. The member municipalities of the District are listed in Table 2-1.

TABLE 2-1

ONEIDA COUNTY SEWER DISTRICT MEMBER COMMUNITIES

Village of Clayville	Village of Holland Patent	Town of Paris	Town of Frankfort
Village of New Hartford	Village of Whitesboro	Town of Marcy	Town of Whitestown
Village of New York Mills	Village of Yorkville	Town of Deerfield	City of Utica
Village of Oriskany	Town of New Hartford	Town of Schuyler	Oneida County Business Park

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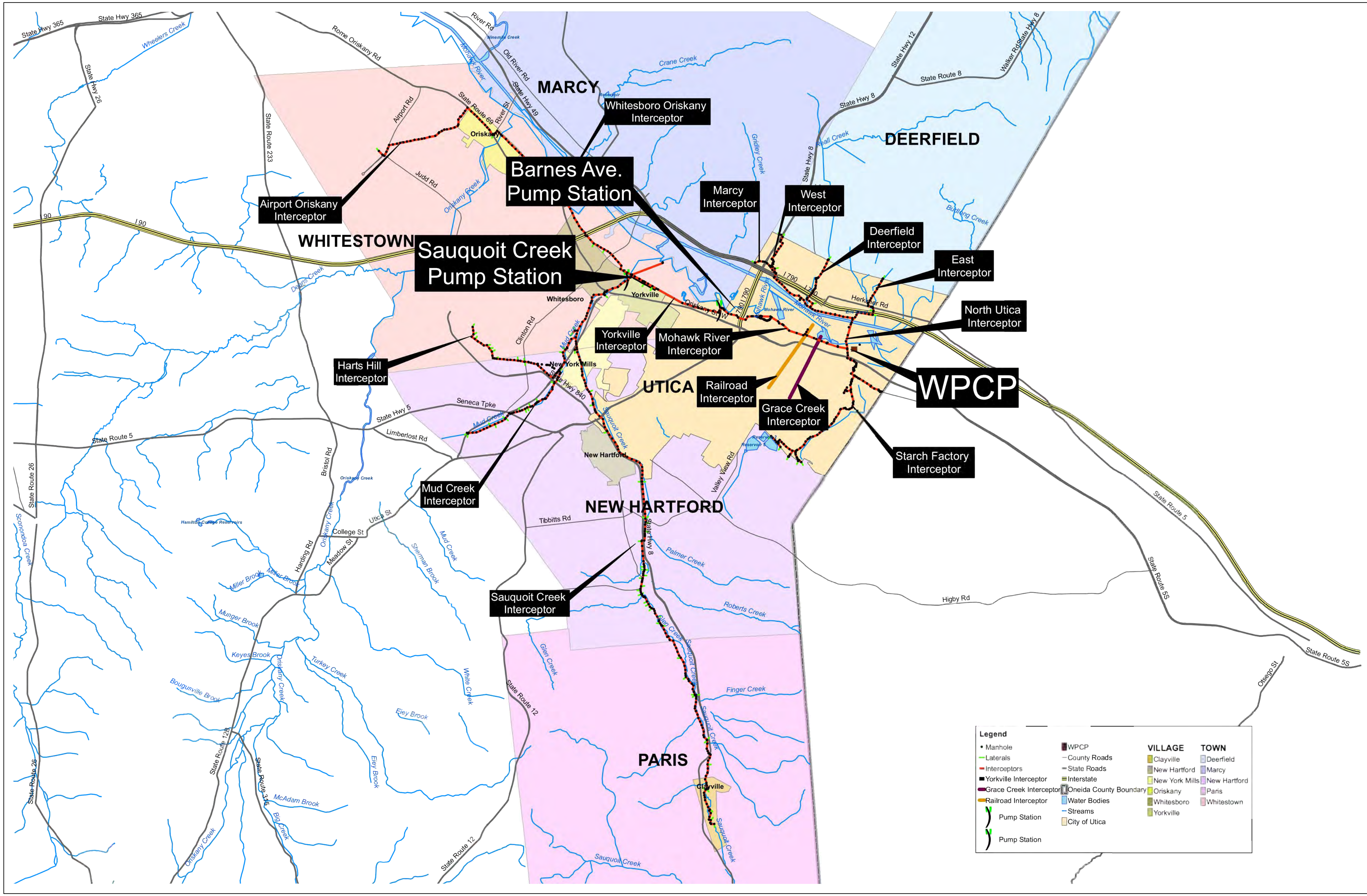
The District currently services a population of approximately 106,000 people and covers an area of approximately 170 square miles. A general map of the District is provided on Figure 2-1.



SCALE: NOT TO SCALE

N

Shumaker Consulting Engineering & Land Surveying, P.C.

Brown AND Caldwell

GHD

WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

ONEIDA COUNTY
SEWER DISTRICT MAP

Job Number	8614782
Revision	A
Date	06/12

Figure 2-1

200 John James Audubon Parkway Suite 101, Amherst NY 14228 USA T 1 716 691 8503 F 1 716 691 8506 E amhmail@ghd.com W www.ghd.com

Plot Date: 22 August 2012 - 9:39 AM Cad File No: G:\86\14782\CADD\Drawings\Figures\ocsd collection system.dwg

2.2 ONEIDA COUNTY WPCP

The WPCP is a regional facility that treats wastewater from the municipalities listed in Table 2-1 and Oneida County Business Park. Wastewater from regions other than the City of Utica includes sanitary sewage and extraneous infiltration and inflow (I/I). Wastewater from the City of Utica is combined sewage. The WPCP is designed and operated to accept sanitary sewage, infiltration and inflow, and some CSO flows. It is standard practice to use any available WPCP hydraulic capacity to treat a portion of the extraneous infiltration and inflow and combined sewage. The WPCP staff currently adjusts operations to treat as much combined sewage from the City of Utica as possible. When the combined sewage from the City of Utica exceeds the available hydraulic capacity of the WPCP, some storage is provided in the interceptor before this excess flow is diverted to a permitted outfall.

Flow is conveyed to the WPCP through a series of large diameter interceptor sewers. The major interceptors which discharge at or near the WPCP include:

- The 60-inch diameter Mohawk River Interceptor which conveys combined flow from the City of Utica. Sanitary flow from the Sauquoit Creek Pump Station (SCPS) and Barnes Avenue Pump Station (BAPS) also discharge to the Mohawk River Interceptor upstream of the WPCP. The Mohawk River Interceptor discharges at the bar screen facility at the WPCP.
- The 42-inch diameter North of Utica Interceptor which conveys sanitary flow from portions of the District north of the Mohawk River. The North of Utica Interceptor basin includes portions of the Town of Marcy, Town of Deerfield, and Town of Schuyler. Like the Mohawk River Interceptor, the North of Utica Interceptor discharges to the bar screen facility at the WPCP.
- The 36-inch diameter Starch Factory Creek Interceptor, which conveys sanitary flow from portions of the Town of Paris and Town of New Hartford. The Starch Factory Creek

Interceptor discharges to the Mohawk River Interceptor on the grounds of the WPCP, immediately upstream of the bar screen facility.

The WPCP operates under the limits set forth in a state pollution discharge elimination system (SPDES) permit, regulated by the New York State Department of Environmental Conservation (NYSDEC). The SPDES permit is discussed in greater detail in Section 5.1 of this Report. The CSO discharges on the Mohawk River Interceptor are unpermitted unless the WPCP can provide treatment for flows of at least 53 million gallons per day (mgd) during winter months (November through May), and at least 48 mgd during summer months (June through October). Current plant operations restrict flow to 55 mgd during wet weather. There are hydraulic restrictions within the WPCP which do not allow for the conveyance of more than 55 mgd through the plant.

In addition to wet weather CSO discharges from the Mohawk River Interceptor, extraneous I/I in the SCPS basin periodically cause an SSO at the pump station. SSO from the SCPS is discharged directly to the Mohawk River.

2.3 REPORT PURPOSE

The NYSDEC and Oneida County (County) entered into Consent Order No. R620060823-67 due to SSO at the Sauquoit Creek Pumping Station. The Consent Order has an effective date of December 12, 2011 and requires mitigation of the SSO at the SCPS.

In addition to the Consent Order with the County, the NYSDEC has required a combined sewer overflow long term control plan (LTCP) as part of the City of Utica’s SPDES permit. The LTCP requires the City to increase its percent capture of CSO flows during wet weather.

As a result of the County’s Consent Order to mitigate SSO at the SCPS, and the City’s LTCP to increase the capture of CSO flows, the WPCP will be required to accept and treat flows beyond its existing capacity.

The Consent Order requires the submission of two engineering reports by August 31, 2012:

- WPCP Evaluation Report for evaluating the expansion of the WPCP (Consent Order Schedule A, Item No. 5)
- SCPS Evaluation Report for evaluating the pumping capacity of the SCPS (Consent Order Schedule A, Item No. 4)

The Consent Order provides the option for submitting these two (2) documents as one report. Accordingly, this report fulfills the Consent Order requirement for the submission of these two (2) required documents.

3.0 EXISTING AND PROJECTED FUTURE FLOWS AND LOADS

3.1 EXISTING FLOWS AND LOADS

The District provided WPCP operational data for 2005 – 2011. Based on these data, a summary of the existing influent flows and loads are presented in Table 3-1. The data in Table 3-1 include:

- Average and maximum 30-day flows
- Peak hourly flows
- Average and maximum 30-day loads for biochemical oxygen demand (BOD), chemical oxygen demand (COD), total suspended solids (TSS), ammonia (NH_3), and total Kjeldahl nitrogen (TKN)

These data are based on measured flows and loads received at the WPCP during wet and dry events. Although pollutant concentrations are not currently measured in CSO and SSO discharges, this report presents an approximation of the CSO and SSO discharge characteristics based on flows measured during the CSO/SSO study and water quality data measured at the WPCP.

TABLE 3-1

EXISTING (2005 – 2011) INFLUENT FLOWS AND LOADS

Parameter		Unit	Summer (June – October)	Winter (November – May)
Flow	Average (1)	mgd	30	42
	Max 30-Day	mgd	48	54
	Peak (3)	mgd	55	55
BOD	Average (1)	mg/L	123	89
		lbs/day	30,600	31,200
	Max 30-Day (2)	mg/L	102	111
		lbs/day	41,000	49,900
COD	Average (1)	mg/L	244	193
		lbs/day	68,500	70,800
	Max 30-Day (2)	mg/L	244	238
		lbs/day	97,700	107,400
TSS	Average (1)	mg/L	90	62
		lbs/day	22,300	21,600
	Max 30-Day (2)	mg/L	71	70
		lbs/day	28,400	31,700
NH_3	Average (1)	mg/L	8	4
		lbs/day	1,900	1,500
	Max 30-Day (2)	mg/L	9	10
		lbs/day	3,600	4,500
TKN	Average (1)	mg/L	15	12
		lbs/day	3,800	3,900
	Max 30-Day (2)	mg/L	5	5
		lbs/day	4,400	5,000

(1) Geometric Mean
(2) Maximum 30-Day Concentration Based on Maximum 30-Day Load at Maximum 30-Day Flow
(3) Existing peak flow of 55 mgd is the result of restricting the influent flow due to hydraulic limitations within the WPCP

3.2 POPULATION GROWTH PROJECTIONS

The project team met with the Oneida County Planning Department in August 2011 to review existing population and population growth projections within the District. From 1990 to 2010, the population of Oneida County decreased from 250,836 to 224,536, representing a 10% population decrease over 20 years. Representatives from the Planning Department do not foresee any significant residential growth within the District in the near future.

The Planning Department referred to population growth projections available from the Cornell University Program for Applied Demographics. These projections predict a similar decreasing trend as shown in recent census data. Figure 3-1 graphically depicts recent census data (through the year 2010), along with population projections to the year 2035.

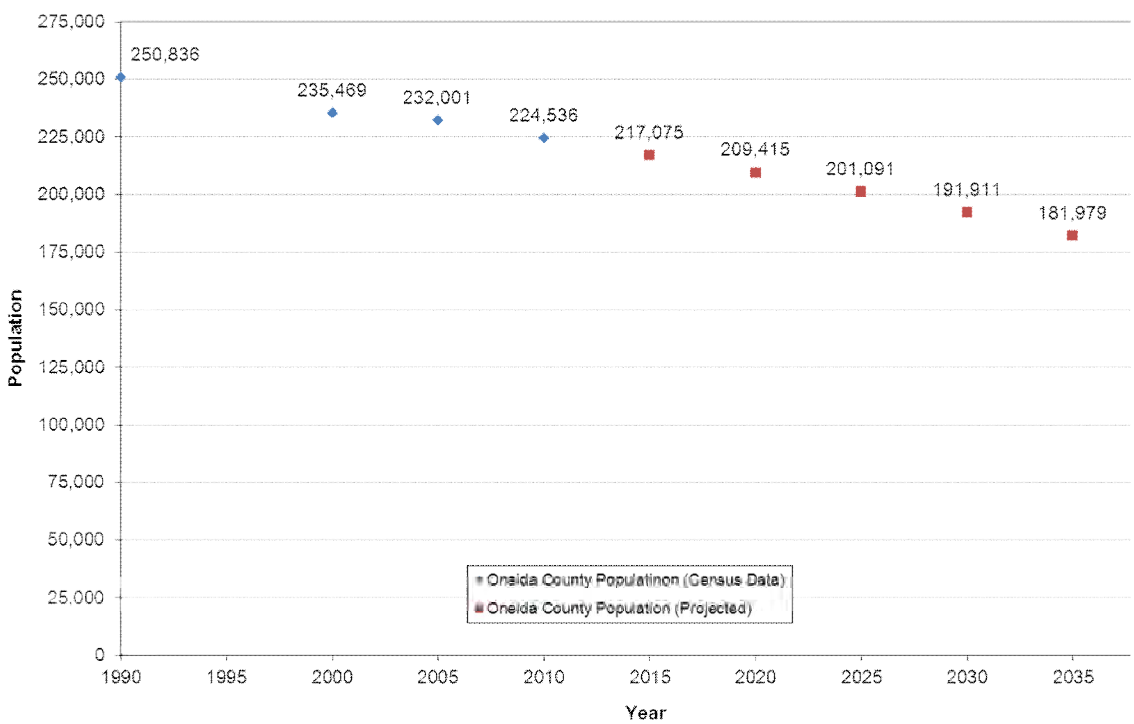


FIGURE 3-1

ONEIDA COUNTY POPULATION

3.3 INDUSTRIAL GROWTH PROJECTIONS

There is greater potential for industrial growth within the District than residential growth. The Mohawk Valley Economic Development Growth Enterprise Corporation (EDGE) has been actively developing and marketing a 428 acre industrial site in the Town of Marcy, known as the Marcy Nanocenter at SUNYIT. The EDGE is making significant investments in infrastructure at the site, including roads, water, sewer, and electric service. The goal is to make the site as shovel ready as possible, to attract a large computer microchip manufacturing plant. Based on discussions with the Planning Department, the potential microchip plant represents the only significant development within the District that would contribute additional flows. Effluent from the microchip manufacturing plant, along with residential and commercial "spin-off" growth which could occur as a result of the plant, represent the only potentially significant new flow sources to the District.

The microchip manufacturing process is closely guarded by the companies which make microchips. Based on discussions with EDGE, the effluent flow from a microchip manufacturing plant could be approximately 6.0 mgd. Residential and commercial spin-off growth is more difficult to estimate. For the purposes of this report, the spin-off flow is projected to be approximately 50% of the microchip plant effluent, or 3.0 mgd, which would include residential and commercial development. The total new flow to the District as a result of the microchip plant effluent and associated spin-off growth is 9.0 mgd. When considering future peak flows to the WPCP, this report will evaluate the microchip plant with and without water saving technology, which can significantly reduce flow.

The effluent flow from the microchip manufacturing plant would not vary significantly by season. Since the site is located in Marcy, the flows would be conveyed through the North of Utica Interceptor to the WPCP.

3.4 PROJECTED CSO AND SSO CAPTURE

In addition to the new flows and loads associated with growth within the District, the project team also evaluated the flows and loads which the WPCP may be required to treat based on additional CSO capture in the City of Utica and SSO capture in the Sauquoit Creek basin.

3.4.1 Projected CSO Capture from the City of Utica

The City of Utica is required to increase its annual percent capture of CSO flows to 85% (by volume) in accordance with their LTCP. Under existing conditions, the wet weather flow to the District's WPCP from the City's combined system is approximately 28 mgd. At this rate, the City currently captures approximately 68% of their annual CSO volume. Based on a calibrated and validated City of Utica collection system model, the percent capture versus wet weather flow to the District's WPCP is presented in Table 3-2 and shown graphically on Figure 3-2.

TABLE 3-2

CITY OF UTICA FLOW VS. PERCENT CAPTURE

WET WEATHER FLOW FROM CITY TO DISTRICT (MGD)	INCREASE TO EXISTING FLOW (MGD)	ANNUAL PERCENT CAPTURE
28	0	68%
38	10	76%
48	20	84%
58	30	87%
68	40	90%

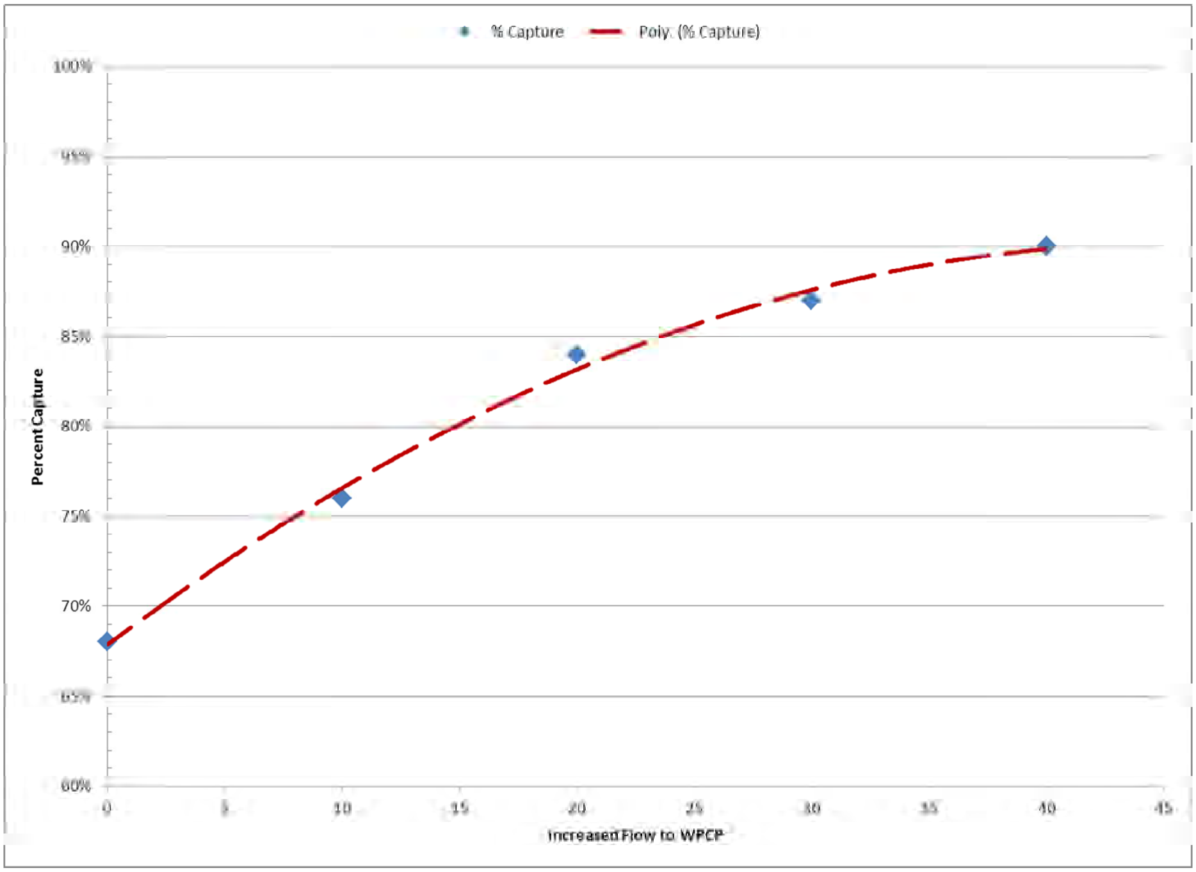


FIGURE 3-2
CITY OF UTICA FLOW VS. PERCENT CAPTURE

The City's collection system model suggests 85% capture can be achieved if the peak flow from the City to the District's WPCP were to increase by approximately 21 mgd, or to 49 mgd. As shown on Figure 3-2, the percent capture increases at a lower rate once the City increases its peak flow in excess of 21 mgd above existing conditions.

3.4.2 Projected SSO Capture from the Sauquoit Creek Basin

The peak flow of the Sauquoit Creek Pump Station (SCPS) is currently approximately 15.0 mgd. The peak capacity of 15.0 mgd represents the peak capacity with the largest unit out of service, as required in the Ten States Standards. This pumping capacity is not adequate to mitigate SSO events at the station.

As with the City of Utica, a calibrated and validated collection system model was developed for the Sauquoit Creek basin including the SCPS. Based on these modeling efforts, the annual SSO volume from the SCPS is approximately 102 million gallons per year. The SSO could be mitigated by increasing the capacity of the station. Table 3-3 summarizes the projected annual SSO volume at various levels of increased SCPS capacity.

TABLE 3-3

SCPS FLOW VS. ANNUAL SSO VOLUME

SCENARIO	PEAK CAPACITY OF SCPS (MGD)	ANNUAL SSO VOLUME (MG)
Existing Conditions	15	102
Scenario 1	20	37
Scenario 2	25	9
Scenario 3	30	1
Scenario 4	35	< 1
Scenario 5	38	0

If the pumping capacity of the SCPS were increased to 35 mgd, or 20 mgd above existing conditions, the model predicts the SSO at the SCPS would be reduced to less than 1 MG per year. This is assuming no I/I reduction as a result of sewer rehabilitation, which is a very conservative assumption as some sewer rehabilitation has already occurred and significant rehabilitation is planned for the future. With a modest amount of I/I removal in the SCPS basin, the annual SSO volume at the pump station is expected to be mitigated if the peak capacity of the station were increased to 35 mgd.

3.5 PROJECTED WPCP FLOW

Future flows to the WPCP were estimated for:

- Average daily flow
- Maximum 30-day flow

- Peak hourly flow

The future average flow is based on the existing average flow, plus anticipated growth within the District. The future maximum 30-day flow is based on existing maximum 30-day flows, plus anticipated growth within the District and likely maximum 30-day CSO and SSO discharges which may be conveyed to the WPCP in the future. The future peak hourly flow is based on anticipated growth within the District, plus peak CSO and SSO volumes which may be conveyed to the WPCP for the County to comply with the Consent Order and the City to comply with their LTCP.

3.5.1 Projected Daily Average Flow to the WPCP

The only anticipated growth within the District is the potential microchip fabrication plant and associated residential and commercial spin-off. On an average basis, existing flows to the WPCP are not expected to increase beyond what may be attributable to the microchip plant development. Based on this potential growth source, the future average WPCP flows are presented in Table 3-4.

TABLE 3-4
FUTURE DAILY AVERAGE FLOW

	SUMMER (JUNE – OCTOBER)	WINTER (NOVEMBER – MAY)
Existing Average Flow (mgd)	30	42
Microchip Plant Effluent and Spin-Off (mgd)	9	9
Future Average Flow (mgd)	**39**	**51**

3.5.2 Projected Maximum 30-Day Average Flow to the WPCP

Since the WPCP currently limits peak flow to 55 mgd, the current maximum 30-day average flow is similar to the peak hourly flow (54 mgd vs. 55 mgd, respectively). The quantity of CSO and

SSO to the river is not metered, so the true maximum 30-day average flow in the system is not known.

To estimate the future maximum 30-day average sanitary flow, industry standard peaking factors were utilized. A typical peaking factor for maximum 30-day average flow is 1.2 times the daily average flow for a sanitary system[1]. This peaking factor was applied to the future average sanitary flow from the existing District to obtain the maximum 30-day average sanitary flow in the existing system. On top of this, an extra 9 mgd was added to account for the potential future flows associated with the microchip plant and associated spin-off growth to obtain the future maximum 30-day average sanitary flow to the WPCP. These flows are summarized in Table 3-5. The values in Table 3-5 represent the maximum 30-day average sanitary flow to the WPCP. The actual maximum 30-day average total flow to the WPCP may be higher depending on rainfall characteristics in the City of Utica basin.

TABLE 3-5

FUTURE MAXIMUM 30-DAY AVERAGE SANITARY FLOW

	SUMMER (JUNE – OCTOBER)	**WINTER (NOVEMBER – MAY)**
Future Maximum 30-Day Sanitary Flow (Non-Microchip Plant) (mgd) (1)	36	51
Microchip Plant Effluent and Spin-off Sanitary Flow (mgd)	9	9
Future Maximum 30-Day Average Sanitary Flow (mgd)	**45**	**60**

(1) Calculated as 1.2 times the daily average flow (rounded up).

3.5.3 Projected Peak Hourly Flow to the WPCP

Together with the District, the project team "bracketed" various levels of peak flow which may be required at the WPCP for the County to comply with their Consent Order and the City to comply with their LTCP. The bracketing also included future growth projections within the District.

Table 3-6 summarizes the peak flow to the WPCP under two (2) future scenarios.

[1] WEF MOP 8 Table 3-4, and Metcalf & Eddy Figure 3-8

TABLE 3-6

FUTURE PEAK FLOW TO WPCP

FLOW SOURCE	EXISTING CONDITIONS (MGD)	FUTURE SCENARIO 1 (MGD)	FUTURE SCENARIO 2 (MGD)
City of Utica	28	39	49
SCPS Basin (1)	15	28	35
North of Utica Basin	6	10	10
Microchip Fabrication Plant (2)	0	3	6
Microchip "Spin-Off" (2)	0	3	3
Starch Factory Creek Basin (1)	6	8	8
Total	**55**	**91**	**111**

(1) Currently conveyed through Mohawk River Interceptor

(2) To be conveyed through North of Utica Interceptor

The future peak flows developed for Scenario 1 represent some additional CSO and SSO capture. In this scenario, an additional 10 mgd of CSO capture would increase the City's annual percent capture to approximately 76%, and the City would be required to provide remote treatment at CSO locations to provide a total annual capture of 85%. An additional 10 mgd of pumping capacity at the SCPS would partially mitigate the SSO, thus requiring a significant amount of I/I reduction through sewer rehabilitation to fully mitigate the SSO. Scenario 1 assumes the microchip plant uses water saving technology to reduce their water and sewer needs.

The future peak flows in Scenario 2 provide the City of Utica with enough capacity to convey an annual percent capture of 85% to the WPCP without the construction of remote facilities. Also in this scenario, the SSO at the SCPS would be mitigated through a combination of pumping wet weather flows to the WPCP and reduction of I/I through sewer rehabilitation. This scenario assumes the full flow from the microchip plant. Subsequent sections of this Report will detail why 111 mgd is the likely maximum peak flow that can be received and treated on the site of the existing WPCP.

3.6 PROJECTED WPCP LOADS

Future loadings to the WPCP were developed using a similar methodology as future flows. For each new flow source (i.e. microchip plant effluent, spin-off development, CSO, and SSO), a likely load was estimated. These loads are summarized in Table 3-7, and represent the maximum loadings projected to be conveyed to the WPCP during wet weather conditions (i.e. when the peak flow is 111 mgd).

TABLE 3-7

FUTURE WPCP LOADS

LOAD SOURCE	BOD LOAD AT 111 MGD (LB/D)	TSS LOAD AT 111 MGD (LB/D)	TKN LOAD AT 111 MGD (LB/D)
Sanitary Flow from Non-City Portions of the District	40,560	28,090	4,875
Combined Flow from the City of Utica	25,520	21,270	3,400
Microchip Plant Effluent (1)	0	2,500	1,000
Michochip Plant Spin-Off Growth (2)	2,900	2,000	350
Future Peak WPCP Influent Load	**68,980**	**53,860**	**9,625**

(1) Based on concentrations per 2008 EDGE study at 6.0 mgd flow

(2) Based on similar concentrations as existing WPCP dry weather influent at 3.0 mgd flow

3.7 BASIS OF EVALUATION

The basis for the WPCP and SCPS evaluation contained herein includes the flows and loads developed in this Section. Additionally, this Report considers how the WPCP can comply with the discharge limits set forth in the existing SPDES permit. If the SPDES permit were to change between the time of this Report and construction of WPCP improvements, this evaluation may need to be modified to account for new regulatory requirements. Section 9 of this Report further details regulatory considerations which may be necessary to consider in the future, such as nitrogen and phosphorus removal.

Since the impact of future regulations cannot be defined at this time, this evaluation is based upon compliance with the effluent concentrations limits set forth in the existing SPDES permit, at the higher flows described in this Section. Based on these existing limits, the treatment standards to which this evaluation is based are presented in Table 3-8. This evaluation does not include enhanced nitrogen or phosphorus removal, as potential regulatory requirements for nitrogen or phosphorus removal are not defined at this time.

TABLE 3-8

BASIS OF EVALUATION

		SUMMER (JUNE – OCTOBER)	**WINTER (NOVEMBER – MAY)**
Daily Average Flow (mgd)		39	51
Peak Hourly Flow (mgd)		91 or 111	91 or 111
Effluent BOD (mg/L)	30-Day Average	N/A	30
	7-Day Average	N/A	45
Effluent CBOD (mg/L)	30-Day Average	25	N/A
	7-Day Average	40	N/A
Effluent TSS (mg/L)	30-Day Average	30	30
	7-Day Average	45	45
Effluent TKN (lb/d)	30-Day Average	1,120	N/A
	Daily Maximum	Monitor	N/A
Effluent Fecal Coliform Effluent (#/100 mL)	30-Day Average	200	N/A
	7-Day Average	400	N/A
Effluent Total Residual Chlorine (mg/L)	30-Day Average	Monitor	N/A
	Daily Maximum	0.1	N/A

4.0 DESCRIPTION AND CAPACITY OF EXISTING COLLECTION SYSTEM

The District owned and operated collection system facilities include 45 miles of interceptor sewers ranging in diameter from 12- to 66-inches, and two (2) pumping stations (Sauquoit Creek and Barnes Avenue). In addition to the wastewater transport and treatment services provide by the District to its member communities, the District also provides the disposal of hauled waste (i.e.: septage, landfill leachate, etc…) from other locations.

The District inspects, samples, and regulates discharges to the WPCP that have the potential to negatively impact the collection system, WPCP, operating facilities O&M personnel or the public. In addition, the District is responsible for the enforcement of the Oneida County Sewer Use Rules and Regulations as amended in Local Law No. 3 of 2008.

4.1 AGE OF COLLECTION SYSTEM

The age of the overall collection systems vary from new to over 100 years old. The District-owned infrastructure was constructed in the late 1960s through the mid-1970s. The older municipal sewers are generally located in the City of Utica and within the Villages' tributary to the SCPS. The sewers outside these areas generally range from new to 60 years old.

4.2 LENGTH OF PIPE BY DIAMETER

Within the overall District service area, there are approximately 519 miles of sanitary sewers consisting predominately of 8- to 12-inches in diameter. The District owns approximately 45 miles of interceptor sewers ranging from 12- to 66-inches in diameter. There are 232 miles of sewers within the SCPS basin and approximately 287 miles outside this area. Additionally, there are approximately 102 miles of combined sewers in the City of Utica. Most of the size specific information is available in the District's and City's GIS. A summary table is provided on Table 4-1.

TABLE 4-1

LENGTH OF PIPE BY DIAMETER

DIAMETER, INCHES	LENGTH OF PIPE, FT
4	345
5	267
6	27,499
8	906,167
9	2,410
10	77,460
12	59,780
14	4,595
15	9,728
16	8,818
18	45,403
20	2,934
21	12,463
24	42,645
27	3,928
30	38,263
36	31,798
42	7,158
48	5,039
60	2,131

5.0 DESCRIPTION AND CAPACITY OF EXISTING WPCP

5.1 WPCP DESCRIPTION

The WPCP was originally constructed in the late 1960s and began operation in 1971. As shown on Figure 5-1, the WPCP site is bounded by the Mohawk River to the north, the Oneida-Herkimer Solid Waste Facility to the East, and railroad tracks to the south. The original plant included two (2) bar screens, three (3) raw waste pumps, three (3) primary settling tanks, two (2) aeration basins, four (4) final settling tanks, and two (2) chlorine contact tanks. Facilities for sludge processing included four (4) sludge thickening tanks, eight (8) centrifuges, and two (2) fluidized bed incinerators. In the 1980s, the WPCP was expanded under a series of projects. New components installed in the 1980s included one (1) additional raw waste pump, one (1) additional mechanical bar screen, two (2) new grit chambers, one (1) additional primary settling tank, one (1) additional aeration basin, four (4) additional secondary settling tanks, dechlorination facilities, six (6) new belt filter presses to replace the previously installed centrifuges, and one (1) additional fluidized bed incinerator. The series of projects in the 1980s represent the latest major expansion at the WPCP. The site plan of the WPCP as it is currently configured is provided on Figure 5-2.

A schematic of the flow through the WPCP, as the plant is currently operated, is provided on Figure 5-3. Preliminary treatment consists of screening and grit removal. Influent flows received at the WPCP are conveyed by gravity through three (3) mechanically cleaned bar screens. The screens have a clear spacing between bars of 1-inch to remove large objects and stringy materials from the influent flow. Screened flows are pumped by the raw waste pump station, consisting of four (4) vertical centrifugal pumps. The pumps are operated on variable frequency drives (VFDs) to vary the pumping rate in response to liquid levels in the wet well. The pumps convey flow through two (2) 30-foot diameter detritus-type grit chambers. Grit collected in the chambers is pumped through cyclone grit separators and grit classifiers. Collected grit is hauled offsite for landfill disposal.

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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

WPCP AERIAL PHOTO

Job Number	8614782
Revision	A
Date	JUNE 2012

Figure 5-1

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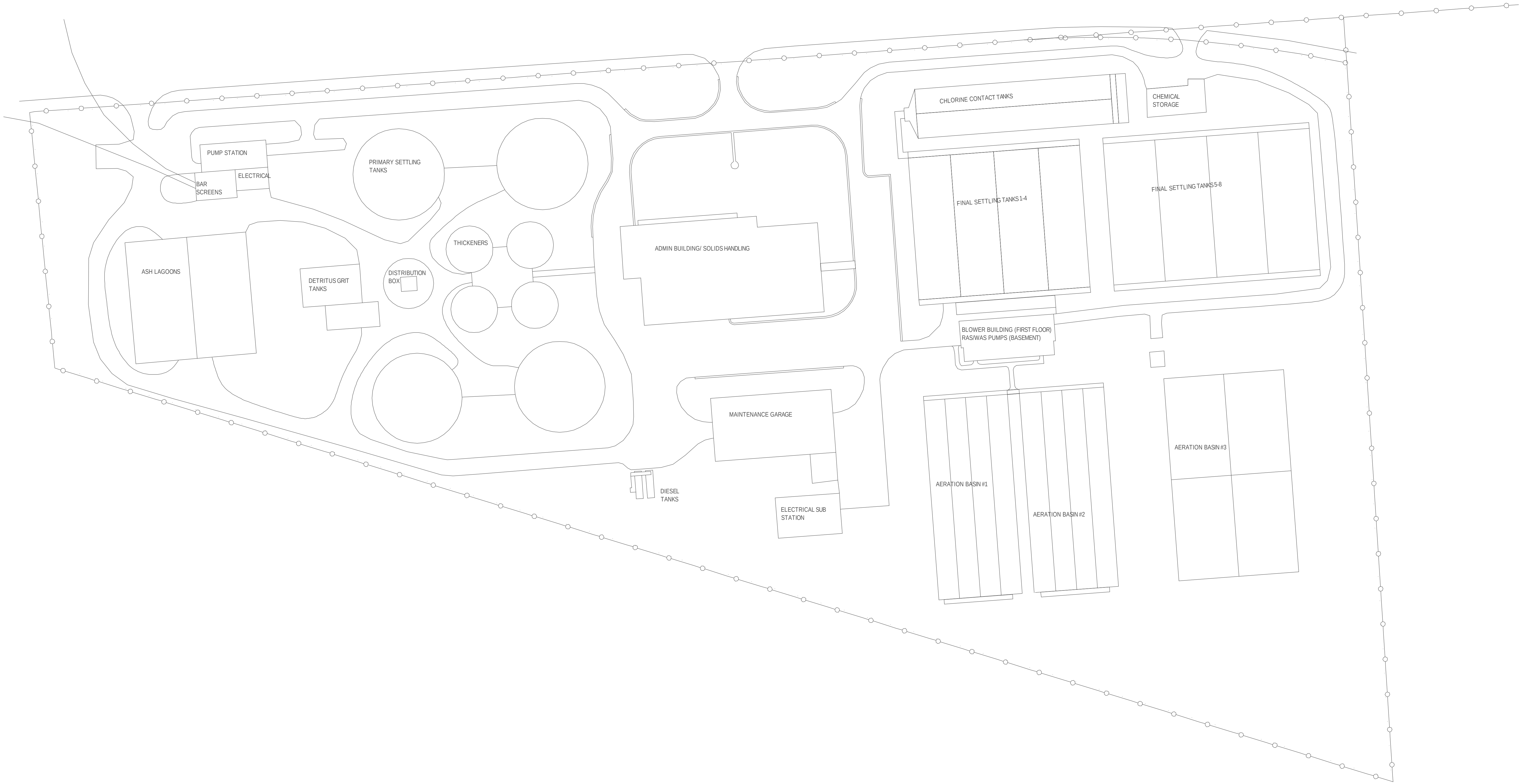
Plot Date: 22 August 2012 - 9:58 AM Cad File No: G:\86\14782\CADD\Drawings\Figures\FIGURES.dwg



0 60' 120' 180'

SCALE 1"=120' AT ORIGINAL SIZE

N

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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

WPCP SITE PLAN

Job Number	8614782
Revision	A
Date	06/12

Figure 5-2

200 John James Audubon Parkway Suite 101, Amherst NY 14228 USA **T** 1 716 691 8503 **F** 1 716 691 8506 **E** amhmail@ghd.com **W** www.ghd.com

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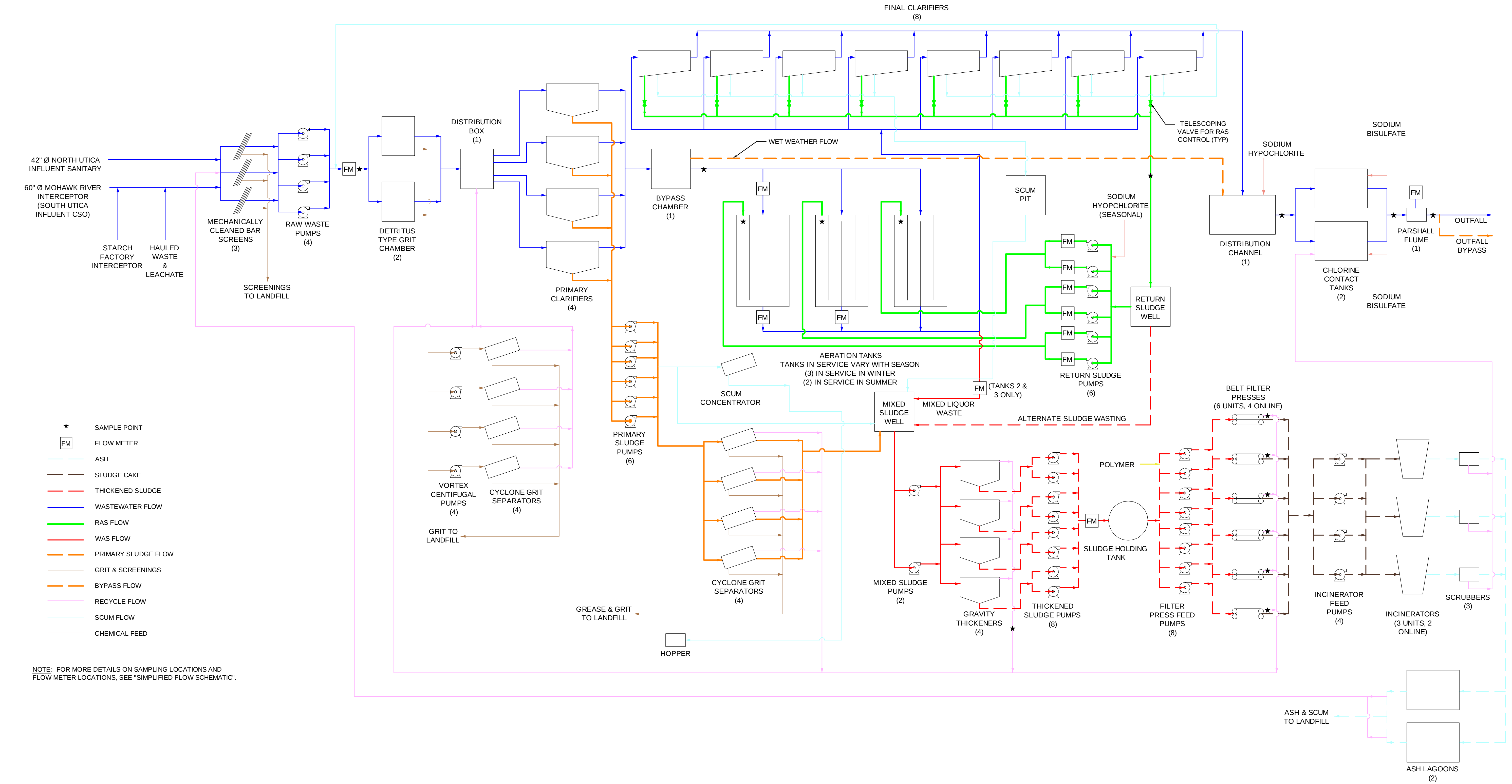

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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

WPCP FLOW
SCHEMATIC

Job Number	8614782
Revision	A
Date	JUNE 2012

Figure 5-3

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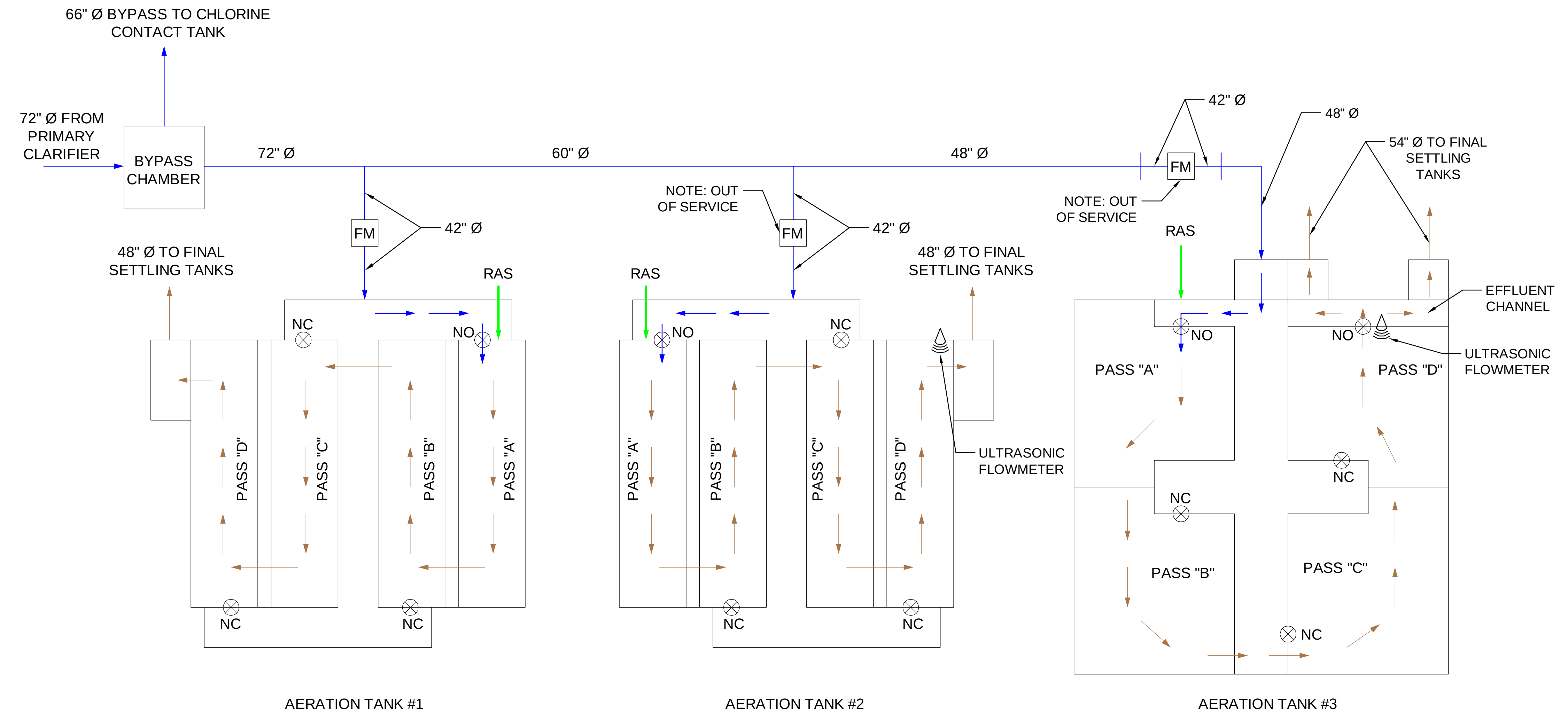
Plot Date: 22 August 2012 - 12:41 PM Cad File No: G:\86\14782\CADD\Drawings\Figures\SCHEMATICS-ALL.dwg



LEGEND

NO	NORMALLY OPEN
NC	NORMALLY CLOSED
FM	FLOWMETER
⊗	SLIDE GATE
	PRIMARY EFFLUENT
	RAS FLOW
	MIXED LIQUOR FLOW

OPERATING NOTES:

1. DURING WINTER, ALL THREE TANKS IN SERVICE.
2. DURING SUMMER, 1 SMALL TANK OFFLINE.
3. LARGE TANK IS NEVER OFFLINE.
4. WHEN ALL 3 TANKS ARE IN SERVICE, STRIVE FOR 30% / 30% / 40% FLOW SPLIT BETWEEN TANK 1 / TANK 2 / TANK 3. MANUALLY ADJUST GATES FOR FLOW SPLIT.
5. OPERATE IN CONVENTIONAL AERATION MODE YEAR-ROUND.
6. HAVE THE ABILITY FOR STEP-FEED MODE, BUT PRIMARY EFFLUENT STRENGTH HISTORICALLY NOT STRONG ENOUGH TO SUPPORT STEP-FEED.
7. TRY TO MAINTAIN SIMILAR MLSS IN EACH TANK.
8. NORMALLY WASTE FROM MLSS LINE.
9. WASTE BASED ON MLSS CONCENTRATION & VISUAL OBSERVATION OF MLSS. DO NOT WASTE BASED ON FLOW, SRT, TIMERS, ETC.
10. MANUALLY ADJUST RAS PUMPS TO MAINTAIN RAS FLOW +/- 30% OF WWTP INFLUENT FLOW.
11. OPERATORS HAVE NEVER HAD LUCK WITH AUTOMATED D.O. CONTROL SYSTEMS.
12. IF D.O. CONCENTRATION GETS TOO LOW, FILAMENT GROWTH IS PREVALENT.
13. OPERATORS HAVE NOTED SIGNIFICANT RAG & GRIT ACCUMULATION ON DIFFUSERS IN TANKS 1 & 2.
14. TANK 3 HAS NEVER BEEN OFFLINE SINCE STARTUP IN1984.

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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

AERATION FLOW SCHEMATIC

Job Number	8614782
Revision	A
Date	JUNE 2012

Figure 5-4

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From the preliminary treatment system, degritted flow is conveyed to a primary clarifier distribution box. The box splits flow between the four (4) circular primary clarifiers. Each primary clarifier has a diameter of 105-feet and a sidewater depth of 10-feet. The primary clarifiers reduce BOD and suspended solids before flow proceeds to the secondary treatment system. Sludge that settles in the primary clarifiers is collected in a central hopper, and pumped through cyclone separators for grit removal. Degritted primary sludge is pumped to gravity thickeners for further processing. Effluent from the primary clarifiers flows by gravity to the secondary treatment system.

The secondary treatment system is an activated sludge type system, consisting of three (3) aeration basins and eight (8) final clarifiers. The system is designed to provide seasonal nitrification and year round BOD removal.

Primary effluent (secondary influent) flows to the three (3) aeration basins by gravity. There are two (2) smaller aeration basins and one (1) large aeration basin. The two (2) smaller basins each utilize up to four (4) passes, and each pass measures 25 feet wide by 252 feet in length. The larger aeration basin also utilizes up to four passes, with each pass measuring 71.5 feet wide by 127 feet in length. The sidewater depth in all three (3) basins is 15 feet. With all three (3) basins in operation, the total aeration volume is approximately 1,300,000 cubic feet. There are four (4) centrifugal blowers, three (3) with a capacity of 15,000 cfm each and one (1) with a capacity of 20,000 cfm, which supply air to fine bubble diffuser systems installed in each aeration basin. The basins are designed to operate in conventional aeration or step feed modes, and can operate in a contact stabilization mode with minimal modifications. Operators prefer to utilize the conventional aeration mode of operation.

A schematic of the aeration system is provided on Figure 5-4. During winter months, all three (3) tanks are in service. During the summer, one (1) of the smaller tanks is offline. The larger tank

(Tank No. 3) has rarely been out of service since it was placed online in 1984. When all three (3) tanks are in service (winter), operators strive for a 30% / 30% / 40% flow split between Tank No. 1, Tank No. 2, and Tank No. 3, respectively. During summer when one (1) of the two (2) smaller tanks is out of service, the desired flow split is 40% / 60% between the small tank in service and Tank No. 3, respectively. Flow to each tank is adjusted manually using the slide gates. Operators utilize the conventional aeration mode year-round, in which all wastewater flow and RAS is directed to “Pass A.” The RAS rate is maintained at approximately 30% of plant influent flow by manually adjusting the speed of RAS pumps.

Wasting from the aeration basins is normally through the mixed liquor line between the aeration tanks and final clarifiers. Wasting rates are based on visual observation of sludge characteristics and MLSS concentration. Wasting is continuous throughout the day.

From the aeration basin, mixed liquor flows to the final clarifiers. There are four (4) smaller tanks and four (4) larger tanks. The four (4) smaller tanks each have three (3) parallel longitudinal compartments, each measuring 16.5 feet wide by 170 feet long. The four (4) larger tanks each have three (3) parallel longitudinal compartments, each measuring 21 feet wide by 170 feet long. The total surface area of all eight (8) tanks is 76,500 square feet. All tanks are equipped with a chain-driven mechanical sludge collector mechanism and surface scum skimmer. Settled sludge flows by gravity to the return sludge well, where it can either be pumped to the aeration basins as return activated sludge or continue to the mixed sludge well where it combines with primary sludge before being pumped to the gravity thickeners and the waste sludge processing system. There are a total of six (6) RAS pumps. Four (4) are rated at 4,640 gpm each and two (2) are rated at 8,100 gpm each.

Secondary effluent flows from the final clarifiers to the chlorine contact tank. Liquid sodium hypochlorite is added seasonally at the contact tank to provide disinfection prior to effluent discharge to the Mohawk River. Liquid sodium bisulfate is also added seasonally for dechlorination.

Solids processing at the WPCP is achieved through gravity thickeners and belt filter presses. Waste sludge is a mixture of primary sludge and secondary sludge. The mixed sludge is pumped from the mixed sludge well to the four (4) gravity thickeners through one (1) of two (2) mixed sludge pumps, each rated at 4,600 gpm. There is no thickener distribution box, and flow split between the thickeners is achieved through separate pipes to each tank from the mixed sludge pump discharge header. Each thickener is 55 feet in diameter. Thickened solids are pumped from the bottom of the thickeners to a sludge holding tank. There are a total of eight (8) thickened sludge pumps (two per thickener), each rated at 150 gpm. Overflow (supernatant) from the thickeners is recycled to the primary clarifier distribution box.

From the sludge holding tank, thickened sludge is pumped to the belt filter presses. There are a total of six (6) belt presses, but only four (4) are currently operational. There are eight (8) belt filter press feed pumps. Dewatered sludge cake is pumped to the incinerators, and filtrate is recycled to the primary clarifier distribution box. Under normal conditions, two (2) belt presses are used per one (1) incinerator. There are three (3) incinerators, but historically (1990 – 2012) only two (2) are operational. At the time of this report, one (1) of the two (2) operating units is out of service due to mechanical breakdowns. Sludge which would normally be conveyed to this incinerator is being temporarily dewatered with an on-site temporary trailer mounted belt filter press. Dewatered sludge from the temporary press is being hauled offsite to a sanitary landfill.

Incinerated sludge (ash) is held onsite in ash lagoons. The lagoons are periodically emptied and ash is sent to a landfill. Overflow from the lagoons is directed to the head of the WPCP, upstream of the mechanical bar screens.

WPCP operators collect flowmeter data and process data at several locations within the WPCP. This data is used for process control and for reporting to the NYSDEC. A simplified flow schematic, showing data collection points within the WPCP, is provided on Figure 5-5.

The WPCP operates under the conditions of their SPDES permit, which is regulated by the NYSDEC. A summary of the existing SPDES permit conditions is provided on Table 5-1.

TABLE 5-1

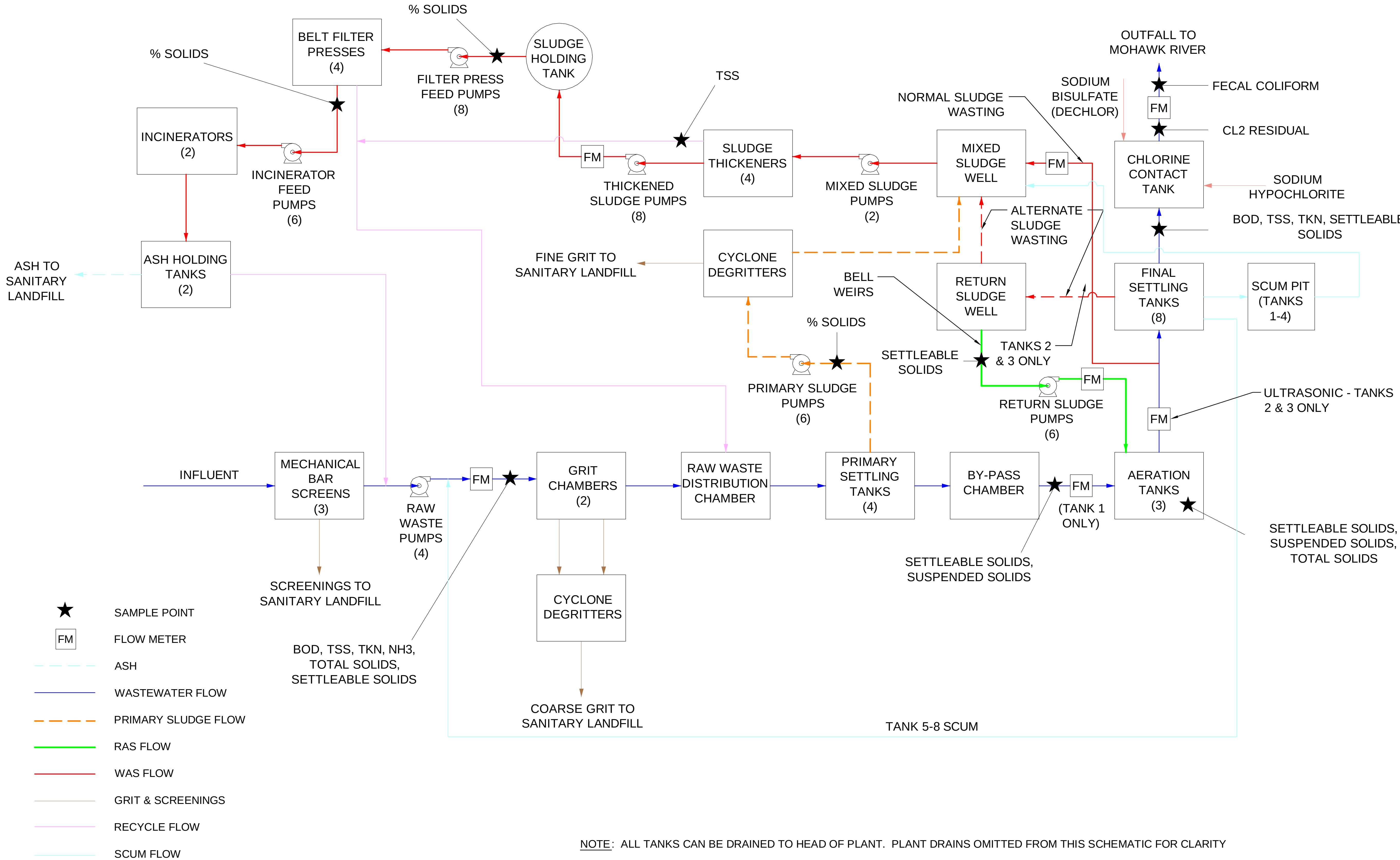
WPCP SPDES PERMIT LIMITS

Parameter		Unit	Summer Permit Limit (June 1 – October 31)	Winter Permit Limit (November 1 – May 31)
Flow	Required Capacity (1)	mgd	48	53
BOD	Max 30-Day Average	mg/L	--	30 (2)
		lb/d	--	12,000 (2)
	Max 7-Day Average	mg/L	--	45
		lb/d	--	18,000
CBOD	Max 30-Day Average	mg/L	25 (2)	--
		lb/d	3,300 (2)	--
	Max 7-Day Average	mg/L	45	--
		lb/d	5,400	--
TSS	Max 30-Day Average	mg/L	30 (2)	30 (2)
		lb/d	10,000 (2)	12,000 (2)
	Max 7-Day Average	mg/L	45	45
		lb/d	15,000	18,000
Settleable Solids	Daily Maximum	ml/l	0.1	0.1
TKN Nitrogen (as N)	Max 30-Day Average	lb/d	1,120	Monitor
Total Residual Chlorine (3)	Daily maximum	mg/L	0.1	--
Fecal Coliform	Max 30-Day Average	per 100 mL	200	--
	Max 7-Day Average	per 100 mL	400	--
pH	(range)	N/A	6.0 – 9.0	6.0 – 9.0

(1) Required WPCP capacity prior to CSO overflow

(2) Effluent values also shall not exceed 15 percent of influent values

(3) Effluent disinfection and associated limits required June 1 through September 30



NOTE: ALL TANKS CAN BE DRAINED TO HEAD OF PLANT. PLANT DRAINS OMITTED FROM THIS SCHEMATIC FOR CLARITY

WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

WASTEWATER FLOW SCHEMATIC - SIMPLIFIED

Job Number 8614782
Revision A
Date JUNE 2012

Figure 5-5

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5.2 WPCP PHYSICAL CONDITION ASSESSMENT

In 2010, GHD Consulting Engineers, LLC prepared a physical condition assessment of the WPCP. This assessment reviewed existing buildings, tanks, and equipment for structural, architectural, electrical, and mechanical conditions. The assessment provided and estimate of the remaining life of major facility components and approximate budgetary replacement values. While not necessarily associated with expanding the WPCP to treat future flows and loads, components of the condition assessment were considered when developing the overall budgetary costs for WPCP expansion described in this evaluation. These components are discussed in greater detail in subsequent sections of this Report.

5.3 AERATION SYSTEM MODELING

To estimate the maximum capacity of the existing aeration basins based on existing conditions, a dynamic model of the basins was created using the BioWin software platform created by EnviroSim Associates, Ltd. This software is the industry leading standard for wastewater simulation.

5.3.1 Model Description and Development

The existing aeration basins operate in a conventional, plug flow configuration. Piping and gates are in place to switch to a step-feed or contact stabilization mode of operation with minimal effort. The BioWin modeling considered these various three (3) modes of operation, and used actual WPCP operating data during wet weather conditions. For all modes of operation, the following criteria were utilized:

1. MLSS limited to 3,000 mg/L based on peak solids loading to the Final Settling Tanks
2. Required effluent NH_3 concentration is 3.0 mg/L during winter conditions
3. 7.2º C winter design temperature

4. Existing aeration volume is not expanded

The above criteria represent a conservative approach to modeling the aeration basins. 7.2° C is a low design temperature, but temperatures this low have been recorded at the WPCP. Although the WPCP is not required to nitrify in the winter, if an effluent NH_3 concentration of 3.0 mg/L can be achieved during winter months, the permitted effluent TKN load of 1,120 lb/d can easily be achieved during summer months.

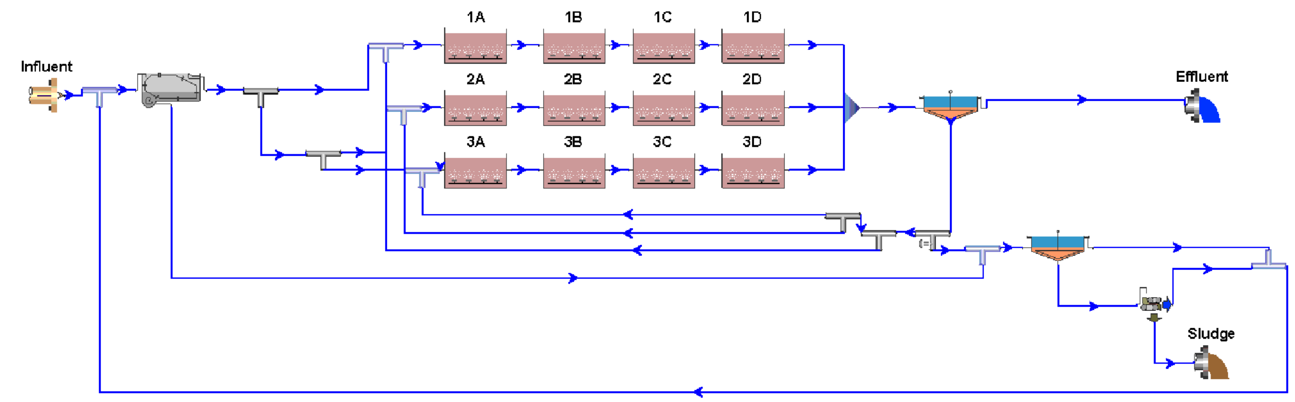
For all three (3) models, actual WPCP data were used to estimate primary effluent characteristics. To simulate new wet weather flows, a side-stream influent was included. Figure 5-6 provides a schematic representation of each of the three (3) models.

Once the models were created, plant influent data from 2009 were used to predict aeration system performance. 2009 data were used because 2009 had several wet weather events and the model could be calibrated to a variety of influent flow conditions.

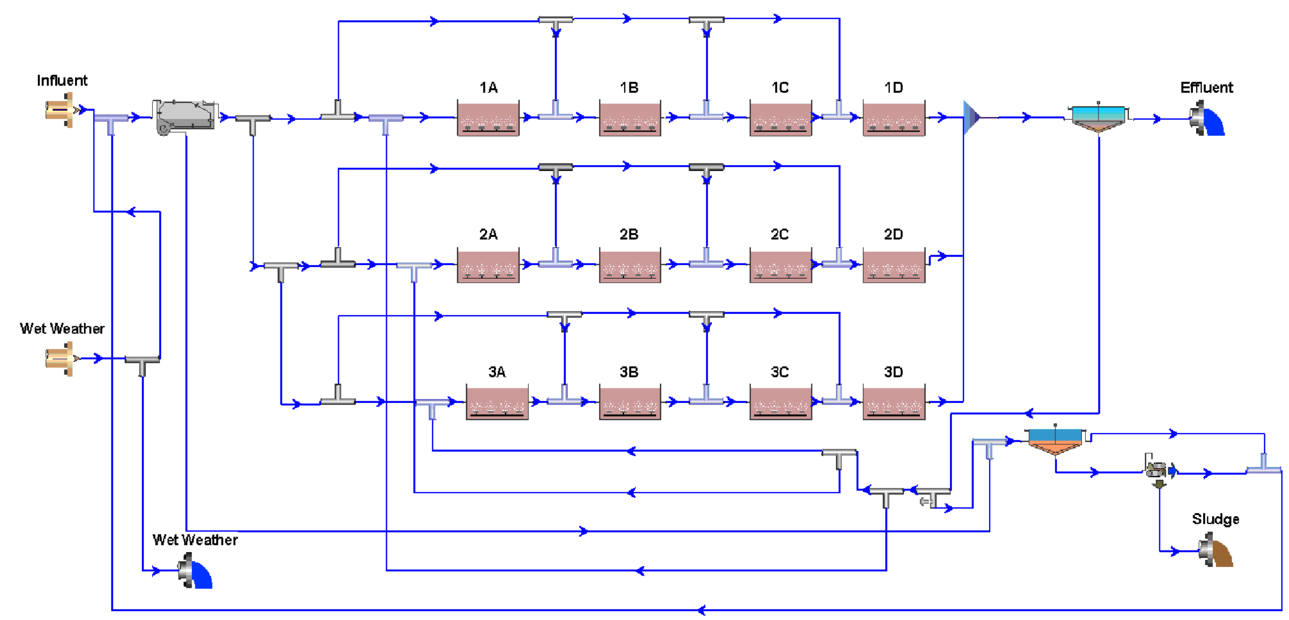
5.3.2 Model Calibration

The model was first run in the conventional aeration mode utilizing existing primary effluent data. The predicted performance of the aeration basins (based on indicators such as mixed liquor suspended solids (MLSS), waste sludge generation, etc.) were compared to actual WPCP data for the same time period as was utilized for the primary effluent data. Assumed values used to develop the model, such as endogenous decay and sludge yield coefficients and bacterial growth rates, were adjusted to calibrate the values predicted by the model to values observed at the WPCP.

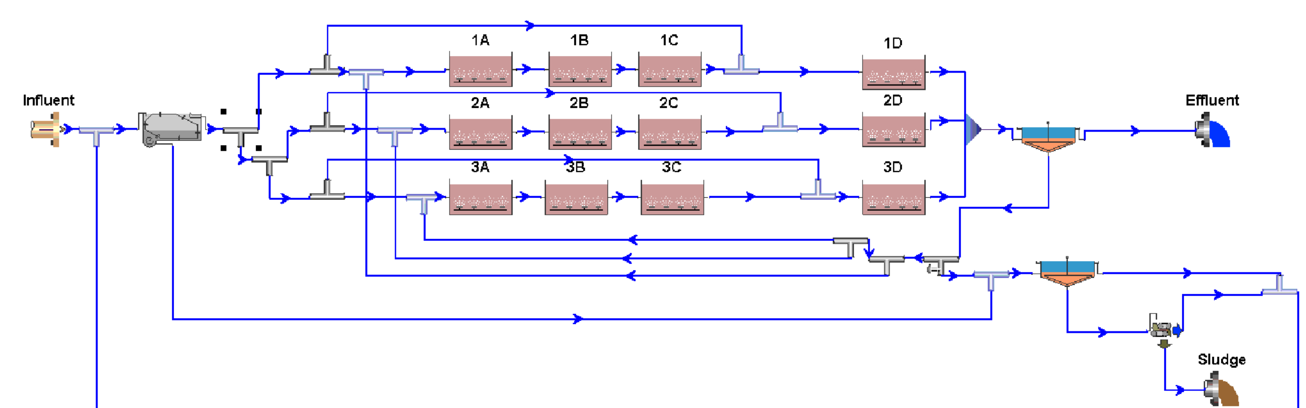
Once the model was calibrated, it was run for the three (3) modes of operation at various levels of increased flow to the WPCP. The model calibration files are provided in Appendix A.



EXISTING CONFIGURATION



STEP-FEED CONFIGURATION



CONTACT STABILIZATION CONFIGURATION

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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

AERATION SYSTEM
MODEL SCHEMATICS

Job Number	8614782
Revision	A
Date	JUNE 2012

Figure 5-6

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5.4 FINAL SETTLING TANK MODELING

On-site testing of the clarifiers was conducted on two (2) separate occasions: August 15 through 16, 2011 and October 31 through November 2, 2011. Results from the first period of testing served to set up and calibrate the computational fluid dynamic (CFD) model, and results from the second round served to validate the model.

The following paragraphs provide an outline of the testing activities on the two (2) occasions:

- ***Mixed Liquor Settling Characteristics***

  Historical sludge volume index (SVI) data was compiled and 50-percentile and 90-percentile values were determined.

  Batch settling tests were conducted using settling column apparatus. Settling tests were conducted for a range of mixed liquor total suspended solids (TSS) concentrations. Various blends of RAS and MLSS were used and mixed liquor was diluted to produce at least six (6) different sludge concentrations for the test series. The Vesilind parameters (V0 and k) were determined, and the SVI of the sludges were also noted.

  Previous correlations between SVI and settling parameters were used, and adjusted as required by the SVI and V0 and k values determined during these tests. The objective was to approximate historical settling parameters, based on the historical SVI values (calculated from sludge density index (SDI) values) recorded at the plant.

- ***Mixed Liquor Flocculation Characteristics***

  Mixed liquor samples were collected at the two (2) ends of the mixed liquor distribution channel feeding the secondary clarifiers. A six-paddle Phipps and Bird jar test apparatus using 2-liter jars was used for testing. The 2-liter jars were filled with 1.8 liters of mixed liquor immediately (i.e. with minimal delay) after collection from the mixed liquor channel.

The stirrer speed was maintained at an intermediate value, sufficient to prevent settling during the test.

Each jar was stirred at the speed determined in (d) for incrementally increasing stirring times. Each jar was allowed to settle for 30 minutes and supernatant samples analyzed for TSS. Because of issues with the TSS results from the first round of tests, a turbidimeter was used during the second round in addition to TSS determinations, to improve the accuracy of the results. From the results, the flocculation and breakup parameters, KA and KB were determined.

- ***Dispersed Suspended Solids (DSS) and Flocculated Suspended Solids (FSS) Measurements***

  The DSS is operationally defined as the supernatant TSS concentrations after 30 minutes of settling in a Kemmerer sampler. The FSS is defined as the supernatant TSS after 30 minutes of flocculation followed by 30 minutes of settling. Samples of mixed liquor were collected from the mixed liquor distribution channel, several times during the testing period, using a Kemmerer sampler, and the DSS determined for each sample.

  The same jar test apparatus as used for measuring the flocculation parameters, was used for the FSS tests. Samples taken during the testing period were placed in the jar apparatus, stirred for 30 minutes at a speed sufficient to maintain the mixed liquor in suspension, and settled for 30 minutes. The supernatant TSS constituted the FSS.

  At the same time that mixed liquor samples were taken for DSS and FSS determinations, secondary effluent samples were taken to determine the effluent suspended solids (ESS).

5.4.1 Stress Testing

A series of on-site stress tests were performed to evaluate the performance of the secondary clarifier under peak load conditions and to determine how close to the critically loaded condition the clarifiers can be operated. The results of the stress tests were compared with the CFD model predictions to confirm accuracy.

Stress testing was approached by gradually increasing the flow to the clarifiers under test until the maximum stress was applied. The flow was then gradually reduced until normal operational conditions were achieved. The maximum stress condition was when the sludge blanket started to build up, or deterioration in effluent quality was noted. The influent and effluent flows to the clarifiers under test were determined as accurately as possible. The RAS flow was also recorded during testing.

Flows were increased to the clarifiers by taking clarifiers off line successively. Measurements were taken when the clarifiers remaining in operation were under “quasi steady state”, defined as having operated for at least a retention time, after changes had been made. The following measurements were taken:

- Water temperature
- Note general clarifier appearance.
- Influent flow to clarifiers
- Influent MLSS to clarifiers
- ESS from clarifiers
- Sludge blanket depth measured at various points along the length of the clarifier. Note blanket history during the test.
- RAS flow
- RAS TSS concentration

5.4.2 Model Calibration and Validation

The rectangular version of the model 2Dc was calibrated based on field testing conducted by the project team. Results from the second round of field testing served to validate the model. Calibration and validations were completed for both the FST No. 4 and FST No. 5 tanks. The model validation and the estimation of the clarifier capacity for the 90 percentile are shown in Appendix B. The model concluded the capacity of FST No. 4 and FST No. 5 are 1,600 gpd/sf and 1,450 gpd/sf, respectively, for a RAS flow of 3.6 mgd for an MLSS of 1,800 mg/L. The model also suggested that a proportional RAS flow might be helpful in maintaining good removal at both high and low surface overflow rate (SOR).

At a MLSS loading to the final settling tanks, the model predicts the capacity of the FSTs to decrease. At a MLSS of 3,000 mg/L, the capacity of FST No. 4 and FST No. 5 are 890 gpd/sf and 800 gpd/sf, respectively. The capacity of the final settling tanks is significantly hindered at MLSS concentrations greater than 3,000 mg/L; which is why the aeration basin modeling used 3,000 mg/L as a maximum MLSS concentration.

5.5 EXISTING WPCP PROCESS CAPACITY

In September 2008, the project team prepared a capacity study of the WPCP. The capacity of each process unit at the WPCP was evaluated. For some processes, such as pumping, the hydraulic capacity only was evaluated. For others, such as aeration, the hydraulic and organic capacities were evaluated. A process by process summary is presented below. The capacity analysis presented below is based on the 2008 study, as well as more recent data and operational information received from WPCP staff.

5.5.1 Mechanical Bar Screen Capacity

There are three (3) equally-sized mechanically cleaned bar screens. Each screen is installed in a 60-inch wide channel and has a 1-inch clear opening between bars. The rated capacity of each screen is 36 mgd based on information presented in the WPCP operations and maintenance (O&M) manual. With two (2) screens in service, the peak capacity is approximately 72 mgd.

The recommended minimum approach velocity is 1.25 feet per second to prevent solids deposition in the channel. The recommended maximum approach velocity is 3.0 feet per second to prevent screenable solids from being forced through the openings between bars. Ideally, the mechanical rake mechanisms are controlled by differential level controllers. As the upstream water surface becomes higher due to accumulation of screenable solids on the bars, the rake removes the solids. However, plant operators typically operate the screens continuously due to the relatively long time to complete a cleaning cycle (2 to 3 minutes per cycle).

5.5.2 Influent Pumping Capacity

There are a total of four (4) equally sized raw waste pumps. Based on pump curve/system curve analysis prepared in 2008, the station can convey a peak flow of approximately 100 mgd to the grit system with one (1) unit out of service.

5.5.3 Grit System Capacity

The grit chambers provide a total volume of 6,030 cubic feet. Typical standards recommend a minimum hydraulic retention time of 1 minute at peak hourly flows. The peak hydraulic capacity of the grit system is 65.0 mgd based on the 1 minute retention time. This compares favorably with the current peak flow through the WPCP of approximately 55 mgd.

5.5.4 Primary Settling Tank Capacity

The four (4) primary settling tanks provide a total surface area of 34,600 square feet. The Ten-States Standards recommend a surface overflow rate of 1,000 gpd/ft^2 at average flow, and up to 2,000 gpd/ft^2 at peak hourly flow. Based on these rates, the existing primary clarifiers have a capacity of 34.6 mgd and 69.3 mgd at average and peak flows, respectively. The primary settling tanks are slightly undersized for existing winter average flow conditions (42 mgd winter average flow from 2005 – 2011), and are adequately sized for existing summer average flow conditions (30 mgd summer average flow from 2005 – 2011). The primary clarifiers are adequately sized for the existing peak flow of approximately 55 mgd.

Although the primary settling tanks are theoretically adequately sized for a peak flow of 69.3 mgd, operators have reported significant issues with the primary settling tanks when flow exceeds 55 mgd. At this flow rate, the scum baffles and scum collection beaches are submerged. The hydraulic issues with the scum baffles and scum beaches are the main reason the WPCP limits peak flows to 55 mgd.

The primary settling tanks are more hydraulically limited than process limited. Based on data from 2005 through 2011, the annual average (summer and winter) WPCP influent TSS concentration is approximately 80 mg/L. The annual average (summer and winter) primary settling tank effluent TSS concentration over the same period of time is approximately 44 mg/L. The primary clarifiers are removing approximately 45% of influent TSS on an annual average basis, which is well within typical performance standards.

5.5.5 Aeration Capacity

The aeration system is required to provide CBOD removal and nitrification during summer months, and BOD removal only during winter months. Therefore, the aeration system has a different capacity during the summer than during the winter. During the summer, one (1) small tank is typically taken out of service and due to generally lower flows and organic load. During summer months, the total available aeration volume is 922,800 ft^3. During winter months, with all three (3) basins online, the available volume increases to 1,300,800 ft^3.

The capacity of the aeration basins can be estimated based on several criterion, mainly hydraulic retention time and organic (BOD) loading rate. During summer months, when nitrification is required, typical standards are a minimum 6-hour hydraulic retention time and a maximum organic loading of 15 lb BOD/1,000 ft^3. During winter months, where nitrification is not required, typical standards are a minimum 4-hour hydraulic retention time and a maximum organic loading of 40 lb BOD/1,000 ft^3. Based on these criteria, the capacity of the aeration basins is presented in Table 5-2.

TABLE 5-2

THEORETICAL CAPACITY OF AERATION BASINS

PARAMETER	**VALUE**	**BASIS**
Peak Hydraulic Flow, Summer	27.6 mgd	6-Hour Retention Time
Peak Hydraulic Flow, Winter	58.4 mgd	4-Hour Retention Time
Maximum Organic Loading, Summer	13,840 lb/d BOD	15 lb BOD per day per 1,000 ft^3
Maximum Organic Loading, Winter	52,030 lb/d BOD	40 lb BOD per day per 1,000 ft^3

The values presented in Table 5-2 are based on typical design standards and are not necessarily specific to the influent characteristics and operation of the Oneida County WPCP. To estimate the maximum capacity of the existing aeration basins based on existing conditions, the BioWin model described in Section 5.3 was utilized.

Based on BioWin model, and utilizing actual WPCP operating data, the capacities of the aeration basins for different modes of operation are presented in Table 5-3.

TABLE 5-3

CAPACITY OF AERATION BASINS BASED ON DYNAMIC MODELING

AERATION MODE	**APPROXIMATE PEAK FLOW (MGD)**	**COMMENTS**
Conventional	70	1. Exceeds capacity of Final Settling Tanks (See Section 5.5.6)
Step Feed	90	1. Exceeds capacity of Final Settling Tanks (See Section 5.5.6)
Contact Stabilization	110	1. Exceeds capacity of Final Settling Tanks (See Section 5.5.6) 2. Effluent NH_3 < 3.0 mg/L cannot be sustained.

Under conventional and step feed configurations, the aeration basins can support a peak flow of 70 mgd and 90 mgd, respectively while operating at a MLSS of 3,000 mg/L and maintaining an effluent NH_3 concentration less than 3.0 mg/L during winter conditions. The Contact Stabilization configuration can support a higher peak flow, but effluent NH_3 cannot be maintained less than 3.0 mg/L during winter conditions.

5.5.6 Final Settling Tank Capacity

The four (4) smaller final settling tanks provide a combined surface area of 33,660 ft^2, and the four (4) larger final settling tanks provide a combined surface area of 42,840 ft^2. The smaller tanks have a sidewater depth of 10 feet, and the larger tanks have a sidewater depth of 12 feet.

During summer months, one (1) tank is taken out of service at a time for routine maintenance. The total surface area available in the summer is 65,790 ft^2, based on one (1) large tank out of service. With all eight (8) tanks in service during the winter, the available surface area is 76,500 ft^2.

The Ten-States Standards list a recommended peak surface overflow rate of 1,200 gpd/ft^2 for final settling tanks at an activated sludge plant. However, the project team conducted stress testing of the final settling tanks in the fall of 2011 to verify the actual peak flow capacity. Field data obtained during stress testing was used to calibrate and validate a computational fluid dynamics (CFD) model of the tanks. Based on preliminary results of stress testing and subsequent CFD modeling:

- The older (smaller) tanks can sustain a peak surface overflow rate of approximately 890 gpd/ft^2 at a MLSS concentration of 3,000 mg/L, and approximately 1,600 gpd/ft^2 at a MLSS concentration of 1,800 mg/L.

- The newer (larger) tanks can sustain a peak surface overflow rate of approximately 800 gpd/ft^2 at a MLSS concentration of 3,000 mg/L, and approximately 1,450 gpd/ft^2 at a MLSS concentration of 1,800 mg/L.

- The tanks become limited at sludge blanket depths of approximately 50% of sidewater depth.

Based on preliminary CFD modeling, the final settling tanks can effectively treat a peak flow of approximately 56 mgd at a MLSS concentration of 3,000 mg/L and the largest tank out of service. This closely represents existing operating conditions at the WPCP during summer months.

If the MLSS concentration were lowered to 1,800 mg/L, the CFD model suggests the final settling tanks may be able to effectively treat a peak flow of 100 mgd with the largest tank out of service.

CFD modeling efforts have estimated the combined treatment capacity of all eight (8) final settling tanks as 65 mgd at a MLSS concentration of 3,000 mg/L. When the final settling tanks are compared to the aeration basins, the final settling tanks are the limiting process in the secondary treatment system.

5.5.7 Chlorine Contact Tank Capacity

The two (2) chambers of chlorine contact tank provide a combined volume of 140,000 ft^3. The recommended contact time for chlorine disinfection is 15-minutes at peak flow. The volume provided by the existing contact tanks is enough to provide 15-minutes of contact time for a peak flow of 100.5 mgd. Although the contact tank can theoretically treat a peak flow of 100.5 mgd, the WPCP outfall can only convey a peak flow of approximately 65 mgd to the Mohawk River.

5.6 EXISTING WPCP SLUDGE PROCESSING CAPACITY

Sludge processing data were provided by the WPCP. These data include:

- Mixed Liquor Supspended Solids Concentration (MLSS)
- Primary Sludge Concentration
- Thickened Sludge Flow
- Thickened Sludge Concentration
- Concentration of Solids in Sludge Cake off Belt Press
- Weight of Sludge Incinerated

A summary of sludge processing data is provided in Table 5-4.

TABLE 5-4

2005 – 2011 SLUDGE PROCESSING DATA

PARAMETER	UNIT	2005 – 2011 ANNUAL AVERAGE	2005 – 2011 SUMMER AVERAGE	2005 – 2011 WINTER AVERAGE	2005 – 2011 MAX 30-DAY SUMMER	2005 – 2011 MAX 30-DAY WINTER
MLSS – Tank 1	mg/L	1,890	1,780	1,960	2,410	2,620
MLSS – Tank 2	mg/L	2,030	1,860	2,050	3,300	2,950
MLSS – Tank 3	mg/L	2,000	1,890	2,070	2,660	3,360
Primary Sludge Concentration	mg/L	2,230	2,240	2,220	3,750	6,680
Thickened Sludge Flow (1)	gpd	63,240	59,390	65,730	85,140	90,110
Thickened Sludge Concentration	mg/L	45,240	40,650	48,030	54,700	64,160
Belt Press Cake Concentration	% Solids	22.1%	23.1%	21.3%	35.8%	33.6%
Weight of Sludge Incinerated (1)	dry lb/d	23,680	20,110	25,860	27,680	32,720

(1) Data available from January 2009 through December 2011

The WPCP does not meter or record waste activated sludge (WAS) flow. Return activated sludge (RAS) is metered but not recorded. Operators manually adjust the speed of RAS pumps to maintain a RAS rate approximately equal to 30% of WPCP influent flow. The 30% rate is maintained throughout the entire year regardless of seasonal operations or influent flow conditions. However, modeling suggests an increased RAS rate during peak flows may increase the capacity of the secondary treatment system. Based on the 30% recycle rate, the approximate RAS flows are presented in Table 5-5.

TABLE 5-5

RETURN ACTIVATED SLUDGE FLOW RATES (1)

Parameter	**Unit**	**2005 – 2011 Summer Value**	**2005 – 2011 Winter Value**
Average RAS Flow	mgd	9.0	12.6
Maximum 30-Day Average RAS Flow	mgd	14.4	16.2
Peak Instantaneous RAS Flow	mgd	16.5	16.5

(1) Approximate based on 30% of WPCP Influent Flow

5.6.1 Gravity Thickener Capacity

The four (4) 55-foot diameter gravity thickeners represent the first process in the waste activated sludge treatment system. Three (3) tanks are currently in service and the fourth unit has never been placed online. The combined surface area provided by all four (4) thickeners is 9,500 ft^2, and the available surface area of the three (3) tanks in service is 7,125ft^2. The typical standard for peak hydraulic loading to a gravity thickener is 200 gpd/ft^2. For the thickeners at the WPCP, the hydraulic capacity is 1.9 mgd if all tanks were operable, and 1.4 mgd with the three (3) tanks currently in service.

The typical standard for solids loading rate to a gravity thickener is eight (8) dry pounds of solids per day per ft^2. The thickeners at the WPCP can accommodate approximately 76,000 dry pounds per day if all four (4) tanks were operable. The loading capacity with the three (3) tanks currently in service is approximately 57,000 dry pounds per day.

The WPCP does not currently measure or record WAS flow to the gravity thickeners. However, flow and percent solids from the thickeners to the belt presses is recorded.

5.6.2 Belt Filter Press Capacity

There are a total of four (4) belt filter presses in operation at the WPCP. According to the basis of design presented in the WPCP O&M manual, each belt filter press is designed for a peak capacity of 780 dry pounds of solids per hour. Combined, the four (4) belt filter presses in operation have a capacity to process approximately 75,000 dry pounds per day.

5.6.3 Incineration Capacity

The WPCP utilized two (2) fluidized bed incinerator systems for final processing of dewatered sludge. According to the WPCP O&M manual, each incinerator system is designed to process up to 1,670 dry pounds of solids per hour. Combined, the two (2) incinerators can process approximately 80,200 dry pounds per day.

5.6.4 Sludge Pumping Capacity

There are various sludge pumping systems at the WPCP, including RAS pumps, WAS (mixed sludge) pumps, thickened sludge pumps, filter press feed pumps, and incinerator feed pumps. A summary of the capacity of sludge pumping systems, based on the rated nameplate data of the pumps, is presented in Table 5-6.

TABLE 5-6

SLUDGE PUMPING CAPACITY

PUMPING SYSTEM	NO. AND CAPACITY OF PUMPS	PEAK CAPACITY WITH LARGEST PUMP OUT OF SERVICE
RAS	4 @ 4,640 gpm, 2 @ 8,100 gpm	26,660 gpm (38.4 mgd)
WAS (Mixed Sludge)	2 @ 4,600 gpm	4,600 gpm (6.6 mgd)
Thickened Sludge	8 @ 150 gpm	1,050 gpm (1.5 mgd)
Filter Press Feed	8 @ 120 gpm	840 gpm (1.2 mgd)
Incinerator Feed	6 @ 20 gpm	100 gpm (0.14 mgd)

5.7 SUMMARY OF WPCP PROCESS CAPACITY

In general, the WPCP is currently meeting its SPDES discharge permit conditions at peak flows. There are hydraulic restrictions at the plant, but effluent quality is not compromised up to peak flows of 55 mgd.

5.8 EXISTING WPCP SOLIDS MASS BALANCE

A solids mass balance was prepared for existing conditions at the WPCP. The mass balance considers influent flows and loads, plus internal recycle flows from the thickeners, belt filter presses, and ash ponds. The mass balance for existing conditions is presented on Figure 5-7.

Once this mass balance was prepared, a similar mass balance was prepared to evaluate the solids handling facilities and determine required upgrades. The analysis of the solids handling and incineration facilities is detailed in Sections 7.7 and 7.8, respectively.

5.9 EXISTING WPCP HYDRAULIC CAPACITY

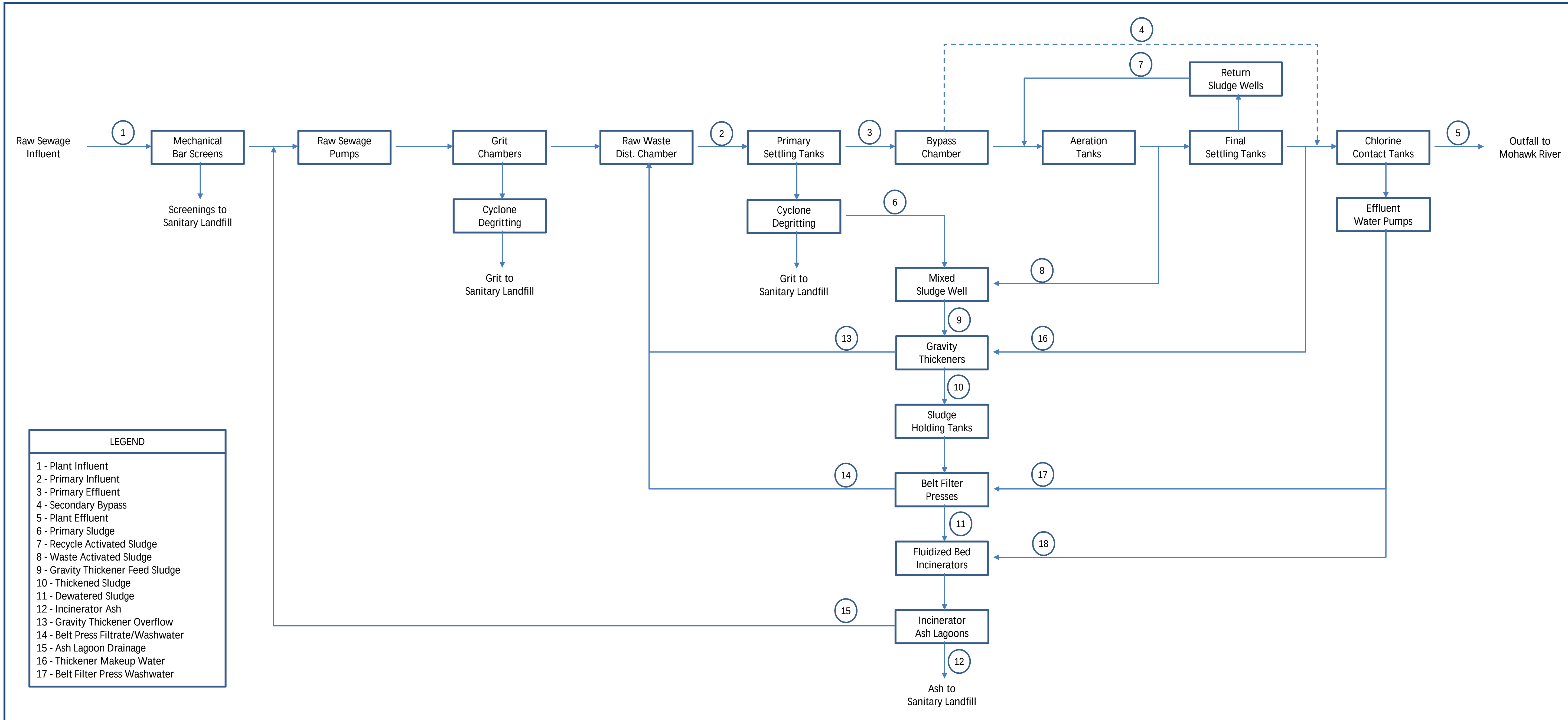
This section describes the calibration of the WPCP hydraulic model and the existing hydraulic capacity of the WPCP. The hydraulic model was used to determine the existing hydraulic capacity of the WPCP for winter operating conditions. The capacities were determined for average receiving water levels as well as the 25 year condition.

5.9.1 Hydraulic Model Calibration

A hydraulic model of the WPCP was developed previously as part of the Water Pollution Control Plant Capacity Study of September 2008. The model was developed with a PC-based hydraulics calculation model, PROFILE, for the gravity flow portion of the WPCP. The model was configured based on physical dimensions and elevations taken from original construction drawings and the 1983 expansion drawings (by Hazen and Sawyer), and appropriate energy losses were included in the model. At that time, the model was calibrated based on the water surface elevations (WSEs) provided in existing plant profile drawings. However the model calibration was recently updated with measured WSEs. Figure 5-8 illustrates the flow paths modeled.

The calibration update was performed in November of 2011. The WSEs throughout the WPCP were measured at two (2) different flow rates including dry-weather flow (DWF) of 33 MGD, and wet-weather flow (WWF) of 54 MGD. The RAS flow was estimated to be 27% for the DWF and WWF conditions. For each of these flow conditions the WPCP was set-up for winter operation, so all process tanks were online including:

- Two (2) Grit Tanks (GTs) – the GTs are the same size and it was assumed each tank received equal (50%) of the total plant flow.
- Four (4) Primary Settling Tanks (PSTs) - the PSTs are the same size and it was assumed each tank received equal (25%) of the total plant flow.
- Three (3) Aeration Tanks (ATs) - the ATs are not the same size. There are two small ATs and one (1) large AT. The flow split to the ATs is 30%, 30%, and 40% to AT#1 (small), AT#2 (small), and AT#3 (large), respectively. The flow split to the ATs is controlled by influent piping and influent gates.
- Eight (8) Final Settling Tanks (FSTs) - the FSTs are not the same size. There are four (4) small FSTs and four large FSTs. The flow split to the FSTs is 45% and 55% to the small and large FSTs, respectively. The flow split to the FSTs is controlled by influent gates.



Flow Stream	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
Flow, MGD	48.00	50.52	49.66	-	47.99	0.86	14.40	0.62	1.49	0.06	0.01		1.43	0.16	0.94	-	0.12	0.94
BOD5, mg/L	70.3	66.8	57.7	57.7	7.0										-	7.0	7.0	7.0
BOD5, lb/day	28,142	28,142	23,877	-	2,805										-	-	7	55
TSS, mg/L	52.1	67.8	36.6	36.6	8.3	1,863	11,202	2,554	2,154				112	391	750	8.3	8.3	8.3
TSS, lb/day	20,857	28,556	15,171	-	3,322	13,432	1,349,520	13,299	26,731				1,337	508	5,855	-	8	65
VSS, mg/L	41.6	42.2	22.8	22.8	5.2	1,161	6,955	1,586	1,340				70	243	-	5.2	5.2	5.2
VSS, lb/day	16,653	17,800	9,456	-	2,061	8,371	837,867	8,257	16,628				831	316	-	-	5	40
TKN, mg/L	9.9	9.4	9.1	9.1	2.4										-	2.4	2.4	2.4
TKN, lb/day	3,963	3,963	3,781	-	968										-	-	2	19
TS, %										5.53	20.85							
TS, lb/day										25,394	24,886	3,550						
TVS, % TS										62.2	62.2	-						
TVS, lb/day										15,797	15,481	-						

FIGURE 5-7

SOLIDS BALANCE
EXISTING CONDITIONS - AVERAGE DAILY FLOW
ONEIDA COUNTY WPCP, ONEIDA COUNTY NY

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Existing

- Two (2) Chorine Contact Tanks (CCTs) – the CCTs are the same size and it was assumed each tank received equal (50%) of the total plant flow.

Measurements of WSEs were taken within process tanks, influent channels, effluent channels, primary distribution box and secondary bypass chamber. The Mohawk River water level was not determined, however it was observed that the level was relatively low and did not impact the WSEs within the WPCP (there was free flow through the effluent channel and flume).

The model calibration was performed by comparing the measured WSEs to the modeled WSEs for DWF and WWF conditions. The energy losses were slightly modified so the modeled WSEs better matched the measured WSEs. Table 5-7 summarizes the measured and modeled WSEs for several locations, and Figure 5-9 provides a profile of the WPCP for the modeled flows. Appendix C provides a full table of calibration results.

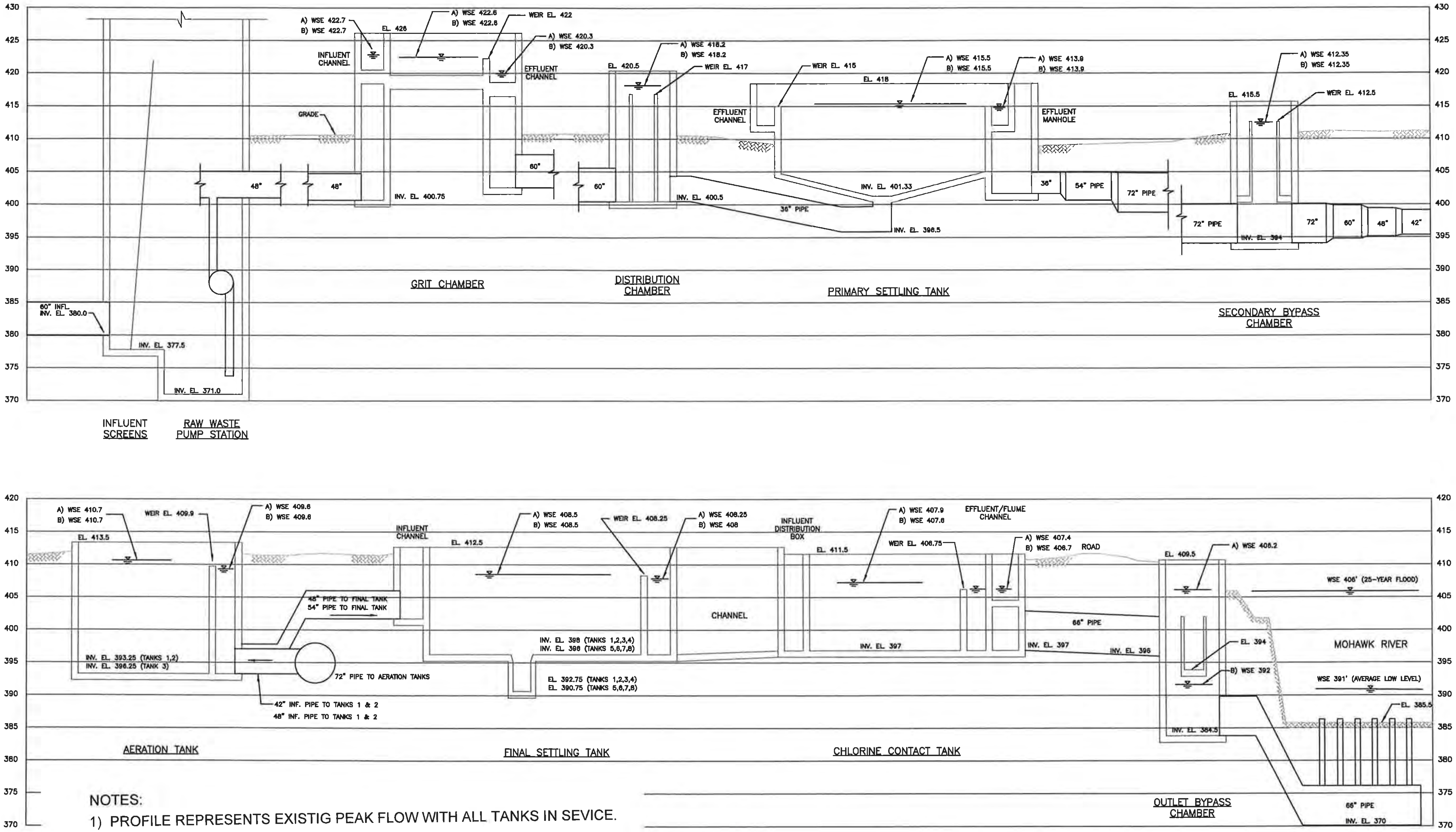
TABLE 5-7

SUMMARY OF WPCP HYDRAULIC PROFILE CALIBRATION

LOCATION	DWF OF 33 MGD		WWF OF 54 MGD	
	MEASURED WSE (FT)	MODELED WSE (FT)	MEASURED WSE (FT)	MODELED WSE (FT)
Grit Tank #1	422.4	422.4	422.5	422.6
Primary Distribution	417.8	417.8	418.2	418.0
Primary Tank #1	415.6	415.5	415.7	415.5
Secondary Bypass	411.2	411.2	411.9	412.0
Aeration Tank #1	410.5	410.5	410.7	410.7
Final Tank #8	408.5	408.4	408.5	408.5
Chlorine Contact Tank	407.3	407.4	407.5	407.6

5.9.2 Summary of Existing WPCP Hydraulic Capacity

The calibrated hydraulic model was used to determine the overall hydraulic capacity of the existing WPCP. The calculations were performed for winter operating conditions and for two (2)



NOTES:

1) PROFILE REPRESENTS EXISTIG PEAK FLOW WITH ALL TANKS IN SEVICE.
2) WSE A) IS 60 MGD AT 25-YEAR RIVER LEVEL.
3) WSE B) IS 60 MGD AT AVERAGE RIVER LEVEL.

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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

WPCP HYDRAULIC PROFILE
60 MGD

Job Number	8614782
Revision	A
Date	JUNE 2012

Figure 5-8

200 John James Audubon Parkway Suite 101, Amherst NY 14228 USA T 1 716 691 8503 F 1 716 691 8506 E amhmail@ghd.com W www.ghd.com

Plot Date: 22 August 2012 - 2:45 PM Cad File No: G:\86\14782\CADD\Drawings\Figures\FIGURES.dwg

receiving water levels. The winter operating condition includes all tanks in service and therefore can be considered the maximum hydraulic condition. This maximum hydraulic capacity was determined for the mean Mohawk River level (391') and the estimated 25-year Mohawk River level (406'). The flow in the model was increased until WSE violated one (1) of the following criteria:

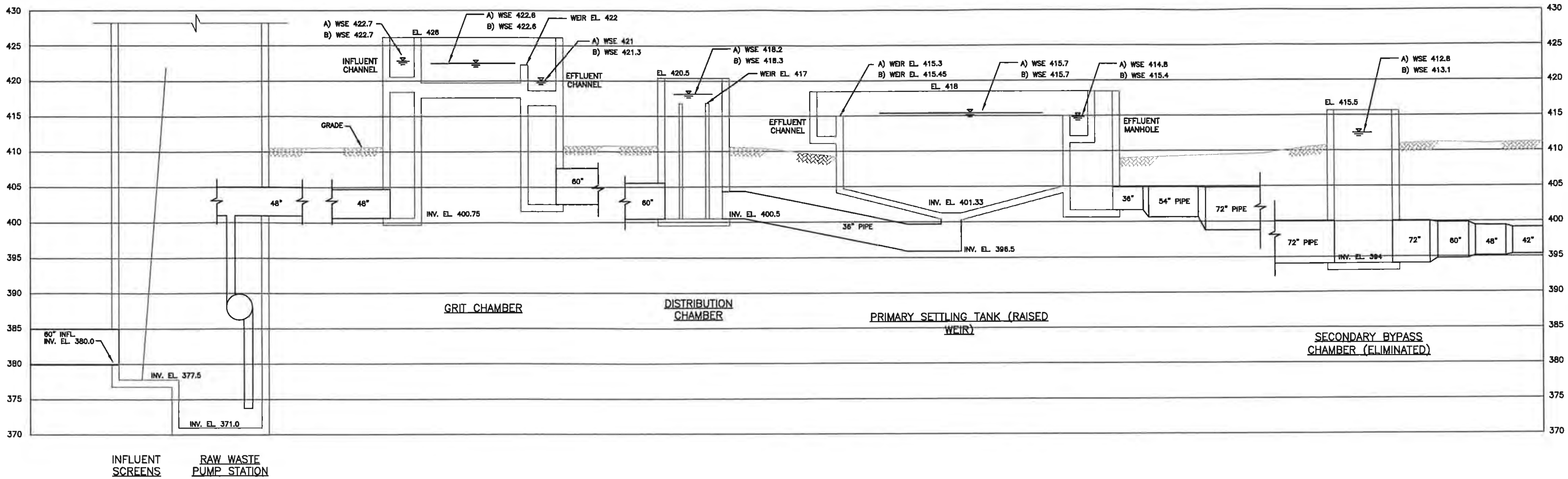
- No submergence of settling tank weirs
- Not less than 18 inches freeboard within aeration tanks
- Not less than 12 inches freeboard within channels and settling tanks
- No activation of secondary bypass

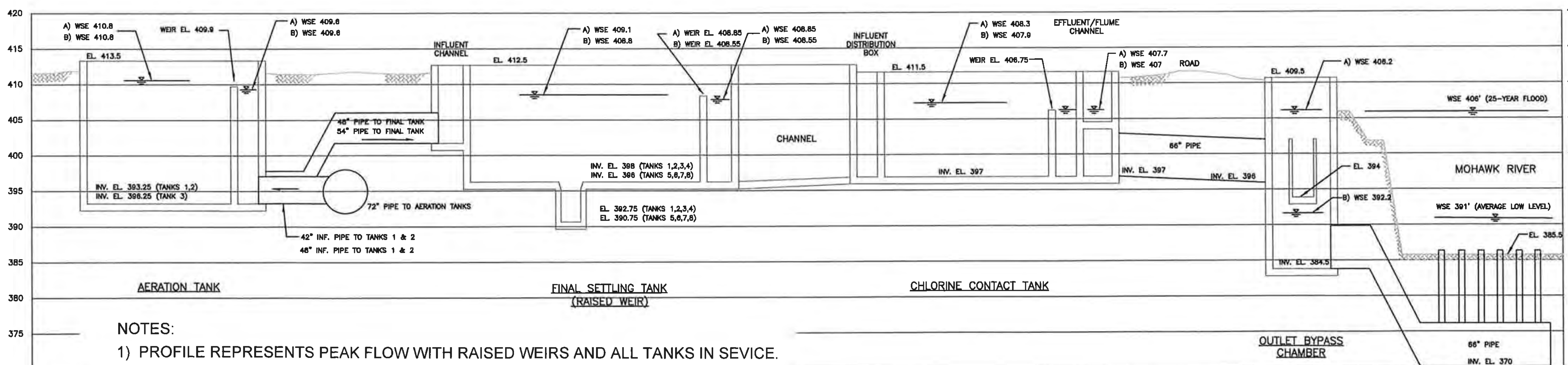
As soon as one of the criteria was violated then the capacity was reached. The existing overall WPCP capacity was found to be approximately 60 mgd, as shown in Table 5-8. Flows above 60 mgd begin to activate the secondary bypass weir. Additionally, for the 25-year river level, flows above 60 mgd begin to submerge the final settling tank weirs. This limitation of the secondary bypass weir is caused by restrictions within the aeration tank influent piping. The pipes which convey flow to the aeration tank influent channels do not allow for proper flow split. In order to keep proper flow balance between the three (3) aeration tanks, the influent gates to aeration tank #1 and #2 are adjusted (partially closed) so enough flow can get to aeration tank #3. The influent gate to aeration tank #3 is fully open.

TABLE 5-8

SUMMARY OF EXISTING WPCP HYDRAULIC CAPACITY

CONDITION	MAX. FLOW (MGD)	LIMITATION
Mean River Level	60	Sec. Bypass Activation
25-Year River Level	60	Sec. Bypass Activation; FST Weir Submergence





NOTES:

1) PROFILE REPRESENTS PEAK FLOW WITH RAISED WEIRS AND ALL TANKS IN SEVICE.
2) SECONDARY BYPASS ELIMINATED.
3) WSE A) IS 70 MGD AT 25-YEAR RIVER LEVEL.
4) WSE B) IS 75 MGD AT AVERAGE RIVER LEVEL.

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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

WPCP HYDRAULIC PROFILE
MODELED FLOWS

Job Number	8614782
Revision	A
Date	JUNE 2012

Figure 5-9

200 John James Audubon Parkway Suite 101, Amherst NY 14228 USA T 1 716 691 8503 F 1 716 691 8506 E amhmail@ghd.com W www.ghd.com

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5.9.3 Summary of Existing Process Hydraulic Capacity

The calibrated hydraulic model was also used to determine the hydraulic capacity of the unit processes. The calculations again assume winter operating conditions with all tanks in service, so therefore represent maximum capacity. In addition, the Mohawk River was assumed to be at an average level so not to affect the WSEs within the WPCP. The flow in the model was increased until WSE violated one (1) of the following criteria:

- No submergence of process weirs
- Not less than 18 inches freeboard within aeration tanks
- Not less than 12 inches freeboard within channels and settling tanks

As soon as one (1) of the criteria was violated then the capacity was reached. The existing process capacities were found to range from 60 to 85 mgd. For each process the limitation was found to be weir submergence of the process. The activation of the secondary bypass was not considered for the process capacities. Table 5-9 summarizes the capacities.

TABLE 5-9

SUMMARY OF EXISTING PROCESS HYDRAULIC CAPACITY

PROCESS	MAX. FLOW (MGD)	LIMITATION	COMMENTS
Grit Tanks	80	Weir Submergence	Weir submergence limitation is due to downstream restrictions.
Primary Settling Tanks	70	Weir Submergence	
Aeration Tanks	85	Weir Submergence; Influent channel freeboard	
Final Settling Tanks	70	Weir Submergence	
Chlorine Contact Tanks	60	Weir Submergence	

The chlorine contact tank capacity was found to be approximately 60 mgd. Restrictions in the WPCP effluent/flume channel cause weir submergence above this flow rate. However, submergence of the chlorine contact tank weir may not have any effect on the disinfection process.

The final settling tank capacity was found to be approximately 70 mgd. The final settling tank #8 weir submerges beyond this flow rate due to energy losses downstream.

The aeration tank capacity was found to be approximately 85 mgd. The aeration tank #1 weir submerges beyond this flow rate due to energy losses downstream including the FST influent gates and aeration tank effluent piping. The FST influent gates were opened slightly to allow more flow to pass while attempting to keep the proper flow splits to the FSTs. In addition, the WSE in aeration tank #1 influent channel violates the freeboard requirement of 12 inches. This is due to the energy loss associated with the influent gate. The AT #1 influent gates were also slightly opened more to allow more flow to pass while attempting to keep proper flow spilt to ATs.

The primary tank capacity was found to be approximately 70 mgd. The primary tank #1 weir submerges beyond this flow rate due to downstream restrictions in the effluent piping and aeration tank influent piping. As described previously, the aeration tank influent pipes do not allow for proper flow split so aeration tank influent gates are adjusted to balance flows between the tanks.

The grit tank capacity was found to be approximately 80 mgd. The grit tank #1 weir submerges beyond this flow rate.

6.0 ALTERNATIVES TO ALLEVIATE HYDRAULIC RESTRICTIONS

6.1 INTRODUCTION TO HYDRAULIC RESTRICTION ALTERNATIVES

The existing hydraulic capacity of the WPCP has been determined to be approximately 60 mgd. The hydraulic limitations are due mainly to secondary bypass weir activation and restrictions in the aeration tank influent piping. Several options were evaluated to remove hydraulic restrictions including raising weirs, modifying flow distribution to aeration tanks, and modifying yard piping. These options were evaluated with all tanks in service and with mean river level. The flow in the model was increased until WSE violated one of the following criteria:

- No submergence of settling tank weirs
- Not less than 18 inches freeboard within aeration tanks
- Not less than 12 inches freeboard within channels and settling tanks

6.2 RAISE WEIRS

The simplest option for increasing hydraulic capacity of the WPCP is to raise/eliminate the secondary bypass weir and raising process weirs. For this evaluation the secondary bypass weir was removed, and it was assumed that the chlorine contact tank weir would be allowed to submerge. The evaluation was performed for mean river level and for the 25-year river level. The flow in the WPCP was increased incrementally and process weirs were raised as much as possible.

The results of this evaluation, summarized in Table 6-1, resulted in a peak flow of 75 mgd for mean river level and 70 mgd for 25-year river level. These flows were achieved by raising the FST weirs and the PST weirs. In addition, the FST and AT influent gates were opened slightly to allow more flow to pass while attempting to keep the appropriate flow balance. The limiting factor was the freeboard in the AT #1 influent channel. This is due to the poor flow split between the ATs which could be mitigated with a new AT influent distribution box.

Raising the FST weirs will require that the scum skimming systems also be raised. The would also entail raising the upper return cog on the sludge scrapers and adding additional links to extend the chains.

TABLE 6-1

SUMMARY OF POTENTIAL WEIR MODIFICATIONS

PROCESS	AMOUNT WEIR RAISED (FT)		COMMENTS
	75 MGD; MEAN RIVER LEVEL	70 MGD; 25-YEAR RIVER LEVEL	
Grit Tanks	0	0	Limiting factor was freeboard in AT #1 influent channel.
Primary Settling Tanks	0.15	0	
Aeration Tanks	0	0	
Final Settling Tanks	0.25	0.55	
Chlorine Contact Tanks	N/A	N/A	

The probable project cost for raising weirs, including raising sludge scrapers and modifying chains on the FSTs, is detailed on Table 6-2.

TABLE 6-2

ENGINEERS OPINION OF PROBABLE COST: WEIR MODIFICATIONS

DESCRIPTION	PROBABLE COST (1)
Raise Final Settling Tank Weirs, Chain/Flights, Scum Baffles	$120,000
Raise Primary Clarifier Weirs	$40,000
Subtotal	**$160,000**
General Conditions, Bonds & Insurance (5% of Subtotal)	**$10,000**
Contingency (20%)	**$35,000**
Total Probable Construction Cost	***$205,000***
Engineering, Administrative, and Legal (20%)	**$45,000**
Total Probable Project Cost (Rounded)	***$250,000***

(1) Year 2012 dollars

6.3 AERATION TANK INFLUENT PIPING

A major hydraulic limitation exists in the influent piping to the ATs. The pipes which convey flow to the aeration tank influent channels do not allow for proper flow split without creating significant energy losses. In order to keep proper flow balance between the three aeration tanks, the influent gates to AT #1 and #2 are adjusted (partially closed) so enough flow can get to AT #3. The influent gate to AT #3 is fully open. The energy losses associated with the AT influent causes the secondary bypass to activate at flows above 60 MGD.

An option to replace the existing AT influent piping was evaluated. The existing influent piping includes a header pipe which ranges in size from 72” to 48”. Each aeration tank is connected to the header pipe with a 42” pipe which is flow metered. For this evaluation the header pipe was replaced with a single 72” pipe. The aeration tank connections were replaced with 60” pipes. These upsized pipes have allowed flow to be conveyed to AT #3 while reducing energy losses. The influent gates for AT #1 and #2 could be opened more to rebalance flows and reduce WSE in the AT influent channels and in the secondary bypass chamber.

As shown in Table 6-3, modifying the AT influent piping resulted in a maximum capacity of 70 MGD and 60 MGD for mean river level and 25-year river level, respectively. For each condition, submergence of the FST weirs was the limiting factor and the secondary bypass weir was not activated.

TABLE 6-3

SUMMARY OF EXISTING WPCP HYDRAULIC CAPACITY

CONDITION	MAX. FLOW (MGD)	COMMENTS
Mean River Level	70	Limiting factor was FST weir submergence.
25-Year River Level	60	

The probable project cost for modifying the AT influent piping is shown on Table 6-4.

TABLE 6-4

ENGINEERS OPINION OF PROBABLE COST: AERATION BASIN INFLUENT PIPING MODIFICATIONS

DESCRIPTION	PROBABLE COST [1]
Replace Header Piping	$220,000
Connections from Header to Aeration Basins	$60,000
Connections from Primary Settling Tanks to Header	$10,000
Connections from Final Settling Tanks to Header	$10,000
Subtotal	**$300,000**
General Conditions, Bonds & Insurance (5% of Subtotal)	**$15,000**
Contingency (20%)	**$65,000**
Total Probable Construction Cost	***$380,000***
Engineering, Administrative, and Legal (20%)	**$80,000**
Total Probable Project Cost (Rounded)	***$500,000***

(1) Year 2012 dollars

6.4 YARD PIPING

While yard pipe sizes will not need to be modified to meet the hydraulic capacities described, rerouting of yard piping will be necessary to facilitate new tankage. It is expected that pipe sizing between structures will remain unchanged.

7.0 ALTERNATIVES TO INCREASE WPCP CAPACITY

7.1 DEVELOPMENT OF WPCP ALTERNATIVES

Several alternatives were developed and evaluated for increasing the process capacity at the WPCP. The alternatives evaluated in detail in this Section are summarized in Table 7-1.

TABLE 7-1

SUMMARY OF WPCP EXPANSION ALTERNATIVES

ALTERNATIVE NO.	GENERAL DESCRIPTION	COMMENT
1	Conventional WPCP Expansion	Expand the plant with similar units as existing
2	Split Flow Concept	Wet weather operating strategy to maximize capacity
3	Aeration Operation Modifications	Change the mode of operation utilized at the aeration basins
4	Integrated Fixed Film Activated Sludge (IFAS)	Install media in aeration basins to supplement activated sludge system with attached growth treatment
5	High Rate Ballasted Flocculation	Utilize microsand or dense sludge as a ballast to promote more efficient settling
6	Solids Handling Alternatives	Various alternatives to modify existing solids handling operations
7	Solids Disposal Alternatives	Various alternatives to continued use of incinerators
8	Non WPCP Expansion Alternatives	Various upgrades which would be required regardless of expanding the plant. Based on useful remaining life of existing equipment and 2010 physical condition assessment.
9	Electrical Alternatives	Improvements necessary to provide electrical capacity for expanded WPCP
10	Other Considerations	Modifications to existing influent piping and constructability

7.2 ALTERNATIVE 1: EVALUATION OF CONVENTIONAL WPCP UPGRADE

When evaluating expansion of the WPCP to accommodate additional flows and loads, the first approach was a conventional expansion. A conventional expansion would include additional tanks with a similar size and capacity as existing units. The quantity of additional process tanks would be based on the required peak flow capacity divided by the capacity per tank of the existing units. Based on this methodology, the required number of new units for the projected peak flow rates of 91 mgd or 111 mgd is presented in Table 7-2.

TABLE 7-2

REQUIRED NEW PROCESSES FOR CONVENTIONAL WPCP EXPANSION

PROCESS	NO. EXISTING UNITS (1)	CAPACITY PER UNIT (MGD) (2)	REQUIRED NEW UNITS AT PEAK FLOW OF 91 MGD	REQUIRED NEW UNITS AT PEAK FLOW OF 111 MGD
Influent Bar Screens	2	36	1	2
Raw Waste Pumps	3	33	0	1
Grit Tanks	2	32	1	2
Primary Settling Tanks	4	17	2	3
Aeration Basins	(3)	70 (4) 90 (5)	1 New Large Basin OR Switch to Step-Feed Aeration	1 Large Basin AND switch to Step-Feed Aeration
Final Settling Tanks	(3)	56 (6)	63% Increase in Surface Area	100% Increase in Surface Area
Chlorine Contact Tank	2	50	0	1

(1) For units where redundancy is required (i.e. pumps, etc.), existing units refers to units in operation at peak flow

(2) Refer to Section 5 for capacity analysis

(3) Existing aeration basins and final settling tanks include multiple units of various sizes. For these two processes, this Table represents peak flow through the process

(4) Utilizing conventional aeration

(5) Utilizing step-feed aeration

(6) Based on CFD modeling and MLSS concentration of 3,000 mg/L.

Table 7-2 anticipates the hydraulic restrictions within the WPCP are addressed and removed, which may not be feasible with a conventional expansion.

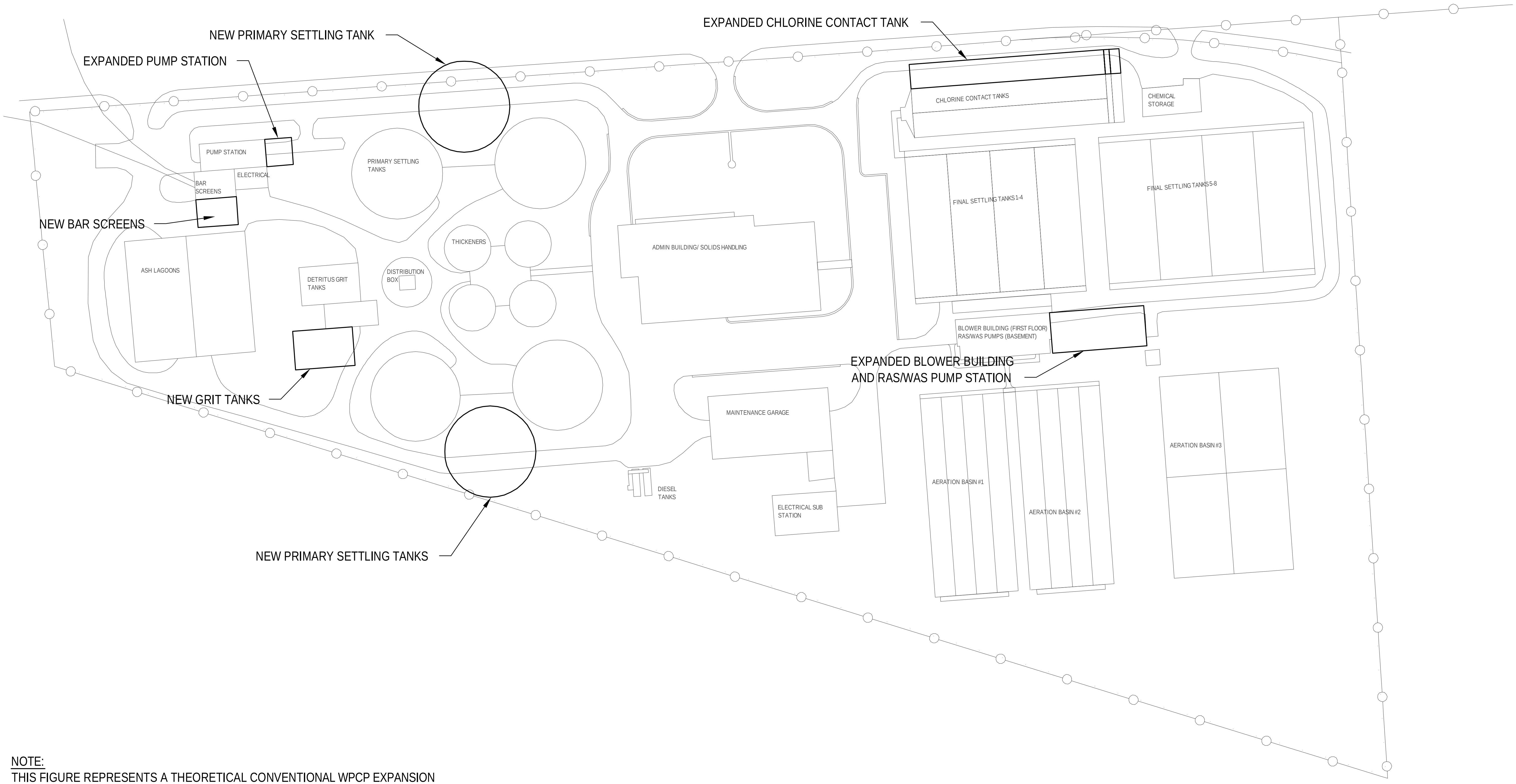
The required new tankage for a conventional WPCP expansion to a peak flow of 91 mgd or 111 mgd are shown on Figures 7-1A and 7-1B, respectively. As evidenced by these figures, physical space at the WPCP is not available to accommodate a conventional expansion within the property (fence) line. Since the site is bounded by the River, the solid waste facility, and the railroad, there is no opportunity to increase the overall footprint of the site. For this reason, and due to the significant hydraulic restrictions within WPCP's existing hydraulic profile, a conventional expansion of the plant was not evaluated in any greater detail than described above.

7.3 ALTERNATIVE 2: EVALUATION OF SPLIT FLOW CONCEPT

Since a conventional expansion of the WPCP is not feasible due to site limitations, several alternatives were evaluated which would provide more treatment capacity but not necessarily require extensive additional tankage. These innovative approaches are necessary due to the very limited physical space available to expand the plant at its current location. All approaches considered would provide the required degree of treatment as defined in Section 3.9. The first such approach involves the "split flow" concept, whereby the WPCP would employ a wet weather operating strategy to maintain separate treatment trains for the combined and sanitary influent sources.

7.3.1 Split Flow Concept Introduction

Two (2) sewer mains currently feed the WPCP. The larger, 60-inch main contains sanitary flow from the Sauquoit Creek Pumping Station (SCPS) and combined sewer flows in the Mohawk River Interceptor (MRI) from central, south, and west Utica. Connecting to the 60-inch main near the water pollution control plant (WPCP) is the Starch Factory Creek Interceptor, which conveys sanitary flow from the western end of the Town of Frankfort, the eastern most section of Utica,



NOTE:
THIS FIGURE REPRESENTS A THEORETICAL CONVENTIONAL WPCP EXPANSION FOR A PEAK FLOW OF 111 MGD. NOT ALL TANKS WOULD FIT ON EXISTING SITE.

0 60' 120' 180'
SCALE 1"=120' AT ORIGINAL SIZE

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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

CONVENTIONAL WPCP EXPANSION
PEAK FLOW OF 91 MGD

Job Number	8614782
Revision	A
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Figure 7-1(A)

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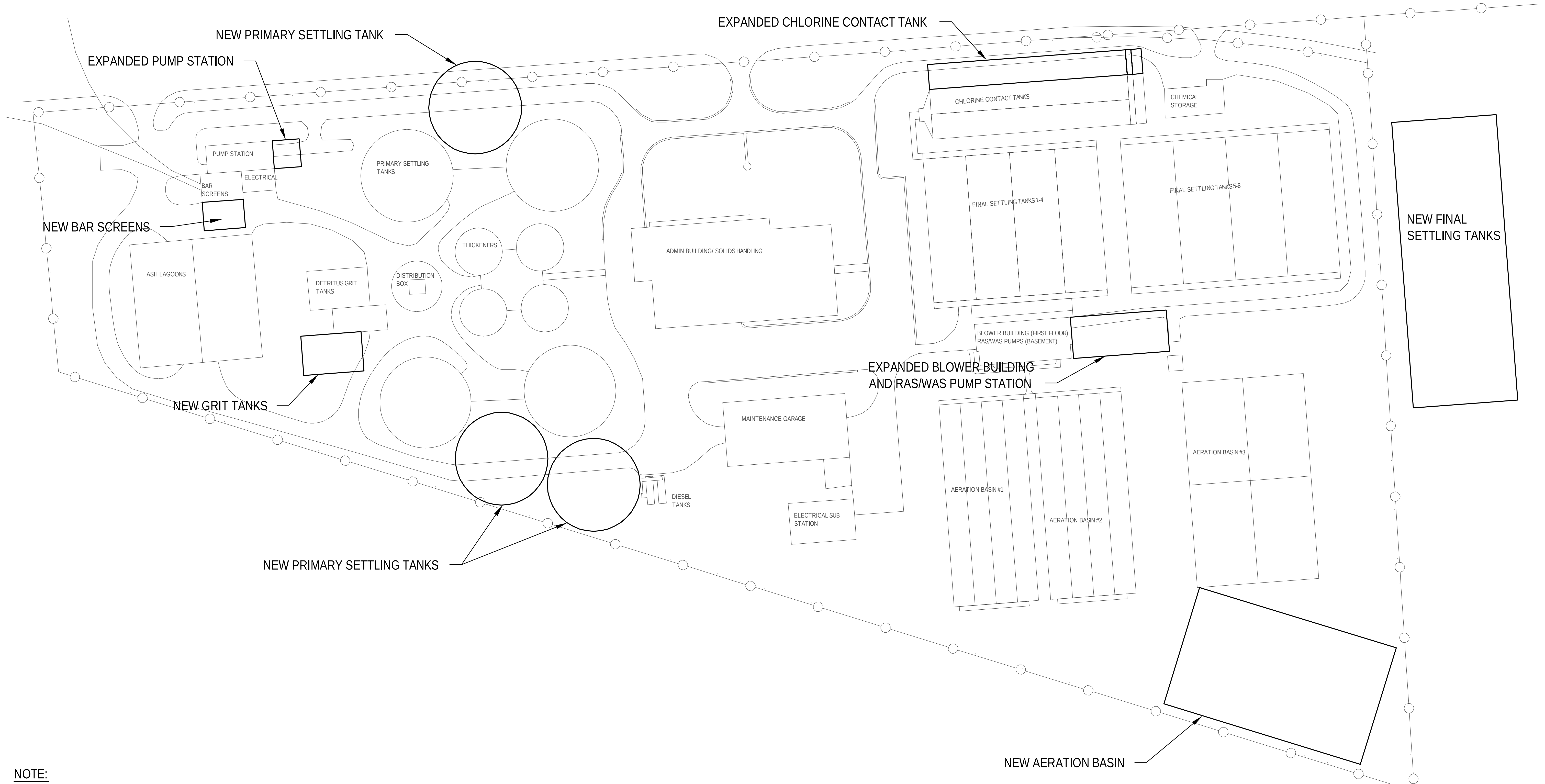
Plot Date: 22 August 2012 - 1:46 PM Cad File No: G:\86\14782\CADD\Drawings\Figures\SITE PLAN CONVENTIONAL METHODS.dwg



NOTE:
THIS FIGURE REPRESENTS A THEORETICAL CONVENTIONAL WPCP EXPANSION FOR A PEAK FLOW OF 111 MGD. NOT ALL TANKS WOULD FIT ON EXISTING SITE.

0 60' 120' 180'
SCALE 1"=120' AT ORIGINAL SIZE

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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

CONVENTIONAL WPCP EXPANSION
PEAK FLOW OF 111 MGD

Job Number	8614782
Revision	A
Date	06/12

Figure 7-1(B)

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Plot Date: 22 August 2012 - 1:47 PM Cad File No: G:\86\14782\CADD\Drawings\Figures\SITE PLAN CONVENTIONAL METHODS.dwg and an easterly portion of the Town of New Hartford. A 42-inch main conveys sanitary flows from the north, including North Utica, the Towns of Marcy, Deerfield, and Schuyler, and the Village of Holland Patent. During wet weather events, the WPCP receives flows that are larger than the plant's capacity, which has historically been limited to 55 mgd (secondary treatment system). For influent flows above 55 mgd, a slide gate on the 60-inch main is lowered, which restricts flow to the plant, causing excess flow to back up and discharge through Utica's primary CSO located on Leland Avenue just upstream of the WPCP. As a potential option to minimize these CSOs from happening, we have reviewed a wet weather operation concept by which flows through the plant could be rerouted to achieve minimum treatment levels for wastewater entering the WPCP.

The basis of the split flow concept is to separate out the combined sewer flows of the MRI from the remaining sanitary sewer flows to the WPCP. This separation allows for distinct and appropriate treatment trains during peak wet weather events and reduces the capacity requirements of secondary treatment facilities. Two (2) solutions were evaluated as part of this split flow concept:

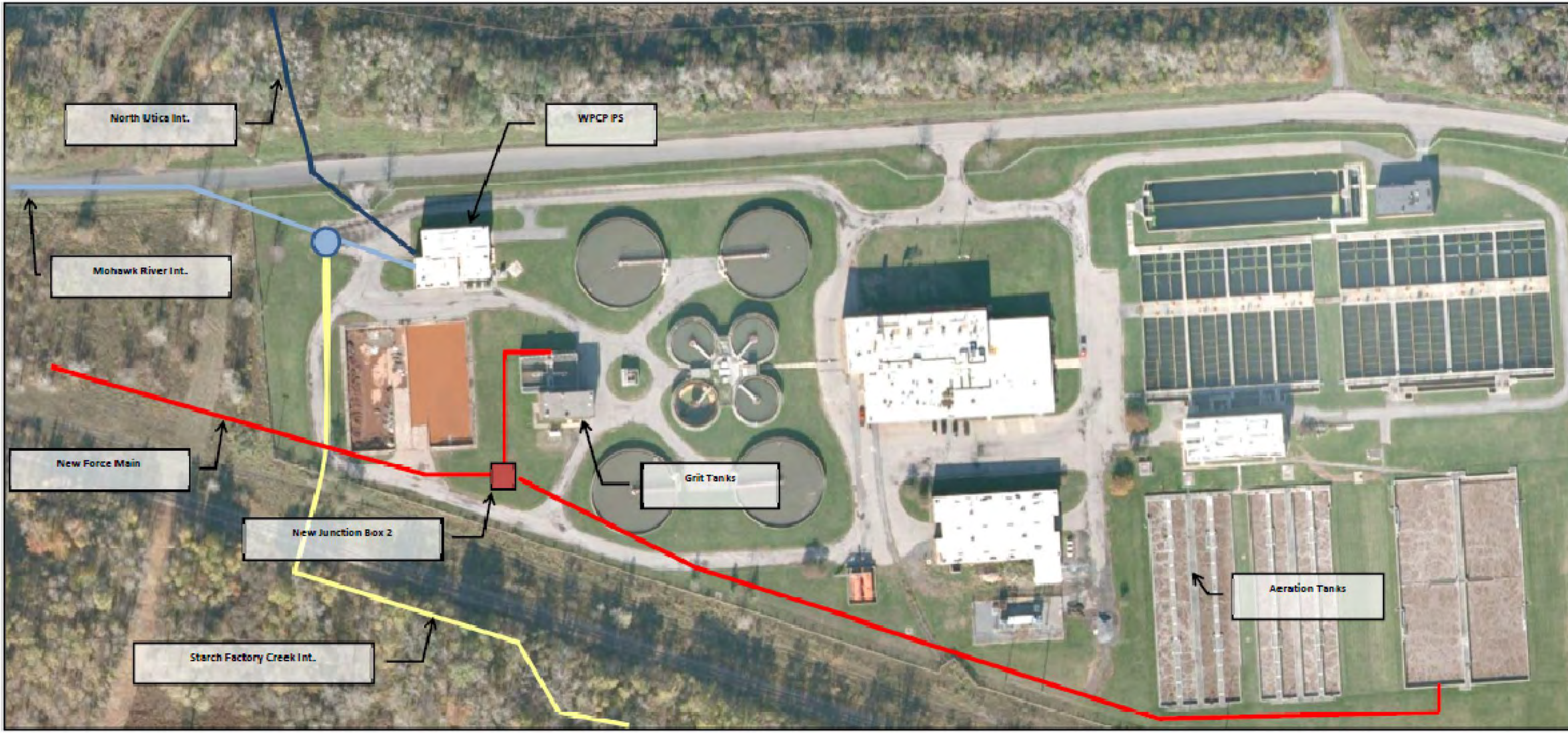
1. a temporary interim solution to limit CSOs that could be implemented on a fast track schedule
2. a permanent solution to elimate CSOs which incorporates a complete separation of combined sewer flows in the MRI.

The interim solution has since been dismissed as a viable option, but is presented in this report as background. Both the interim solution and the permanent solution are described in more detail in the subsequent sections.

7.3.2 Split Flow Concept Interim Solution

Previous modeling of the SCPS had shown that an increase in capacity of 5 mgd at the SCPS would significantly reduce the number of sanitary sewer overflows upstream of the WPCP. The purpose of the interim flow split solution at the WPCP was to increase the flow from the SCPS from a peak of approximately 15 mgd to a peak of approximately 20 mgd in order to reduce sanitary sewer overflows at the SCPS. This interim solution could be implemented on a shorter timeline than the WPCP improvements and provide some benefit of reduced sewer overflows while permanent improvements where ongoing at the WPCP. The interim solution would include a new forcemain from the SCPS to the WPCP and junction boxes described herein, which would be incorporated into the permanent solution in the future. Refer to Figure 7-2 for a schematic of the interim solution.

Execution of the interim solution would require new construction. A new 36-inch forcemain from the SCPS to Junction Box 1 (JB 1) would be constructed to provide additional flow capacity from the SCPS. A new 48-inch forcemain from JB 1 to a new JB 2, located at the WPCP, would allow the new forcemain to bypass the MRI and convey flow directly to the WPCP. JB 1 would be located at the point where the existing SCPS forcemain empties into the MRI. The purpose of this structure would be to isolate both forcemains, divert flow to the MRI, or direct flow to JB 2. A plan view sketch of JB 1 and JB 2 are shown in Figures 7-3 and 7-4, respectively. JB 2 would direct flow from the SCPS to either the existing grit system or to the secondary treatment process, bypassing the primary treatment process.



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WPCP AND SCPS EVALUATION
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SPLIT FLOW
INTERIM SOLUTION

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Date	JUNE 2012

Figure 7-2

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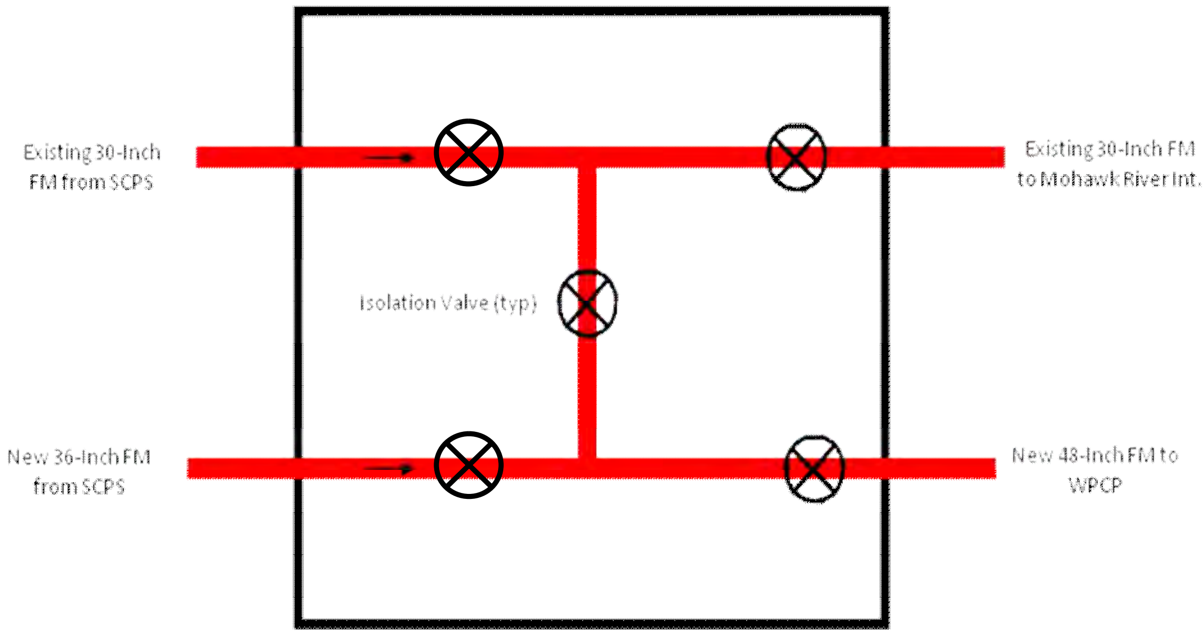


FIGURE 7-3

SPLIT FLOW JUNCTION BOX 1

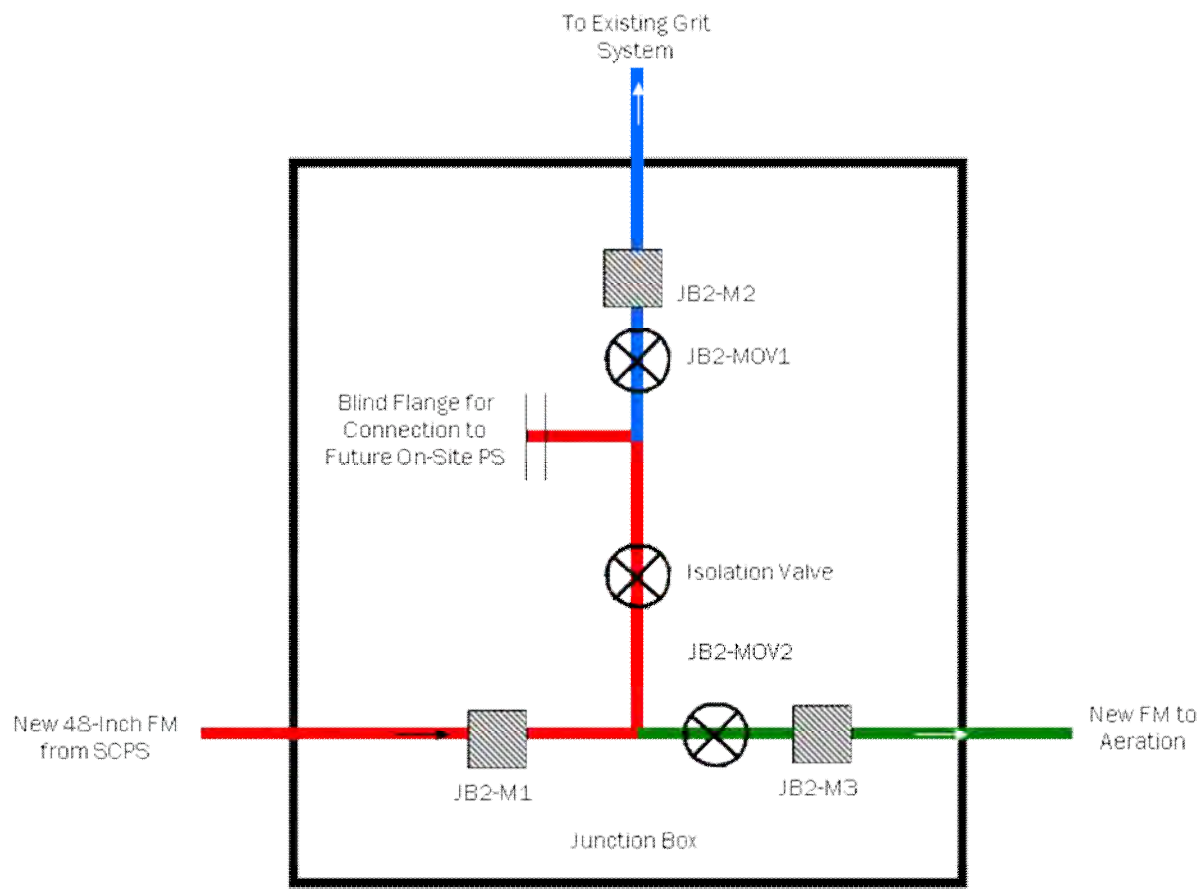


FIGURE 7-4

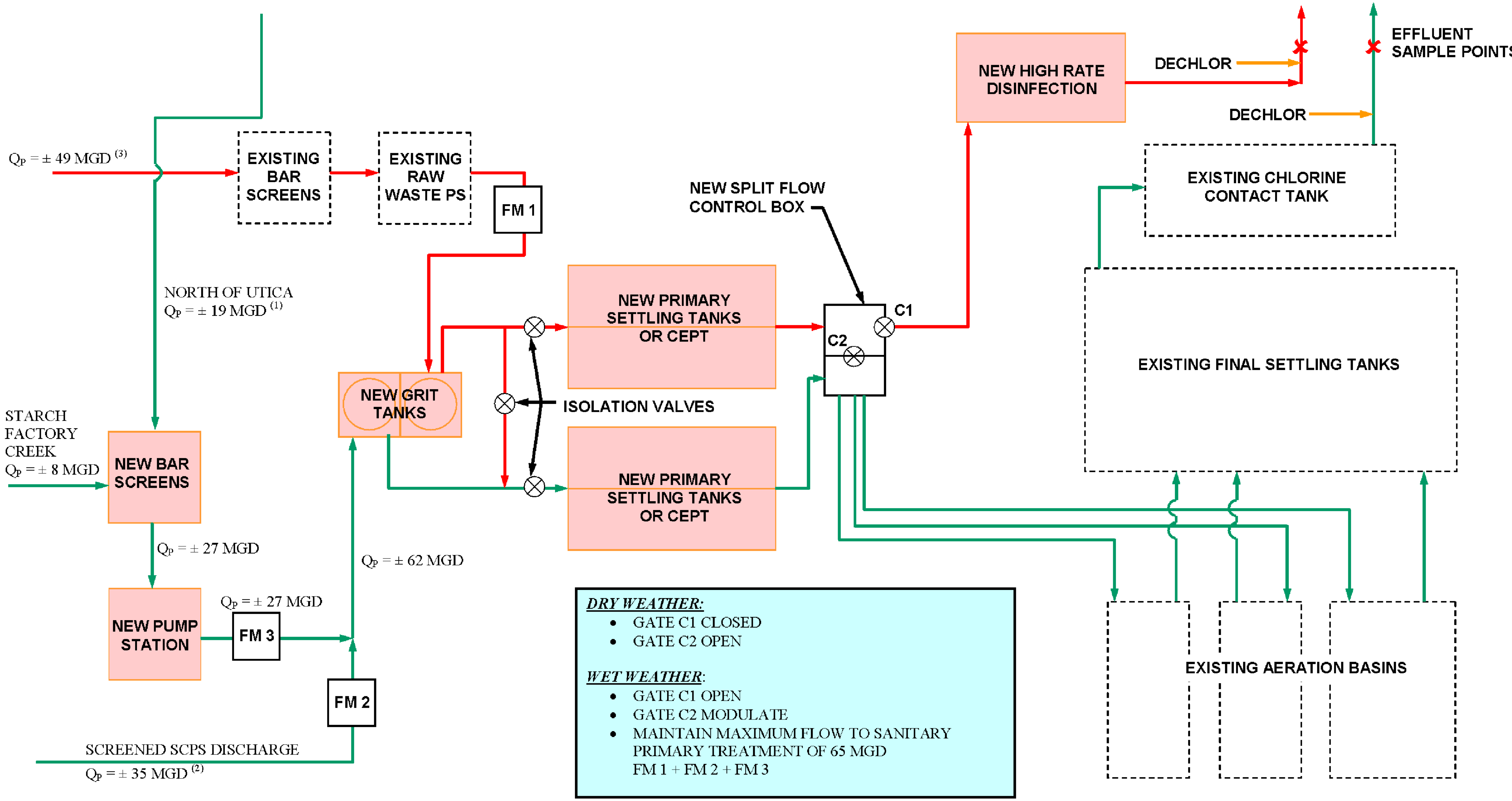
SPLIT FLOW JUNCTION BOX 2

The interim solution does have some system limitations. Under high flow conditions, both the existing and new forcemains must be in operation. Additionally, not all equipment and new construction could be reused for the permanent solution. For example, the pipe carrying flow from JB 2 to the aeration system would not be used as part of the permanent solution and would be abandoned and the pipe carrying flow from JB 2 to the existing grit system would either be oversized for the interim solution or would need to be replaced as part of the permanent solution. In addition, further hydraulic capacity analysis of the primary clarifiers and aeration basins has indicated that more primary clarification capacity is required and incorporation of an additional wet weather bypass flow of 5 mgd to 20 mgd to the aeration basins may require more significant physical modifications to the existing basins. Due to these limitations of the interim solution, this option was eliminated from consideration.

7.3.3 Split Flow Concept Permanent Solution

For simplicity, this section assumes the peak flow to the WPCP will be 111 mgd. However, the split flow operational strategy would be the same if the total flow were 91 mgd.

The purpose of the permanent flow split solution at the WPCP is to increase the flow from the SCPS from a peak of 15 mgd to a peak of 35 mgd in order to reduce sanitary sewer overflows at the SCPS, and to separate the combined sewer flows of the MRI to adequately treat wet weather flows from the MRI in separate treatment facilities. A distinct treatment train for the MRI combined sewer flows allows for appropriate treatment of combined sewer flows during wet weather and maximizes the use of the existing secondary treatment facilities (aeration basins and final settling tanks) for sanitary sewer flows, so that new secondary treatment facilities are not required. The permanent solution will incorporate the new forcemain from the SCPS and junction boxes that are part of the proposed interim solution. Refer to Figure 7-5 for a schematic of the permanent split flow solution.



NOTES:
(1) North of Utica Interceptor includes existing NOU peak flow (±10 mgd) plus microchip plant effluent (±6 mgd) plus microchip "spin-off" (±3 mgd)
(2) SCPS includes existing peak flow (±15 mgd) plus 20 mgd additional sanitary flow to mitigate SSO.
(3) Peak flow of 49 mgd from City of Utica exceeds 85% annual capture for the City.

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WPCP AND SCPS EVALUATION
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SPLIT FLOW SCHEMATIC

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Figure 7-5

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The permanent solution requires modifications to the existing WPCP. New construction will include a new below grade influent pump station, sized to pump approximately 27 mgd. All flow from the North Utica interceptor and Starch Factory interceptor will be redirected to the new influent pump station. A screening structure, sized for all flow from North of Utica and Starch Factory will be constructed upstream of the pump station. The screening structure will include two bar screens, each with a capacity of passing 27 mgd. A screenings washer/compactor will also be installed in the screening structure. All screening related equipment will be on emergency power. Similar screening facilities will be included at the SCPS separately for 35 mgd. The total flows of 62 mgd from the North of Utica, Starch Factory, and SCPS will be conveyed to the new grit removal system.

Flow from the MRI (combined flow) will enter the existing WPCP screening structure and then flow to the existing influent pump station. These systems may be upgraded as part of a rehabilitation project. Screened effluent will discharge to the new combined sewer grit removal system.

Two new grit removal systems (one for combined flows and one for sanitary flows) will be constructed adjacent to the existing grit building. Each grit system will include two (2) vortex type grit removal tanks each sized at half the peak flow, with provisions for bypassing if one unit is out of service. Combined flow from the MRI will be directed to two (2) units each sized for approximately 25 mgd, and sanitary flow from the new pump station and SCPS will be directed to two (2) units each sized for approximately 31 mgd. There will be no comingling of combined and sanitary flow at this point. Flow splitting structures (automated weir gates), downstream of primary settling tanks, will divert combined flow to the sanitary flow stream to maximized secondary treatment, when sanitary flow is less than 65 mgd. A schematic of the automated weir gates after primary settling tanks is provided on Figure 7-6A, and the hydraulic profile through the WPCP with the split flow alternative is provided on Figure 7-6B. Upstream of the primary settling tanks, valves or gates could be installed for isolation and taking banks of settling tanks out of service during low flow conditions for scheduled maintenance.

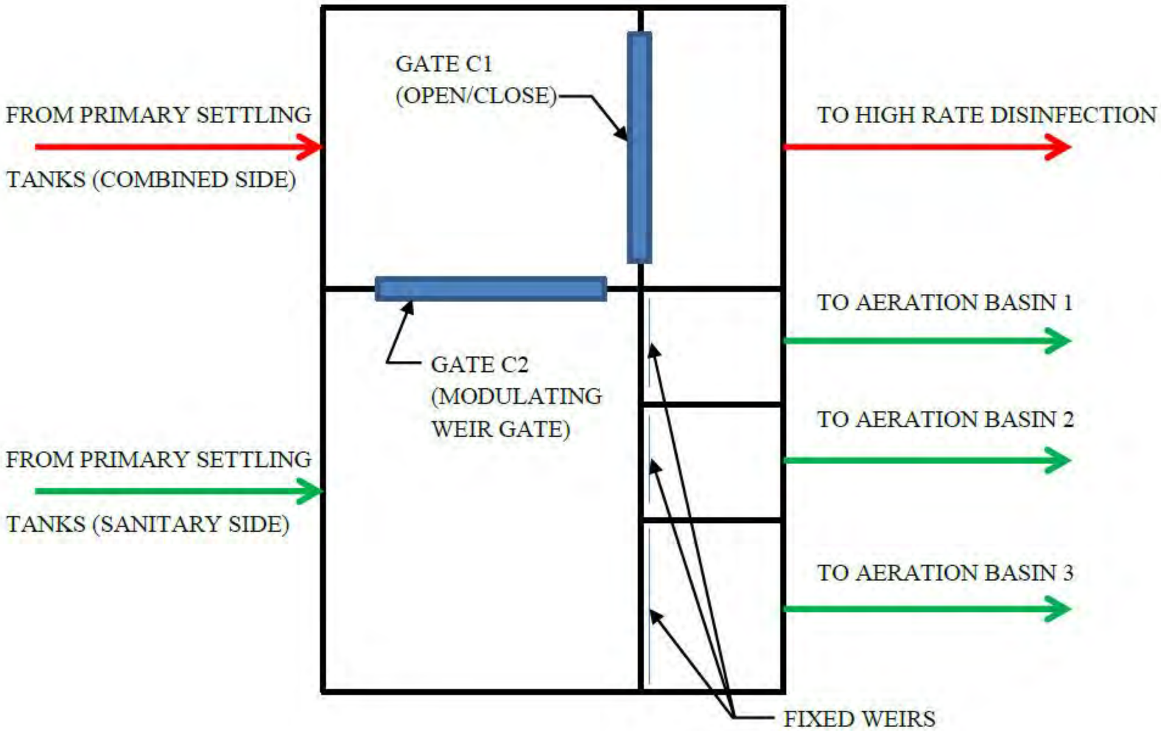
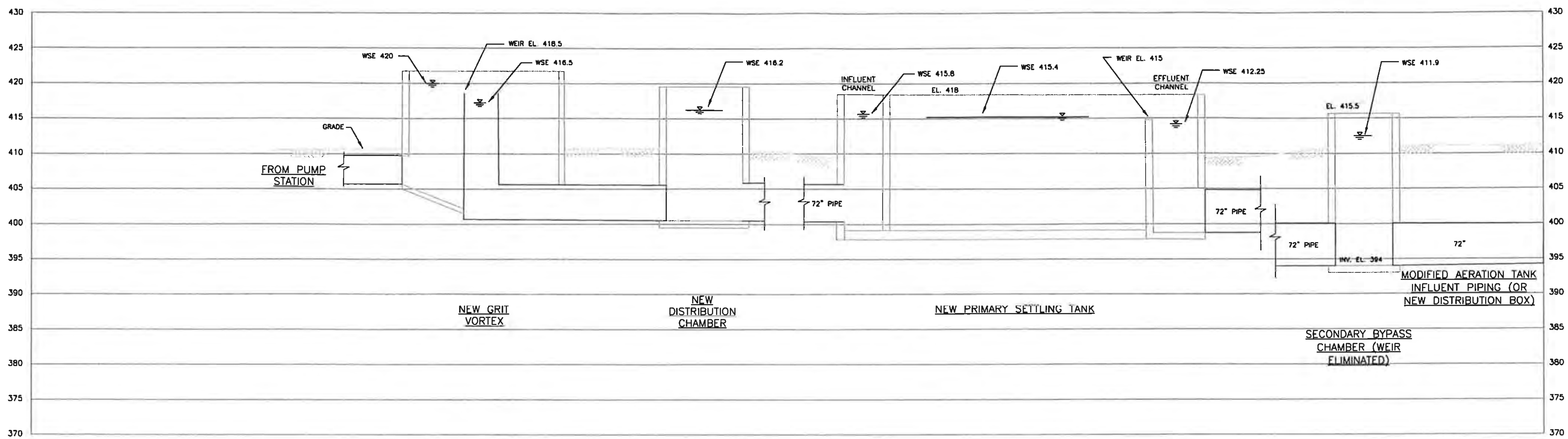
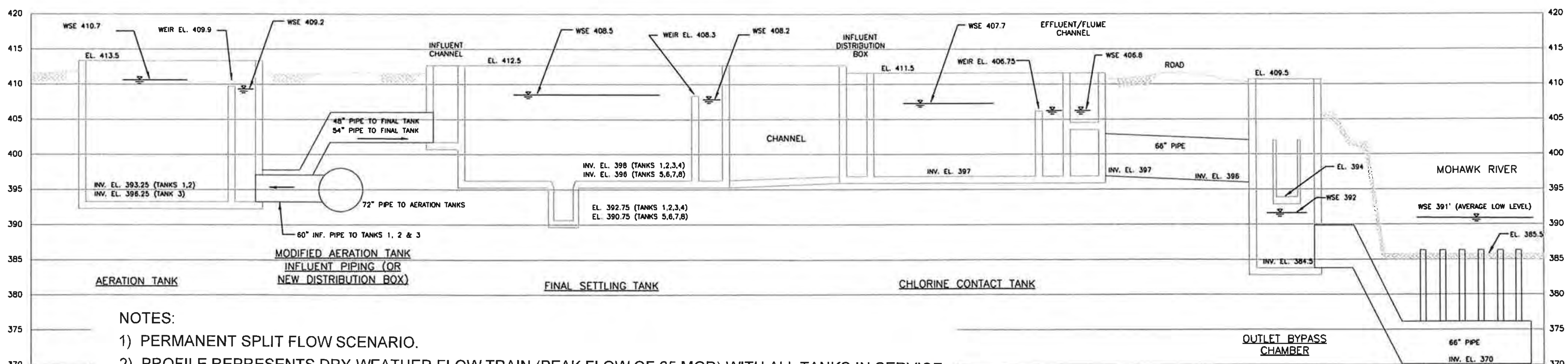


FIGURE 7-6A

SPLIT FLOW DISTRIBUTION BOX





NOTES:

1) PERMANENT SPLIT FLOW SCENARIO.
2) PROFILE REPRESENTS DRY-WEATHER FLOW TRAIN (PEAK FLOW OF 65 MGD) WITH ALL TANKS IN SERVICE.
3) NEW GRIT AND PRIMARY TANKS, AND EXISTING SECONDAY TANKS.
4) SECONDAY BYPASS ELIMINATED, AND MODIFIED AETRATION INFLUENT.

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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

WPCP HYDRAULIC PROFILE
SPLIT FLOW ALTERNATIVE

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Figure 7-6(B)

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The probable project cost for the new distribution box is shown in Table 7-3.

TABLE 7-3

ENGINEERS OPINION OF PROBABLE COST: SPLIT FLOW DISTRIBUTION BOX

DESCRIPTION	PROBABLE COST (1)
Weirs	$200,000
Excavation	$50,000
Backfill	$50,000
Concrete Walls	$100,000
Concrete Slab	$120,000
Miscellaneous Metals	$80,000
Subtotal	**$600,000**
Electrical, Controls, and Instrumentation (15% of Subtotal)	**$90,000**
General Conditions, Bonds & Insurance (5% of Subtotal)	**$30,000**
Contingency (20%)	**$150,000**
Total Probable Construction Cost	***$870,000***
Engineering, Administrative, and Legal (20%)	**$170,000**
Total Probable Project Cost (Rounded)	***$1,000,000***

(1) Year 2012 dollars

Instrumentation for system controls would be required for the permanent solution as follows and illustrated schematically on Figures 7-5 and 7-6:

- Raw Waste Pump Station Flowmeter (FM 1)
- SCPS Discharge Flowmeter (FM 2)
- Sanitary Pump Station Flowmeter (FM 3)
- Gate C-1 motor operated control weir gate(s)
- Gate C-2 motor operated control weir gate(s)

Flow meters will be installed on the existing raw waste pump station (FM 1), the SCPS discharge forcemain (FM 2), and the new sanitary pump station (FM 3). Flow to the sanitary primary clarifier will be maintained at 65 mgd or less. When total flows to the WPCP (FM 1 + FM 2 + FM 3) are less than 65 mgd, weir Gate C-1 is fully open and weir Gate C-2 is fully closed, and combined and sanitary flows will be discharged to the sanitary primary clarifier. When flows (FM 1 + FM 2 + FM 3) exceed 65 mgd (Storm Flow Mode), weir gate C-2 will open and gate C-1 will modulate so that a portion of the combined flow will be discharged to the combined flow primary clarifier, and the flow to the sanitary primary clarifier will be held at 65 mgd. When sanitary flows reach the maximum of 65 mgd, all combined flow will be directed to the combined flow primary clarifier. In Storm Flow Mode, weir gate C-2 will be used to raise the hydraulic grade line in the combined flow chamber so that sufficient head is developed to convey combined flow into the sanitary flow chamber.

Two (2) new rectangular primary clarifiers will be constructed in the location of the existing circular clarifiers. One (1) set of clarifiers will be sized for approximately 49 mgd for combined flow from the MRI. A second, larger set of primary clarifiers will be sized for 62 mgd, and will handle sanitary flow and a combination of combined and sanitary flows during dry weather events when the combined and sanitary flow is less than 65 mgd. This will maximize flows that receive secondary treatment. Both clarifiers will include a distribution box at the head of the primary clarifier gallery to split flows between six (6) trains of each clarifier.

Flow from the sanitary primary clarifier will be discharged to the existing secondary process. Flow from the combined flow primary clarifier will discharge to the high rate disinfection system and then to the wet weather outfall.

A new wet weather disinfection system will be constructed for high rate disinfection of combined flows during wet weather events. This system will include a contact tank and chemical feed systems. Discharge from this disinfection system will go to a dedicated wet weather outfall. The disinfection system will be a high rate process with chemical added at the discharge of the primary clarifiers.

The disinfection chamber would be designed for a peak flow of 49 mgd. The flow would be dosed at 5-10 mg/L sodium hypochlorite with a 5-minute contact time. The chamber would be 240-feet long, 15-feet wide, and 8-feet deep. The discharge from the disinfection chamber would be directed towards an outfall.

An existing bypass outfall will be evaluated for use as an outfall for the disinfection discharge. This 200-foot outfall will need to be evaluated using a camera for visual inspection. An ultrasonic inspection may need to follow if the visual inspection is inconclusive. The probable cost for the high rate disinfection chamber is presented in Table 7-4.

TABLE 7-4

ENGINEERS OPINION OF PROBABLE COST: HIGH RATE DISINFECTION CHAMBER

DESCRIPTION	PROBABLE COST (1)
Excavation	$50,000
Backfill	$40,000
Concrete Walls	$150,000
Concrete Slab	$170,000
Chemical Storage and Feed System	$1,000,000
High Rate Mixer	$250,000
Miscellaneous Metals	$80,000
Subtotal	**$1,740,000**
Electrical, Controls, and Instrumentation (15% of Subtotal)	**$260,000**
General Conditions, Bonds & Insurance (5% of Subtotal)	**$90,000**
Contingency (20%)	**$420,000**
Total Probable Construction Cost	***$2,510,000***
Engineering, Administrative, and Legal (20%)	**$500,000**
Total Probable Project Cost (Rounded)	***$3,000,000***

(1) Year 2012 dollars

7.3.4 New Forcemain from SCPS to WPCP

The split flow concept includes upgrades to the SCPS and force main. A detailed discussion of these recommended upgrades is included in Section 8 of this report.

7.3.5 Alternative 2A: Split Flow Sanitary Screen Facility and Pump Station

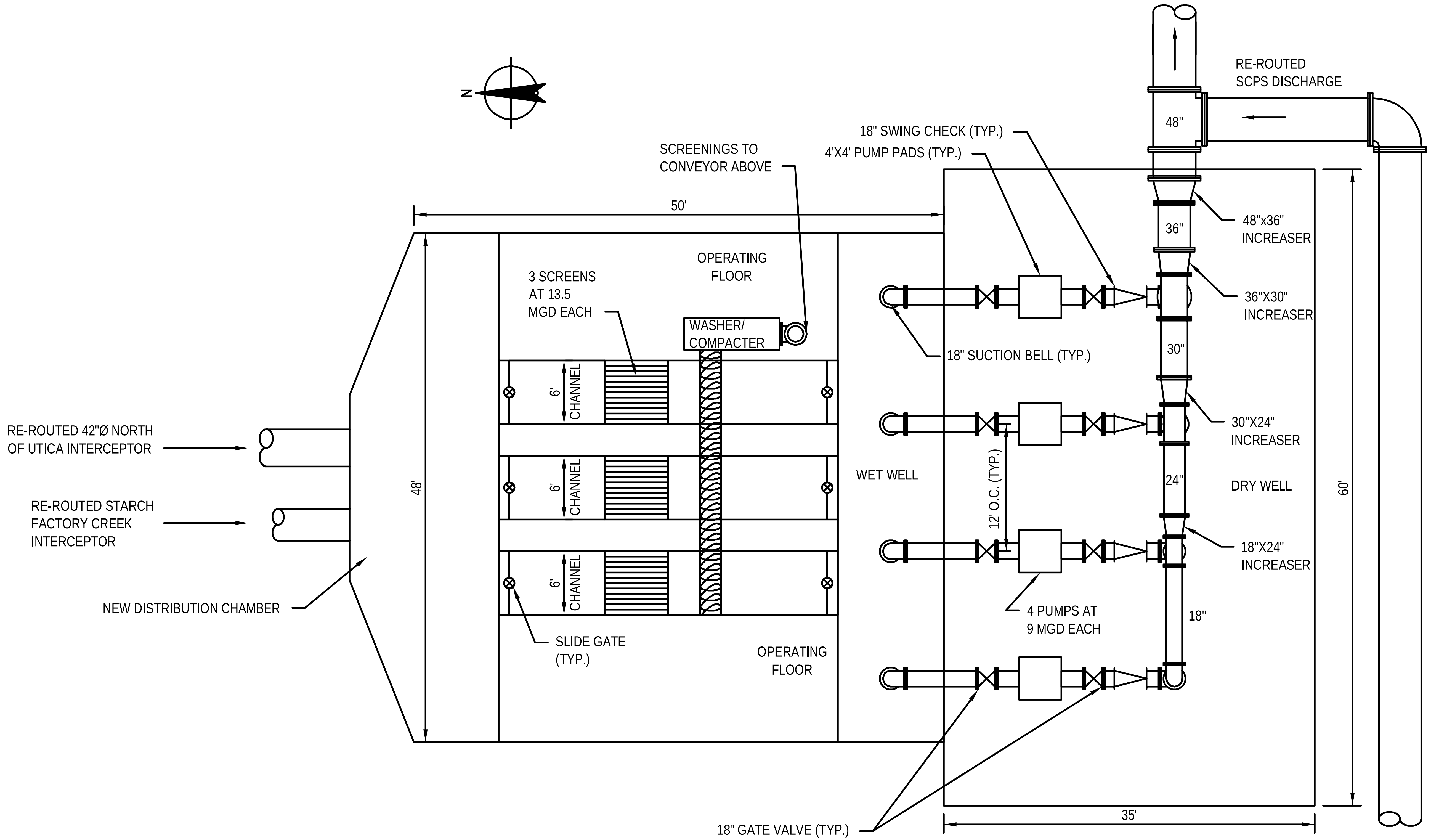
Similar to the existing screens and raw waste pump station at the WPCP, the proposed screening and pumping processes for the split flow concept are contained in one (1) structure.

For the new sanitary screen facility the first level consists of the operating floor, which contains the screenings collection equipment (washer/compacter) and electrical controls at ground level with three (3) subsurface levels. Below the operating floor, the first sublevel provides access to the screens, the second sublevel provides access to the slide gate operators and operations on the screen, while the third level is a wet well that contains the slide gates and submerged portions of the screens. The Screen Facility building dimensions are approximately 48 ft. wide, by 50 ft. long, by 50 ft. high.

For the sanitary pump station, the first sublevel contains the discharge header, the second sublevel contains the dry well for the sanitary pumps and access to pump suction and discharge valves, while the third sublevel contains the wet well for the screened influent. The Pump Station building dimensions are approximately 60 ft. wide, by 35 ft. long, by 50 ft. high. Figure 7-7 shows the layout of the sanitary screen facility and pump station.

The new sanitary screen facility and Pump Station would be located west of the existing Ash Lagoons. Piping modifications would be required to tie into the existing North of Utica Interceptor and Starch Factory Creek piping. New piping would need to be installed to route the piping from each source into the new Screen Facility and Pump Station. An overall site plan is shown on Figure 7-8.

Sanitary flows enter the screen facility through the re-routed North of Utica Interceptor and the re-routed Starch Factory Creek Interceptor. Approximately 19 mgd flows from the North of



0 5' 10' 15'

SCALE 1"=10' AT ORIGINAL SIZE

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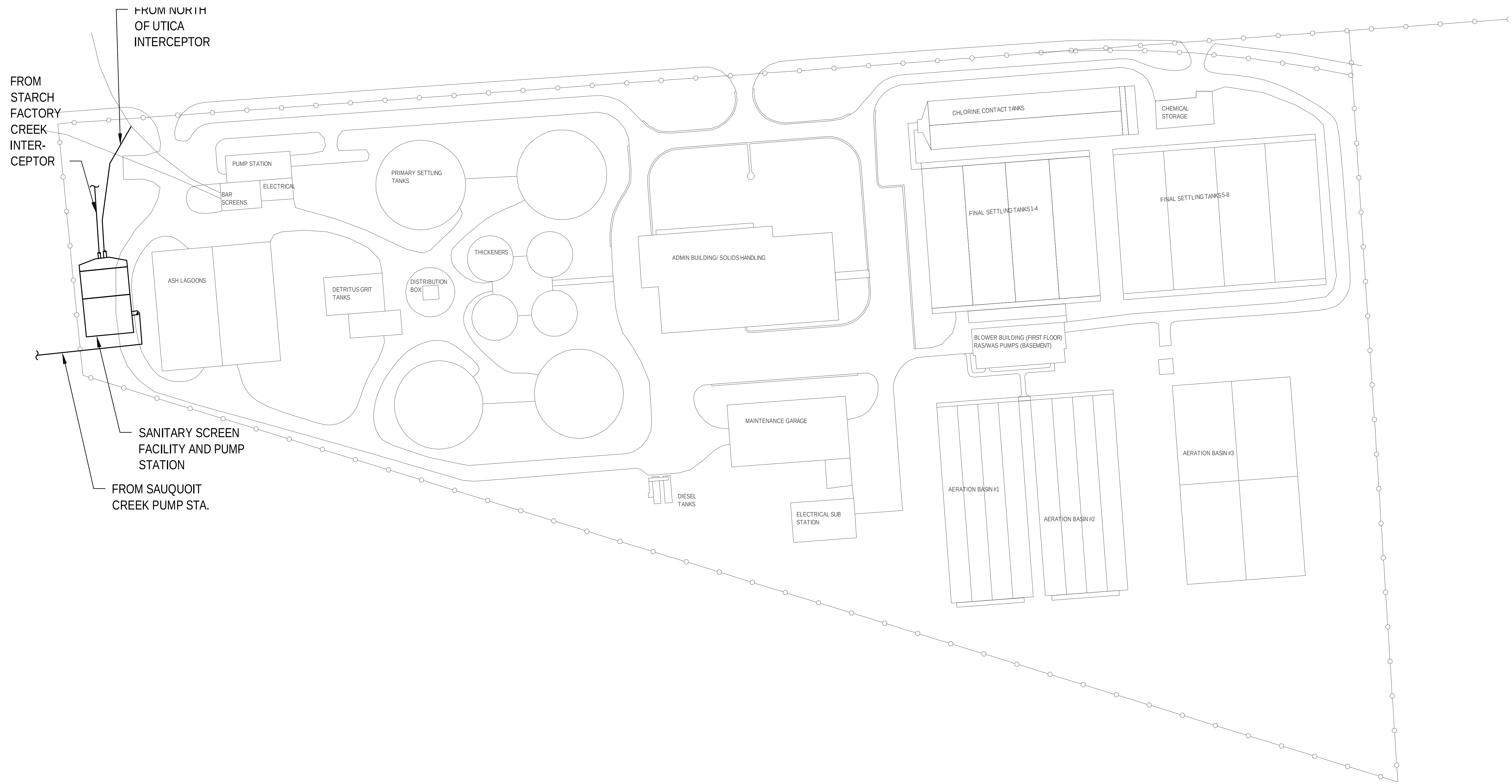
WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY
SPLIT FLOW CONCEPT
SANITARY PUMP STATION
AND SCREEN FACILITY LAYOUT

Job Number	8614782
Revision	A
Date	JUNE, 2012

Figure 7-7

200 John James Audubon Parkway Suite 101, Amherst NY 14228 USA **T** 1 716 691 8503 **F** 1 716 691 8506 **E** amhmail@ghd.com **W** www.ghd.com

Plot Date: 22 August 2012 - 1:51 PM Cad File No: G:\86\14782\CADD\Drawings\Figures\pump building.dwg



WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY
SPLIT FLOW CONCEPT

SITE PLAN - SANITARY SCREEN FACILITY AND PUMP STATION

Job Number	8614782
Revision	A
Date	06/12

Figure 7-8

200 John James Audubon Parkway Suite 101, Amherst NY 14228 USA T 1 716 691 8503 F 1 716 691 8506 E amhmail@ghd.com W www.ghd.com

Plot Date: 22 August 2012 - 12:30 PM Cad File No: G:\86\14782\CADD\Drawings\Figures\SITE PLAN.dwg

Utica Interceptor and 8 mgd flows from the Starch Factory Creek Interceptor. Flows from the SCPS discharge to the sanitary pump station discharge forcemain outside of the screen facility and pump station structure. It is assumed that the SCPS flows are screened prior to pumping and that the pumps can provide sufficient pressure to combine with the flow from the new sanitary pump station.

Flows to the screen facility enter a trapezoidal-shaped distribution chamber for optimum flow distribution and then separate into three (3) channels to feed each bar screen. Ttwo (2) operating and one (1) standby mechanically-cleaned bar screens, each with a capacity of 13.5 mgd, are proposed. The screens have a clear opening between bars of 1/2-inch, are upward cleaning, and discharge out of the back of the unit. Each bar screen extends from the operating floor level to the influent channel invert. Each screen channel is 6 ft. wide with slide gates up and downstream of the bar screens for isolation. Screenings fall onto a screw conveyor that transports the material to a washer/compacter.

Discharge flow from the screen channels recombines in a single wet well that feeds four (4) (three (3) duty and one (1) standby) vertical, centrifugal non-clog pumps. Each pump is rated for 9 mgd and 60 ft. TDH and is located in the dry well. The pumps would operate on VFDs to vary the pumping rate in response to liquid levels in the wet well. Gate valves are provided on the pump suction and discharge piping for isolation. Swing check valves are also provided on the discharge of each pump in order to protect the pump from backflow and water hammer. Within the Pump Station building, the pump discharge branch piping combine into a discharge header that contains a magnetic flowmeter downstream of all branch piping. Outside of the Pump Station building the SCPS flow tees into the new Pump Station discharge header (containing North of Utica and Starch Factory Creek flows) and flows onto the grit system.

The Engineer's opinion of probable cost for the sanitary screen facility and pump station is shown in Tables 7-5 and 7-6, respectively. The estimates are separated by the Screen Facility and Pump Station.

7.3.6 Alternative 2B: Split Flow Grit System

A new vortex grit removal system is proposed for the split flow concept. To provide additional operating flexibility and for compliance with the Ten-States Standards, two (2) grit systems are proposed for the sanitary pump station discharge (includes North of Utica, Starch Factory Creek, and SCPS flows) and two (2) grit systems are proposed for the combined flows from the existing raw waste pump station. Two (2) 31 mgd grit systems and two (2) 25 mgd grit systems are proposed to treat the Sanitary Pump Station and Raw Waste Pump Station flows, respectively. Providing two (2) grit systems, rather than one (1) for each split flow, allows for some grit removal, within each split flow, even if one (1) tank is out of service.

Other grit removal technologies were investigated, but the vortex-type system was reviewed with plant operators and NYSDEC as the most favorable alternative. Vortex-type grit systems are proprietary systems in which the entrance and exit configurations are critical to the efficient operation of the units. The flow enters and exits tangentially at essentially the same elevation, follows a vortex flow pattern and then exits through the top of the tank. A rotating turbine maintains constant flow velocity and its adjustable pitch blades separate organics from the grit particles, keeping organics in suspension. Grit settles by gravity into a hopper, at the bottom of the tank, and the solids are removed from the hopper by a grit pump. The grit is then discharged into a grit concentrator, along a screw conveyor, and into a container for disposal.

The main structural components of the vortex grit system include concrete influent/effluent channels and a circular concrete grit chamber and hopper. The dimensions of the proposed grit system are shown in Table 7-7.

TABLE 7-7

GRIT SYSTEM DIMENSIONS

GRIT SYSTEM CAPACITY	GRIT SYSTEM DIMENSIONS (1)				
	INFLUENT/EFFLUENT CHANNEL			GRIT TANK	
	WIDTH (FT.)	LENGTH (FT.)	HEIGHT (FT.)	TANK DIAMETER (FT.)	TANK HEIGHT (FT.) (2)
25 mgd	4.0	10 (3)	2.5	16	14
31 mgd	4.5	12 (3)	3	18	16

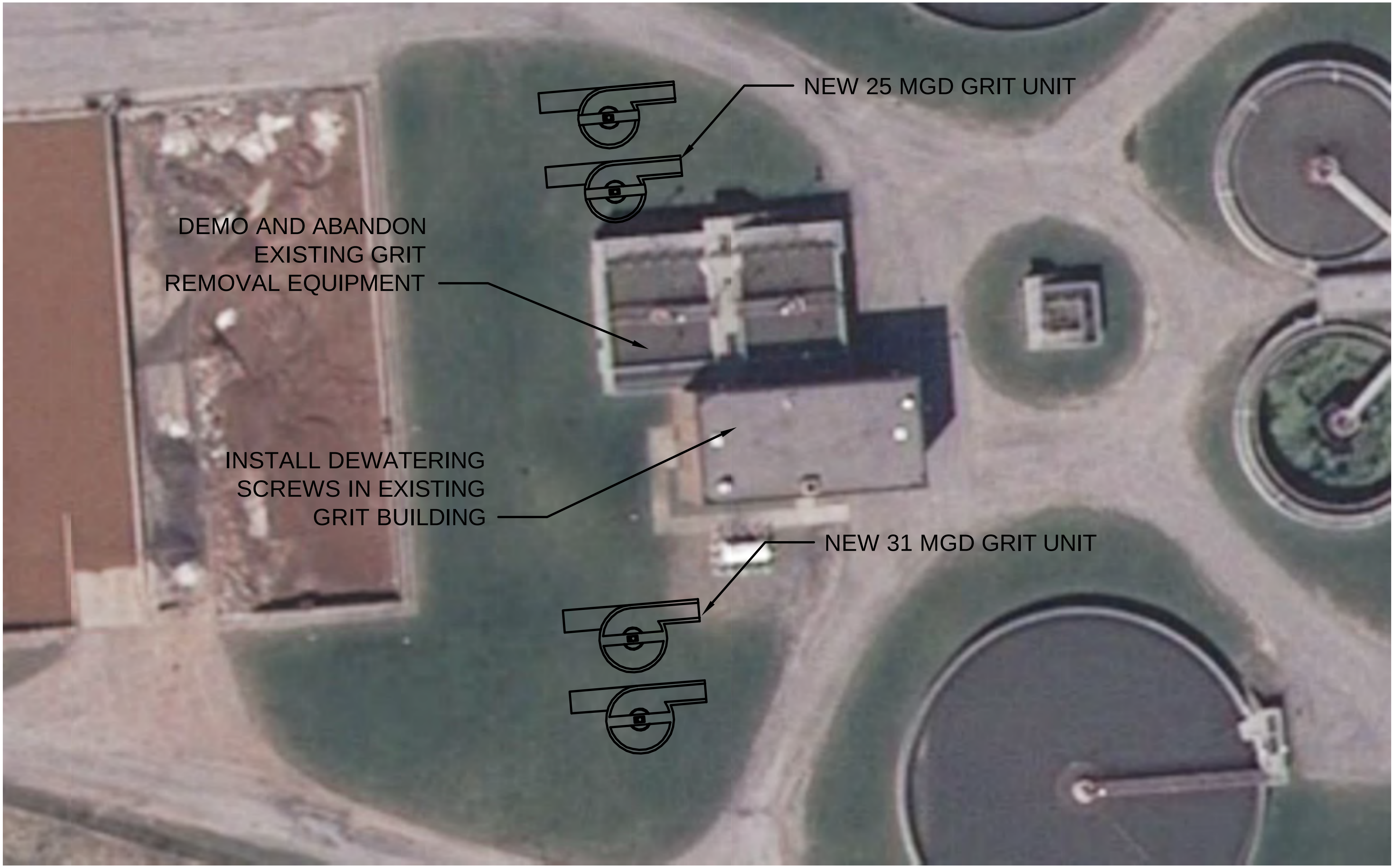
(1) Based on Smith and Loveless Pista ® grit system

(2) Includes grit hopper

(3) Dimension shown for influent channel. Effluent channel dimension varies

Figure 7-9 shows a plan view of the proposed grit system. The new grit tanks to treat the 62 mgd sanitary flows would be located south of the existing detritus grit tanks. New piping would be installed to connect the new sanitary pump station to the grit tanks and the new primary settling tanks. The new grit tanks treating the 49 mgd flow from the raw waste pump station would be located north of the existing detritus grit tanks. Piping modifications and new piping would be required to connect the new grit tanks to the raw waste pump station and the new primary settling tanks. Once the new grit tanks are installed and operational, the existing detritus grit tanks can be taken out of service and demolished. An overall site plan is shown on Figure 7-10.

The system can be designed for wastewater flow streams and grit removal to be at the existing hydraulic grade lines and locations reducing the risk of impacts to other systems. Since flows are equally split at the existing raw waste and new sanitary pump stations, a distribution box is no longer necessary and the head gained by removing the distribution box will allow the grit system



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ONEIDA COUNTY WPCP
WPCP AND SCPS EVALUATION

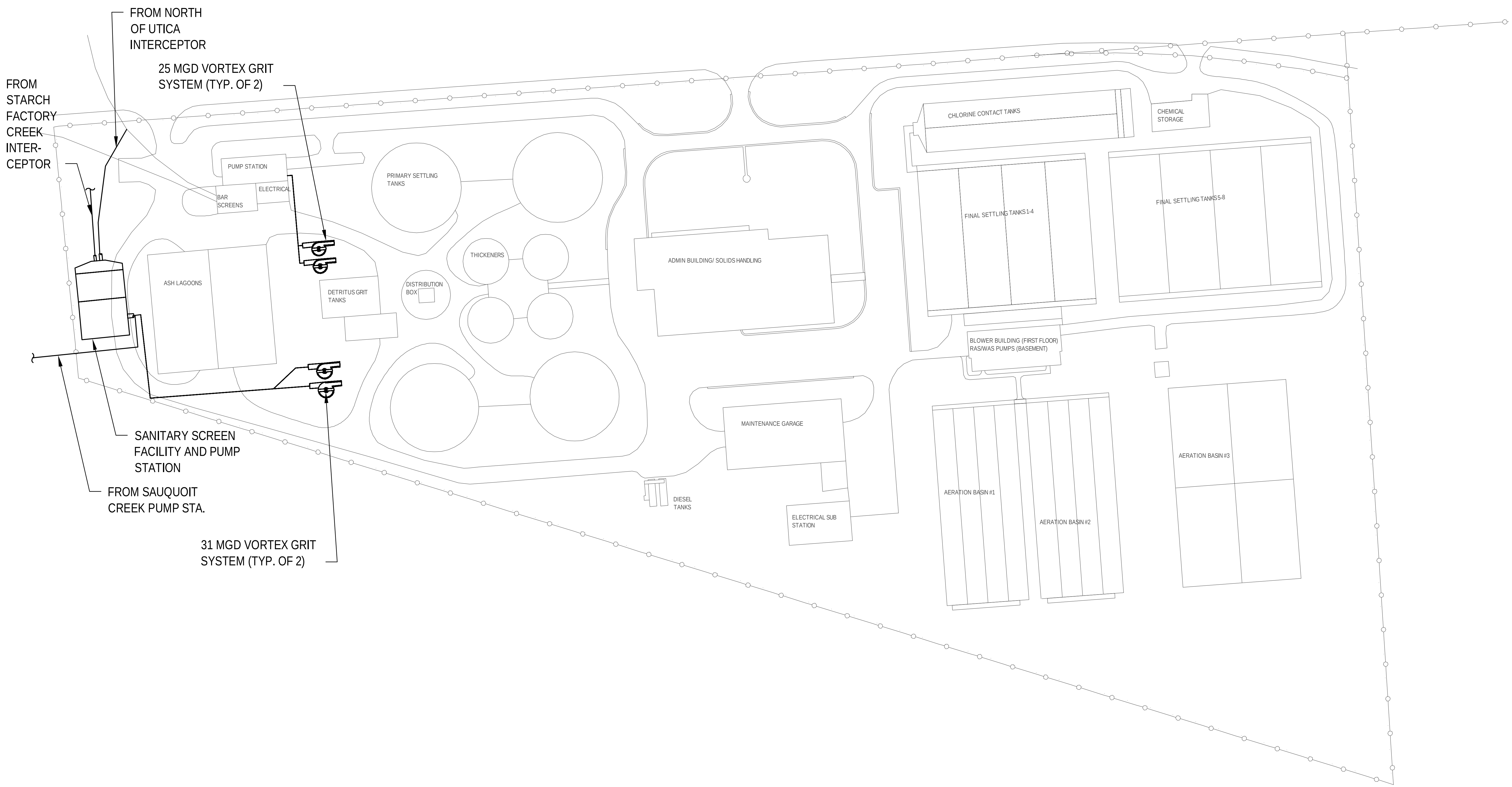
SPLIT FLOW CONCEPT GRIT
SYSTEM - SITE PLAN

Job Number	8614130
Revision	A
Date	05/12

Figure 7-9

200 John James Audubon Parkway Suite 101, Amherst NY 14228 USA T 1 716 691 8503 F 1 716 691 8506 E amhmail@ghd.com W www.ghd.com

Plot Date: 22 August 2012 - 1:54 PM Cad File No: G:\86\14782\CADD\Plans and Elevations for Cost Estimate\111 MGD Grit System.dwg



0 60' 120' 180'

SCALE 1"=120' AT ORIGINAL SIZE

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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY
SPLIT FLOW CONCEPT
SITE PLAN - GRIT SYSTEM

Job Number	8614782
Revision	A
Date	06/12

Figure 7-10

200 John James Audubon Parkway Suite 101, Amherst NY 14228 USA T 1 716 691 8503 F 1 716 691 8506 E amhmail@ghd.com W www.ghd.com

Plot Date: 22 August 2012 - 12:31 PM Cad File No: G:\86\14782\CADD\Drawings\Figures\SITE PLAN.dwg

and downstream settling tanks to be raised and provide greater head for the aeration basins, final settling tanks, and chlorine contact tanks.

The Engineer's opinion of probable cost for the new grit system is shown in Table 7-8.

7.3.7 Alternative 2C: Split Flow Primary Settling Tanks

The primary objective of primary settling is to remove readily settleable (suspended) solids and BOD before flow proceeds to the secondary treatment system. Sludge that settles in the primary settling tanks is collected in a hopper and then pumped to gravity thickeners for further processing. Effluent from the primary settling tanks would flow by gravity to the secondary treatment system. Three (3) alternatives were investigated for the new primary settling tanks for the split flow concept, including:

1. Conventional Settling Tanks (described in Section 7.3.7.1)
2. Chemically Enhanced Primary Treatment (described in Section 7.3.7.2), and
3. Ballasted Flocculation (described in Section 7.3.7.3).

7.3.7.1 Alternative 2C-1: Conventional Primary Settling Tanks

Rectangular primary settling tanks with chain-and-flight sludge collectors are proposed for the split flow concept. The solids settling in the tank are scraped to transverse troughs, equipped with cross collectors at the end of each tank, with chain-and-flight collectors that convey solids to collection hoppers. Solids pumping facilities are located close to the collection hoppers at the end of each tank.

Two (2) separate concrete tanks are proposed for the Split Flow Concept. One (1) primary settling tank would treat the 62 mgd sanitary flows and one (1) tank would treat the 49 mgd raw waste pump station flows. The settling tank treating the 62 mgd sanitary flows would be broken into two (2) basins (with a common wall) and further divided into four (4) passes per basin, for a total of eight (8) passes. The settling tank treating the 49 mgd raw waste pump station discharge would be broken into two (2) basins (with a common wall) and further divided into three (3) passes per basin, for a total of six (6) passes. The dimensions of each basin are based on maintaining a peak surface overflow rate of 2,000 gpd/ft^2 and a 4:1 length to width ratio. The number of passes per basin was determined by maintaining the chain-and-flight mechanism width less than 20 ft. Table 7-9 shows the proposed dimensions of each basin. A 10 ft. long distribution box and 10 ft. effluent channel would run the width of each basin and transverse sludge collection troughs would also span the entire width of each tank. Figure 7-11 shows a plan view of the primary settling tanks.

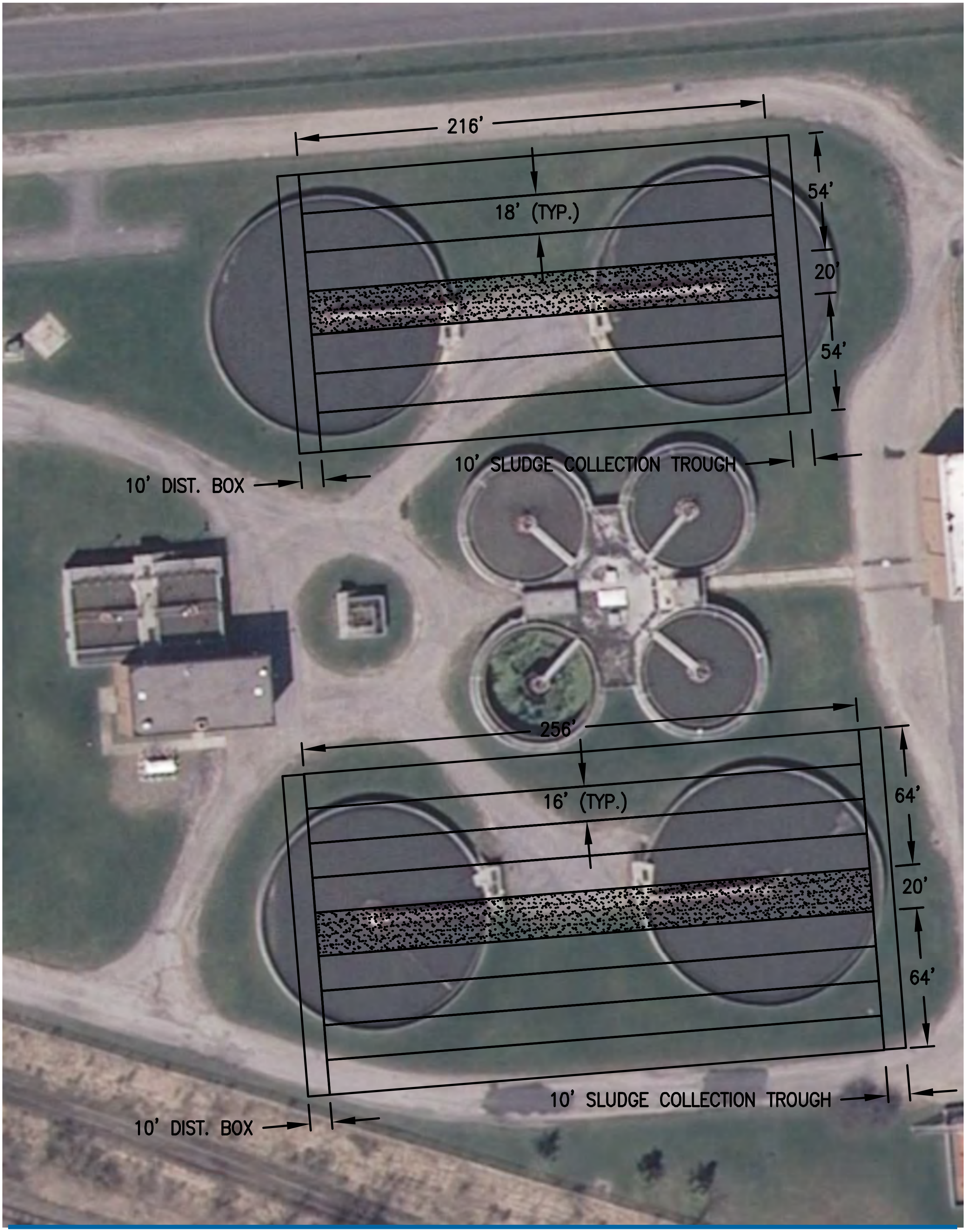
TABLE 7-9

CONVENTIONAL PRIMARY SETTLING TANK DIMENSIONS

	Conventional Primary Settling Tank Dimensions					
	Overall Basin Dimensions		Total No. of Passes/Basin	Pass Dimensions		SWD (ft.)
Flow	Width (ft.)	Length (ft.) (1)		Width (ft.)	Length (ft.)	
49 mgd	108	236	6	18	216	12
62 mgd	128	276	8	16	256	

(1) Includes 10-foot influent distribution chamber and ten-foot effluent distribution chamber

The new settling Tanks to treat the 62 mgd sanitary flows would be situated in the location of the existing southern primary clarifiers (south of the existing sludge thickeners). The new settling tanks treating the 49 mgd flow from the raw waste pump station would be situated in the location of the existing northern primary clarifiers (north of the existing sludge thickeners). Piping modifications and new piping would be required to connect the new primary settling tanks to the new grit tanks and the aeration basins. Construction would have to be staged in order to keep part of the WPCP in operation while installing the new settling tanks, since the new tanks would be installed in place of the existing clarifiers. An overall site plan is shown on Figure 7-12.



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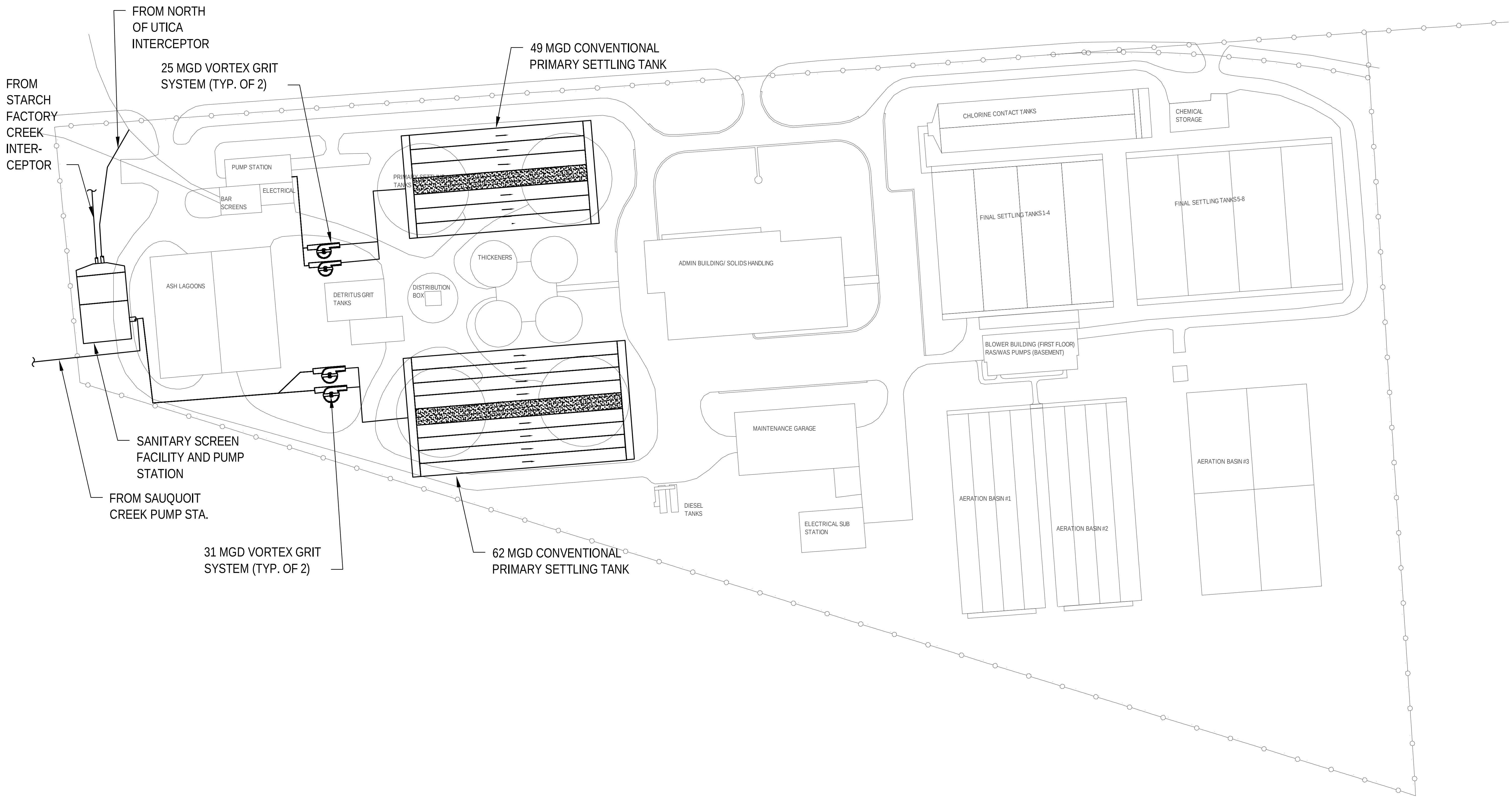
ONEIDA COUNTY SEWER DISTRICT
WPCP/SCPS EVALUATION
SPLIT FLOW CONCEPT
PRIMARY SETTLING TANKS

Job Number	86-14782
Revision	A
Date	02/12

Figure 7-1



0 60' 120' 180'
SCALE 1"=120' AT ORIGINAL SIZE

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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY
SPLIT FLOW CONCEPT
SITE PLAN - CONVENTIONAL PRIMARY SETTLING TANKS

Job Number	8614782
Revision	A
Date	06/12

Figure 7-12

200 John James Audubon Parkway Suite 101, Amherst NY 14228 USA T 1 716 691 8503 F 1 716 691 8506 E amhmail@ghd.com W www.ghd.com

Plot Date: 22 August 2012 - 12:34 PM Cad File No: G:\86\14782\CADD\Drawings\Figures\SITE PLAN.dwg

TABLE 7-10

ENGINEER'S OPINION OF PROBABLE COST: CONVENTIONAL PRIMARY SETTLING TANKS

DESCRIPTION	MATERIAL				INSTALLATION (1)		TOTAL COST (2)
	BASIS	NO. UNITS	PER UNIT	SUBTOTAL	PER UNIT	SUBTOTAL	
Demo Existing Clarifiers	EA	4	$375,000	$1,500,000	--	--	$1,500,000
Site Clearing	SF	71,300	$2	$140,000	--	--	$140,000
Excavation	CY	22,130	$30	$660,000	--	--	$660,000
Bedding	CY	2,420	$30	$70,000	--	--	$70,000
Foundation Piles	LS	1	$1,000,000	$1,000,000	--	--	$1,000,000
Sheeting/Bracing	SF	13,600	$25	$340,000	--	--	$340,000
Backfill	CY	2,800	$20	$60,000	--	--	$60,000
Hauling	CY	19,320	$20	$390,000	--	--	$390,000
Concrete Walls	CY	2,560	$800	$2,050,000	--	--	$2,050,000
Base Slab	CY	4,850	$700	$3,400,000	--	--	$3,400,000
Clarification Equipment - Chain and Flights	EA	14	$100,000	$1,400,000	$30,000	$420,000	$1,820,000
New Primary Sludge Pumps (1 WAS Pumps per Tank)	EA	4	$70,000	$280,000	$21,000	$84,000	$360,000
Sludge Piping	LF	1,000	$200	$200,000	--	--	$200,000
Site Dewatering & Erosion Control	LS	1	$410,000	$410,000	--	--	$410,000
Painting	LS	1	$180,000	$180,000	--	--	$180,000
Miscellaneous Metals (Grating, Handrail etc.)	LS	1	$220,000	$220,000	--	--	$220,000
Site Restoration	SF	71,300	$2	$140,000	--	--	$140,000
Subtotal							**$12,900,000**
Electrical, Controls and Instrumentation (15% of Subtotal)							**$1,900,000**
General Conditions, Bonds & Insurance (5% of Subtotal)							**$650,000**
Contingency (20%)							**$3,100,000**
Total Probable Construction Cost							***$18,600,000***
Engineering, Administrative, and Legal (20%)							**$3,700,000**
Total Probable Project Cost (Rounded)							***$22,300,000***

(1) For items without installation cost, installation is included in material cost.
(2) Year 2012 Dollars.

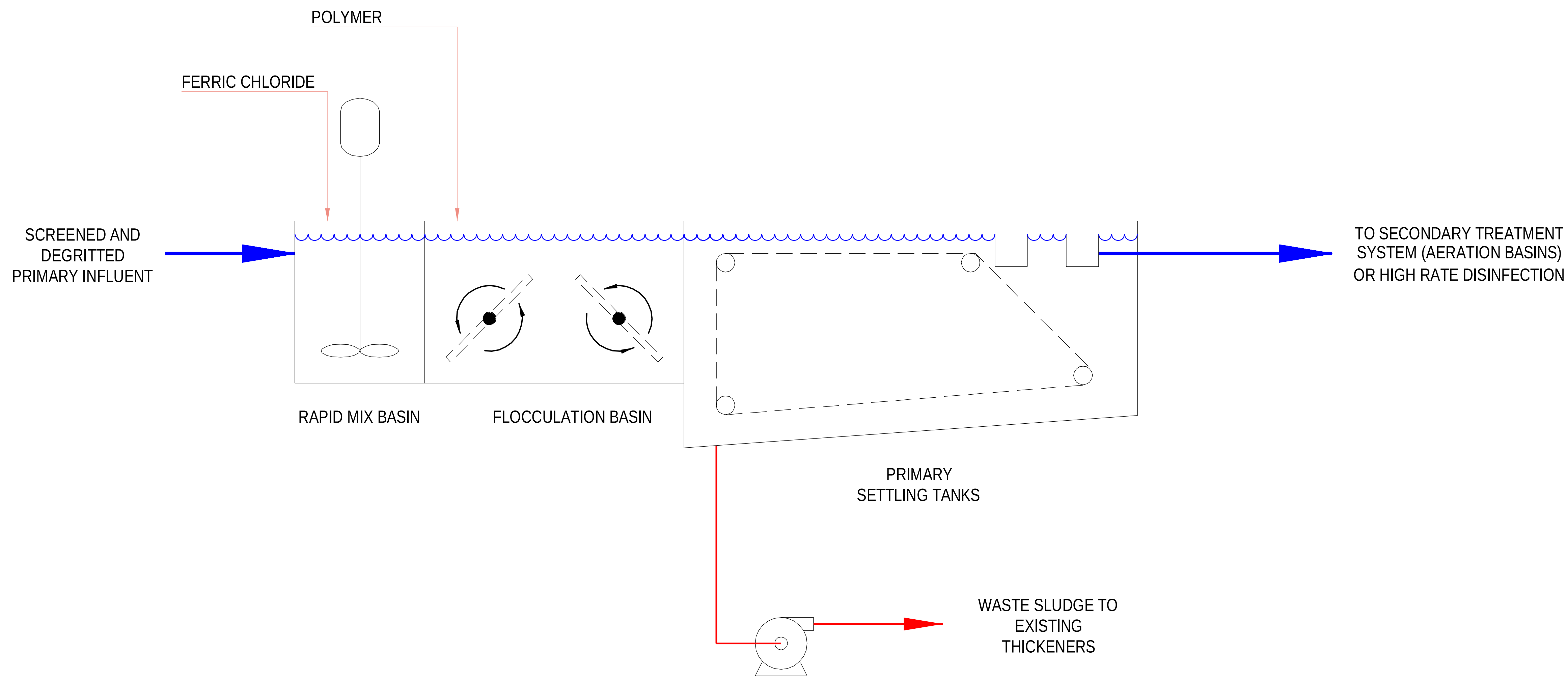
The Engineer's opinion of probable cost for the new conventional primary settling tanks is shown in Table 7-10.

7.3.7.2 Alternative 2C-2: Chemically Enhanced Primary Treatment (CEPT)

Physical-chemical treatment of wastewater was heavily relied upon before the development and widespread adoption of biological treatment. Recently however, a variation of the early process that relies on considerably lower dosages of chemicals, has found applicability in large-scale wastewater treatment processes in which high seasonal hydraulic loading variations are experienced, where there is limited space availability, and where the characteristics of the receiving stream require treatment levels higher than primary, but not as stringent as secondary treatment.

CEPT would be used in place of the primary settling tanks. CEPT utilizes the addition of metal salts and coagulant chemicals (polymer) to the influent upstream of the settling tanks to increase flocculation and settling of solids. Optimized polymer addition and mixing is provided to maximize flocculation of wastewater solids. The benefit of CEPT is that efficient removals of suspended solids can be achieved at high surface overflow rates associated with the peak wet weather flows. New CEPT facilities include chemical mixing and flocculation tankage, rectangular primary settling tanks and chemical handling facilities. Figure 7-13 shows a process flow schematic for the CEPT system.

For CEPT operation, a coagulant (such as ferric chloride) would be added to a rapid mix chamber. In this chamber, intense mixing ($G = 300\ sec^{-1}$) with a short hydraulic detention time (1 min.) disperses the coagulant throughout the primary influent. A typical ferric chloride dose in the rapid mix chamber of a CEPT facility is 30 mg/L, and a typical hydraulic detention time is approximately 1 minute.



LEGEND

- WASTEWATER FLOW
- SLUDGE FLOW
- CHEMICAL INJECTION

NOTE:
ONE TREATMENT TRAIN SHOWN; TYPICAL FOR ALL TRAINS

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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY
SPLIT FLOW CONCEPT
CEPT
FLOW DIAGRAM

Job Number	8614782
Revision	A
Date	JUNE 2012

Figure 7-13

200 John James Audubon Parkway Suite 101, Amherst NY 14228 USA **T** 1 716 691 8503 **F** 1 716 691 8506 **E** amhmail@ghd.com **W** www.ghd.com

Plot Date: 22 August 2012 - 1:04 PM Cad File No: G:\86\14782\CADD\Drawings\Figures\SCHEMATICS-ALL.dwg

Following rapid mix, flocculation basins would be installed to allow floc particles to grow. A low dose of polymer (< 0.5 mg/L) could be added to assist floc particles to bind together. A typical CEPT flocculation basin has a hydraulic detention time of 20 minutes, and less intense mixing (G = 50 sec^{-1}) than rapid mix. The longer detention time and gentler mixing will create more ideal conditions for floc formation. A surface overflow rate of 4,000 gpd/ft^2 (at maximum flow) and a 4:1 length to width ratio were used to size the rectangular clarifiers. The number of passes per basin was determined by maintaining the chain-and-flight mechanism width less than 20 ft. Ten ft. long transverse sludge collection troughs would span the entire width of each tank.

Two (2) separate CEPT systems would be installed for the Split Flow Concept. One (1) CEPT system would treat the 62 mgd sanitary flow and the other system would treat the 49 mgd raw waste pump station flow. Dimensions of the CEPT facilities were calculated based on the parameters described above and are shown in Table 7-11.

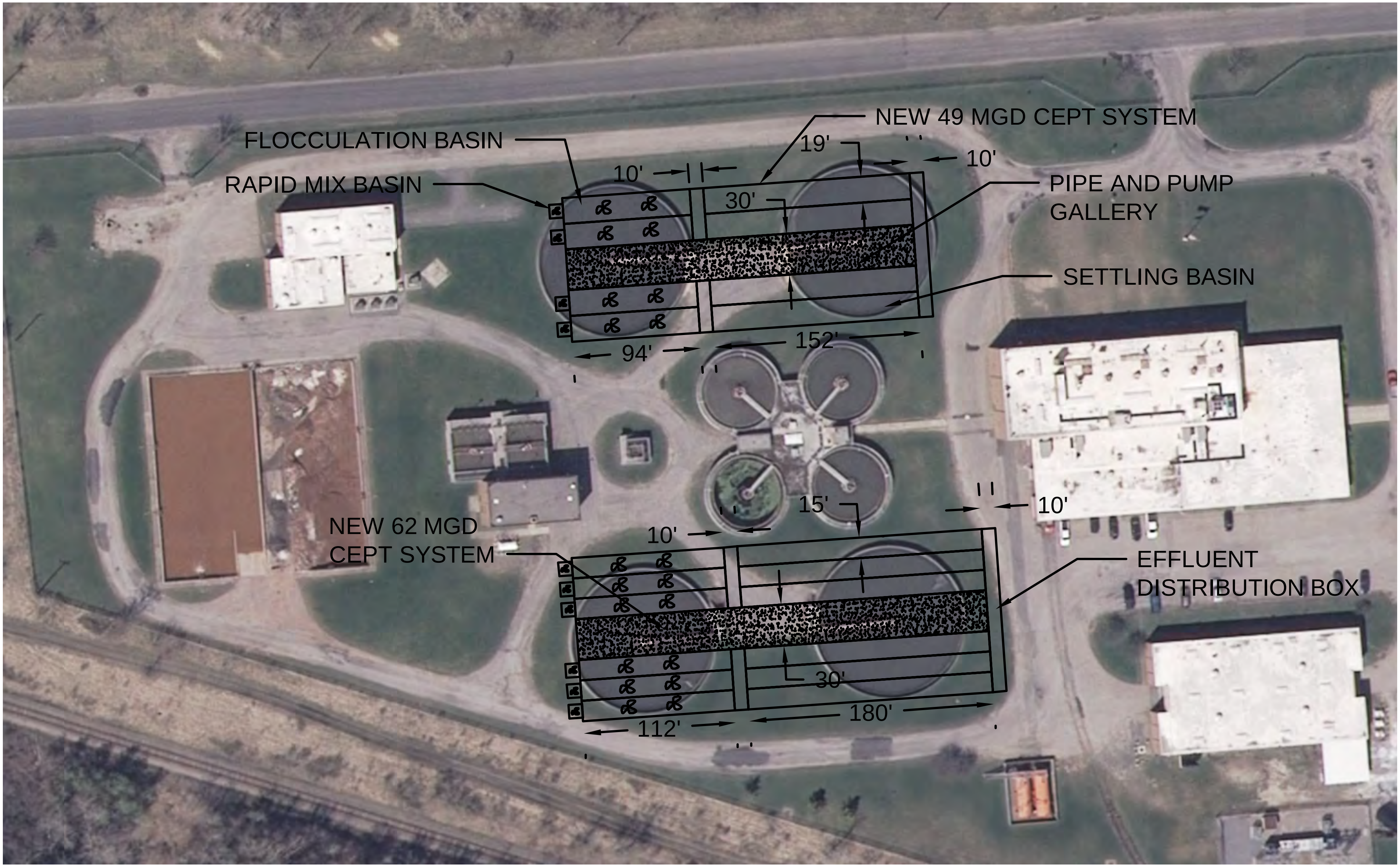
TABLE 7-11

CEPT TANK DIMENSIONS

	CEPT Tank Dimensions								
	Overall Basin Dim.		Rapid Mix Dim.		Flocculation Basin Dim.		Settling Basin Dim.		
Flow	Width (ft.) (1)	Length (ft.) (2)	Width (ft.)	Length (ft.)	Width (ft.) (3)	Length (ft.)	Width (ft.) (3)	Length (ft.)	SWD (ft.)
49 mgd	106	276	10	10	38	91	38	152	12
62 mgd	120	322	10	10	45	112	45	180	

(1) Includes a 30-foot pipe and pump gallery through center of basins
(2) Includes 10-foot transverse trough for sludge collection and 10-foot effluent distribution chamber
(3) Width per pass

The new CEPT system to treat the 62 mgd sanitary flows would be situated in the location of the existing southern primary clarifiers (south of the existing sludge thickeners). The new CEPT system treating the 46 mgd flow from the raw waste Pump Station would be situated in the location of the existing northern primary clarifiers (north of the existing sludge thickeners).



DRAWING NOT TO SCALE

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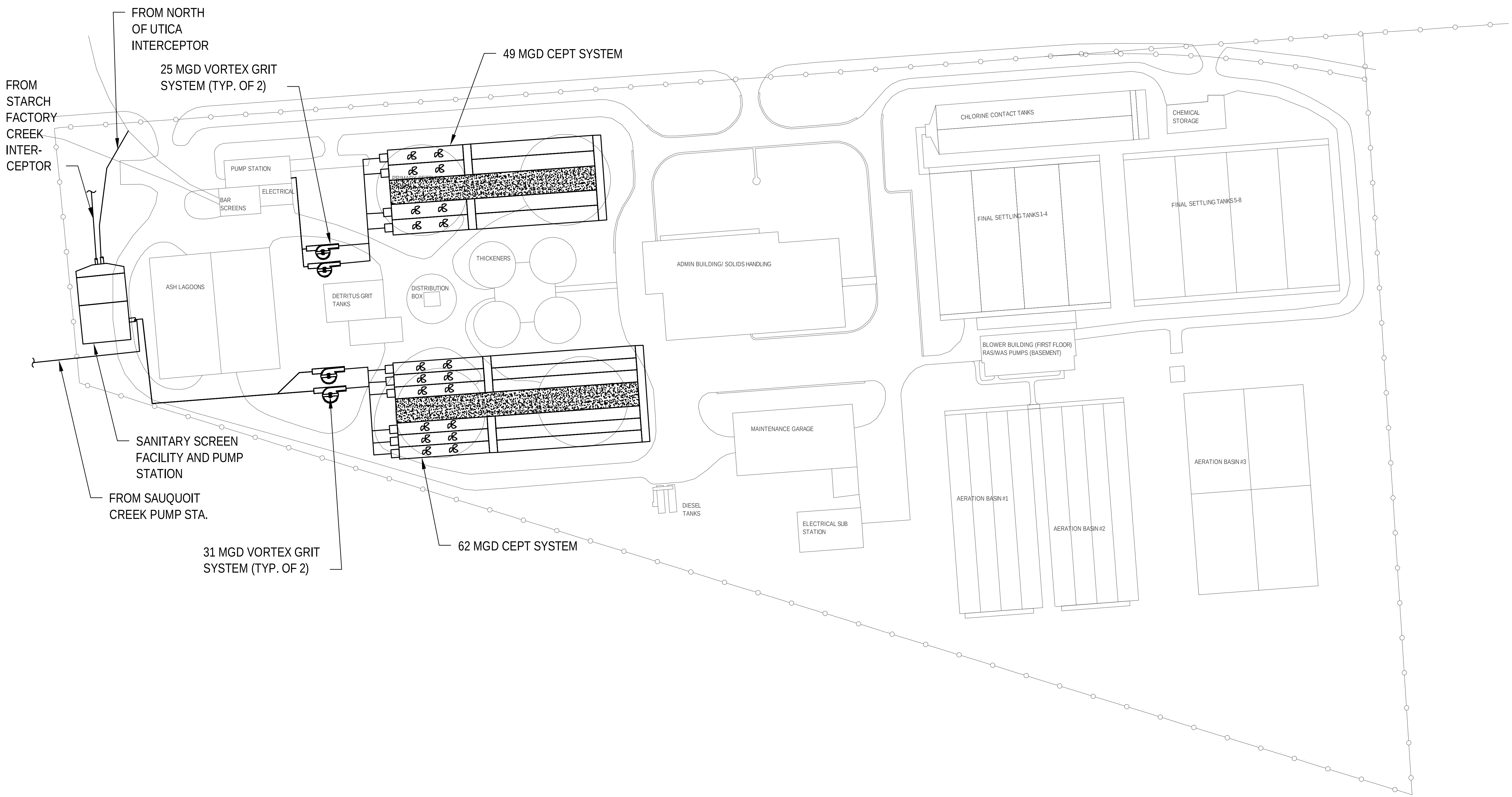
Oneida County WPCP
WPCP and SCPS Evaluation
SPLIT FLOW CONCEPT
CEPT SYSTEM LAYOUT

Job Number	8614130
Revision	A
Date	May 2012

Figure 7-14

200 John James Audubon Parkway Suite 101, Amherst NY 14228 USA **T** 1 716 691 8503 **F** 1 716 691 8506 **E** amhmail@ghd.com **W** www.ghd.com

Plot Date: 22 August 2012 - 1:56 PM Cad File No: G:\86\14782\CADD\Plans and Elevations for Cost Estimate\111 MGD CEPT System.dwg



0 60' 120' 180'

SCALE 1"=120' AT ORIGINAL SIZE

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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY
SPLIT FLOW CONCEPT
SITE PLAN - CEPT SYSTEM

Job Number	8614782
Revision	A
Date	06/12

Figure 7-15

200 John James Audubon Parkway Suite 101, Amherst NY 14228 USA T 1 716 691 8503 F 1 716 691 8506 E amhmail@ghd.com W www.ghd.com

Plot Date: 22 August 2012 - 12:35 PM Cad File No: G:\86\14782\CADD\Drawings\Figures\SITE PLAN.dwg

TABLE 7-12

ENGINEER'S OPINION OF PROBABLE COST: CEPT

DESCRIPTION	MATERIAL				INSTALLATION (1)		TOTAL COST (2)
	BASIS	NO. UNITS	PER UNIT	SUBTOTAL	PER UNIT	SUBTOTAL	
Demo Existing Clarifiers	EA	4	$375,000	$1,500,000	--	--	$1,500,000
Site Clearing	SF	76,400	$2	$150,000	--	--	$150,000
Excavation	CY	25,180	$30	$760,000	--	--	$760,000
Bedding	CY	2,540	$30	$80,000	--	--	$80,000
Foundation Piles	LS	1	$1,000,000	$1,000,000	--	--	$1,000,000
Sheeting/Bracing	SF	21,350	$25	$530,000	--	--	$530,000
Backfill	CY	3,790	$20	$80,000	--	--	$80,000
Hauling	CY	21,380	$20	$430,000	--	--	$430,000
Concrete Walls	CY	2,760	$800	$2,210,000	--	--	$2,210,000
Base Slab	CY	5,090	$700	$3,560,000	--	--	$3,560,000
Clarification Equipment	EA	10	$80,000	$800,000	$24,000	$240,000	$1,040,000
New Primary Sludge Pumps (1 WAS Pump per Tank)	EA	4	$70,000	$280,000	$21,000	$84,000	$360,000
Pump Gallery and Controls Building - 49 MGD CEPT	SF	800	$250	$200,000	--	--	$200,000
Pump Gallery and Controls Building - 62 MGD CEPT	SF	1,000	$250	$250,000	--	--	$250,000
Polymer Storage Tank	EA	2	$50,000	$100,000	$15,000	$30,000	$130,000
Coagulant Storage Tank	EA	2	$50,000	$100,000	$15,000	$30,000	$130,000
Coagulant Feed System	EA	2	$75,000	$150,000	$22,500	$45,000	$200,000
Polymer Feed System	EA	2	$100,000	$200,000	$30,000	$60,000	$260,000
Flocculators	EA	30	$50,000	$1,500,000	$15,000	$450,000	$1,950,000
Mixers	EA	10	$100,000	$1,000,000	$30,000	$300,000	$1,300,000
Sludge Piping	LF	1,000	$200	$200,000	--	--	$200,000
Site Dewatering & Erosion Control	LS	1	$500,000	$500,000	--	--	$500,000
Painting	LS	1	$450,000	$450,000	--	--	$450,000
Miscellaneous Metals (Grating, Handrail etc.)	LS	1	$240,000	$240,000	--	--	$240,000
Site Restoration	SF	76,400	$2	$150,000	--	--	$150,000
Subtotal							**$17,700,000**
Electrical, Controls and Instrumentation (15% of Subtotal)							**$2,700,000**
General Conditions, Bonds & Insurance (5% of Subtotal)							**$900,000**
Contingency (20%)							**$5,300,000**
Total Probable Construction Cost							***$26,600,000***
Engineering, Administrative, and Legal (20%)							**$5,300,000**
Total Probable Project Cost (Rounded)							***$32,000,000***

(1) For items without installation cost, installation is included in material cost.
(2) Year 2012 Dollars.

Piping modifications and new piping would be required to connect the new CEPT tanks to the new grit tanks and the aeration basins. Construction would have to be staged in order to keep part of the WPCP in operation while installing the new CEPT Tanks, since the new tanks would be installed in the place of the existing clarifiers. Figure 7-14 shows a plan view of the CEPT Tanks and an overall site plan is shown on Figure 7-15.

The Engineer's opinion of probable cost for the new CEPT system is shown in Table 7-12.

7.3.7.3 Alternative 2C-3: Ballasted Flocculation

Ballasted flocculation, or high rate treatment (HRT), employs physical/chemical treatment and utilizes special flocculation and sedimentation systems to achieve rapid settling. The advantages of HRT include: units are compact resulting in smaller footprints, startup times are rapid to achieve peak efficiency, and highly clarified effluent is produced. The main components of HRT are enhanced particle settling and the use of inclined plate or tube settlers. HRT is a sedimentation process that utilizes particles of sand or thickened sludge which are recycled in the process along with chemical addition to aid in the flocculation and settling of solids to achieve a high removal rate of solids. The technology is projected to achieve an average solids removal rate of 85 percent. Implementing this technology as primary sedimentation for the split flow alternative would include construction of mixing, flocculation and settling tanks and the associated chemical and support facilities. While achieving high solids removal rates, this technology has high capital and operation costs and is more complex than other solids removal technologies.

The two (2) HRT technologies investigated are the Actiflo® process which uses microsand as the ballast and the Densadeg® process which uses thickened sludge as the ballast. The HRT technologies can treat flows at 30,000 gpd/sf of clarifier surface area or higher overflow rates which are significantly greater than for conventional primary clarifiers. The key benefits of HRT over primary clarifiers and CEPT is the smaller footprint required to treat an equivalent flow and the higher solids removal. A process description of the two technologies is presented below. Although they have operational differences, each of the technologies was considered viable HRT alternatives which would produce a similar effluent.

The Actiflo® process is a high performance and compact clarification system using microsand enhanced flocculation and settling. The Actiflo® process consists of three (3) compartments or zones: a mixing zone, maturation zone, and settling zone. A coagulant (ferric chloride) is added to the wastewater in a separate coagulation tank to destabilize the solids. The coagulated wastewater then enters a second tank, called the injection tank, where microsand (80 to 120 micron) and polymer are added. The microsand provides a large contact area and acts as a ballast, thereby accelerating the settling of floc. The destabilized suspended solids bind to the microsand particles by polymer bridges. In the third tank, the maturation tank, the particles agglomerate and grow into high density flocs known as microsand ballasted flocs, which settle quickly to the bottom of the fourth tank (the settling tank). The efficiency of settling is further increased by the use of lamella tubes in the settling tank.

The solids/microsand mixture collected at the bottom of the Actiflo® settling tank is pumped to hydrocyclones where the sludge is separated from the microsand by the centrifugal force of the vortex action. The recovered clean microsand is then recycled to the injection tank whereas the separated solids would be continuously discharged to the existing thickeners for sludge processing. The Actiflo® system produces a relatively thin sludge, typically less than 0.5% solids.

A process flow schematic for the Densadeg® and Actiflo® systems is provided on Figure 7-16.

Two (2) separate Actiflo® systems would be installed for the split flow concept. One (1) system would treat the 62 mgd sanitary flows and the other system would treat the 49 mgd raw waste pump station flows. Dimensions of the Actiflo® systems were provided by the manufacturer and are shown in Table 7-13. In addition to the coagulation, maturation, and settling tanks, a 50 ft. x

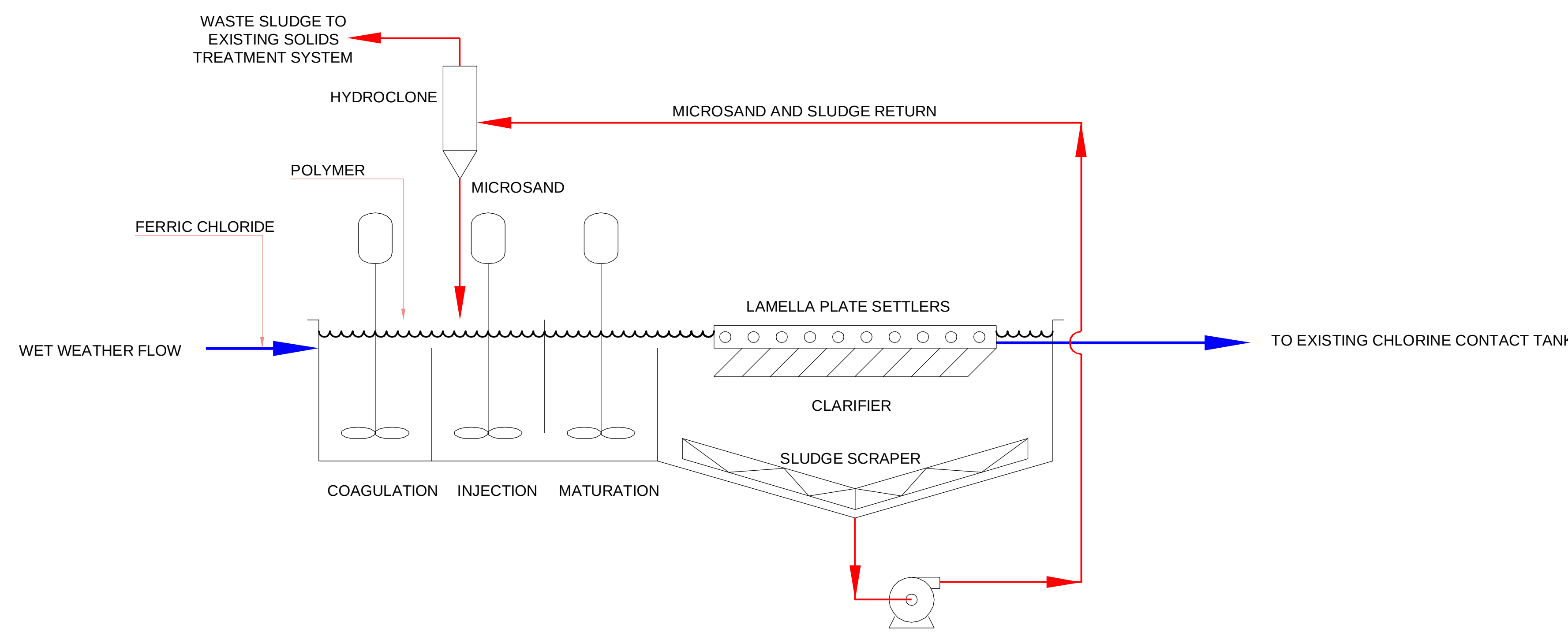
50 ft. building would be constructed to house the equipment (sand/sludge pumps, polymer feed system, coagulant feed system, and associated sand/chemical storage) required for the Actiflo® systems.

TABLE 7-13

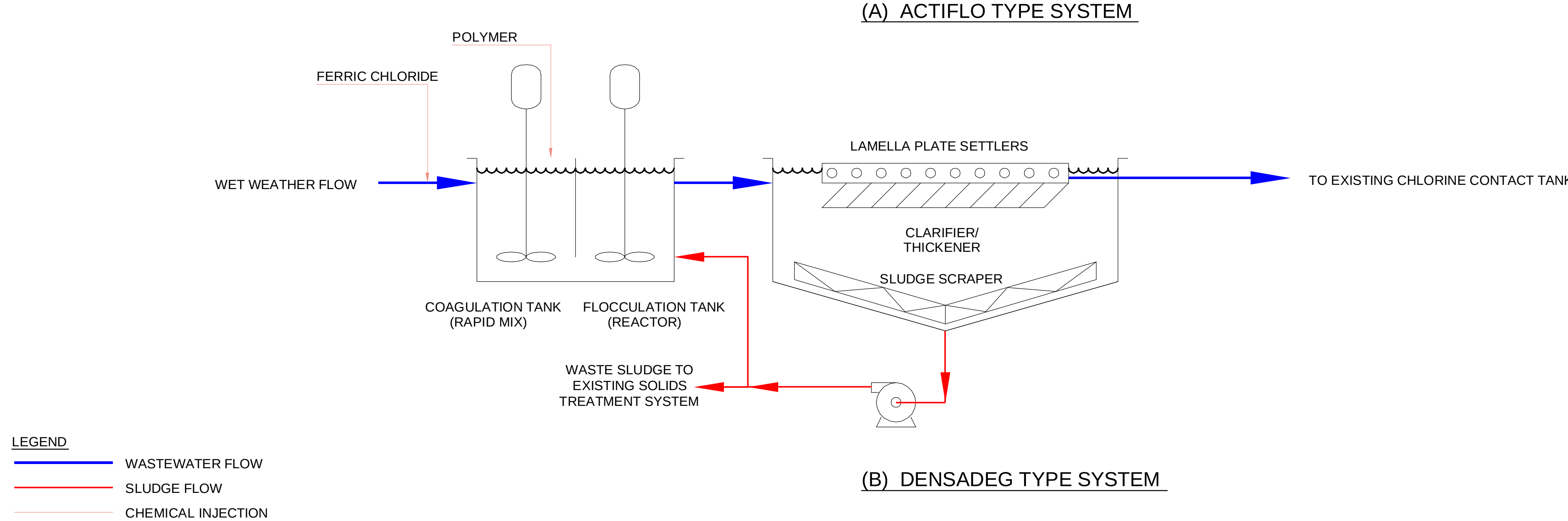
ACTIFLO® SYSTEM DIMENSIONS

SIZE	ACTIFLO® DIMENSIONS								
	OVERALL SYSTEM DIM.		COAGULATION BASIN DIM.		MATURATION BASIN DIM.		SETTLING BASIN DIM.		
	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	SWD (FT.)
49 mgd	29	72	18	18	29	24	29	29	27
62 mgd	32	80	22	22	32	24	32	32	28

The Densadeg® process incorporates three process zones: the reactor zone (rapid mix), the pre-settling/thickener zone (reactor), and the clarification zone. A process schematic is shown in Figure 7-16. In the reactor zone, influent wastewater is combined with reactants (ferric chloride and polymer) and preformed solids that have been recirculated from the pre-settling/thickener zone. As they flow upward through a draft tube, the wastewater reactants and thickener solids are mixed by a turbine, thus forming a flocculated mixture. Exiting the draft tube, the flocculated mixture moves downward. Near the bottom of the reactor, a significant amount of the mixture re-enters the draft tube, to be sheared and mixed with the initial products. Internal recirculation is carried out at a rate of up to 10 times the influent flow rate. Located near the bottom of the reactor vessel is a baffled opening that allows the mixture to exit the reactor. Moving upward between the baffle and the reactor shell, the slurry passes over a submerged weir into the pre-settling/thickener zone. Here, separation of the solids and supernatant occurs.



(A) ACTIFLO TYPE SYSTEM



(B) DENSADEG TYPE SYSTEM

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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

HIGH RATE TREATMENT
PROCESS FLOW SCHEMATIC

Job Number	8614782
Revision	A
Date	JUNE 2012

Figure 7-16

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The heavy (dense) sludge produced by the Densadeg® process settles to the bottom of the clarifier. Aided by a slow moving rake, settled solids are thickened to approximately 2% to 4%. Thickened solids could be pumped to the existing thickeners. The supernatant flows upward from the baffle opening that divides the thickener/clarifier. Lamella tubes, through which all of the supernatant must pass, provide for high rate removal of the remaining solids. A series of weir troughs, located above the tubes, collect the clarified effluent.

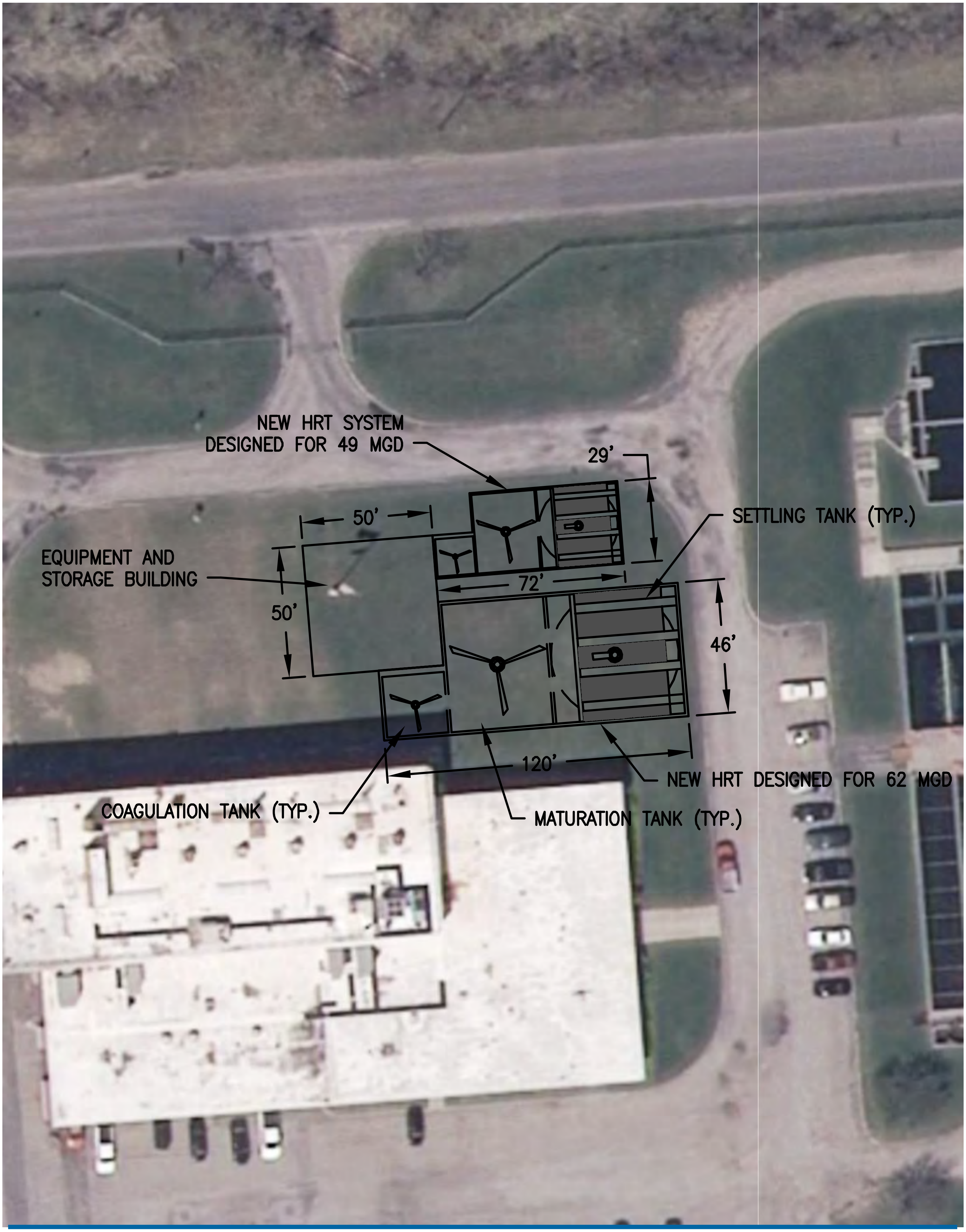
Two (2) separate Densadeg® systems would be installed for the split flow concept. One (1) system would treat the 62 mgd sanitary flows and the other system would treat the 49 mgd raw waste pump station flows. A pipe and pump gallery would be provided for each Densadeg® system. Dimensions of the Densadeg® systems were provided by the manufacturer and are shown in Table 7-14. Figure 7-17 shows a plan view of an HRT system (Actiflo®) and an overall site plan is shown on Figure 7-18. These site plans would be similar with the Densadeg® system.

TABLE 7-14

DENSADEG SYSTEM DIMENSIONS

	DENSADEG® DIMENSIONS						
	OVERALL SYSTEM DIM.		DIMENSIONS PER TRAIN		PIPE/PUMP GALLERY DIM.		
DENSADEG®	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGT H (FT.)	SIDEWATER DEPTH (FT.)
49 mgd	60	119	40	27	40	25	31.5
62 mgd	80	121	40	27	20	121	29.5

HRT facilities could replace the existing primary clarifiers or be used in conjunction with the existing clarifiers. Since the HRT systems have such small footprints, they could be built while all four existing primary clarifiers are in operation. The new HRT system to treat the 62 mgd sanitary flows and 49 mgd raw waste flows would be located north of the Administration/Solids Handling Building.



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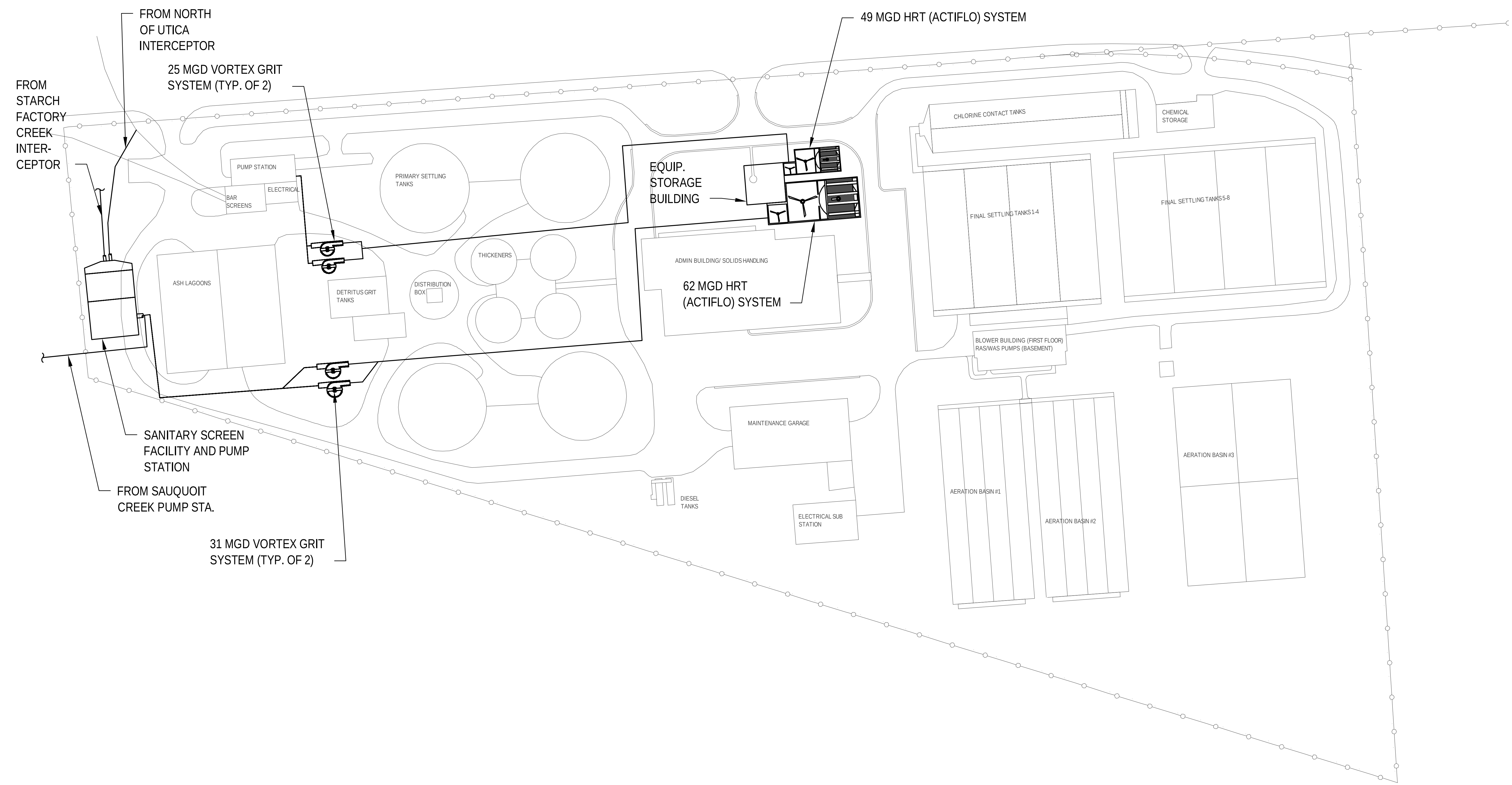
N

ONEIDA COUNTY SEWER DISTRICT
WPCP/SCPS EVALUATION
SPLIT FLOW CONCEPT
HIGH RATE TREATMENT SYSTEM
(ACTIFLO)

Job Number	86-14782
Revision	A
Date	02/12

Figure 7-17



0 60' 120' 180'

SCALE 1"=120' AT ORIGINAL SIZE

N

WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY
SPLIT FLOW CONCEPT

SITE PLAN - HRT (ACTIFLO) SYSTEM

Job Number	8614782
Revision	A
Date	06/12

Figure 7-18

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TABLE 7-15

ENGINEER'S OPINION OF PROBABLE COST: ACTIFLO® HRT

DESCRIPTION	MATERIAL				INSTALLATION (1)		TOTAL COST (2)
	BASIS	NO. UNITS	PER UNIT	SUBTOTAL	PER UNIT	SUBTOTAL	
Demo Existing Clarifiers	EA	4	$375,000	$1,500,000	--	--	$1,500,000
Site Clearing	SF	10,900	$2	$20,000	--	--	$20,000
Excavation	CY	7,680	$30	$230,000	--	--	$230,000
Bedding	CY	290	$30	$10,000	--	--	$10,000
Foundation Piles	LS	1	$250,000	$250,000	--	--	$250,000
Sheeting/Bracing	SF	13,000	$25	$330,000	--	--	$330,000
Backfill	CY	2,350	$20	$50,000	--	--	$50,000
Hauling	CY	5,330	$20	$110,000	--	--	$110,000
Concrete Walls for HRT System and Pump Gallery	CY	720	$800	$580,000	--	--	$580,000
Base Slab	CY	550	$700	$390,000	--	--	$390,000
49 MGD Actiflo HRT Equipment	EA	1	$2,300,000	$2,300,000	$690,000	$690,000	$2,990,000
62 MGD Actiflo HRT Equipment	EA	1	$2,800,000	$2,800,000	$840,000	$840,000	$3,640,000
Equipment and Storage Building	SF	2,500	$250	$630,000	--		$630,000
Davit Crane (for Sand Replacement)	LS	2	$10,000	$20,000	--	--	$20,000
Miscellaneous Piping	LF	1,000	$100	$100,000	--	--	$100,000
Site Dewatering & Erosion Control	LS	1	$160,000	$160,000	--	--	$160,000
Painting	LS	1	$220,000	$220,000	--	--	$220,000
Miscellaneous Metals (Grating, Handrail etc.)	LS	1	$190,000	$190,000	--	--	$190,000
Site Restoration	SF	10,900	$2	$20,000	--	--	$20,000
						Subtotal	**$11,400,000**
						Electrical, Controls and Instrumentation (15% of Subtotal)	**$1,700,000**
						General Conditions, Bonds & Insurance (5% of Subtotal)	**$570,000**
						Contingency (20%)	**$3,400,000**
						Total Probable Construction Cost	***$17,100,000***
						Engineering, Administrative, and Legal (20%)	**$3,400,000**
						Total Probable Project Cost (Rounded)	***$20,500,000***

(1) For items without installation cost, installation is included in material cost.

(2) Year 2012 Dollars.

TABLE 7-16

ENGINEER'S OPINION OF PROBABLE COST: DENSADEG® HRT

DESCRIPTION	MATERIAL				INSTALLATION (1)		TOTAL COST (2)
	BASIS	NO. UNITS	PER UNIT	SUBTOTAL	PER UNIT	SUBTOTAL	
Demo Existing Clarifiers	EA	4	$375,000	$1,500,000	--	--	$1,500,000
Site Clearing	SF	15,400	$2	$30,000	--	--	$30,000
Excavation	CY	14,150	$30	$420,000	--	--	$420,000
Bedding	CY	450	$30	$10,000	--	--	$10,000
Foundation Piles	LS	1	$250,000	$250,000	--	--	$250,000
Sheeting/Bracing	SF	18,400	$25	$460,000	--	--	$460,000
Backfill	CY	3,160	$20	$60,000	--	--	$60,000
Hauling	CY	10,990	$20	$220,000	--	--	$220,000
Concrete Walls for HRT System and Pump Gallery	CY	1,200	$800	$960,000	--	--	$960,000
Base Slab	CY	850	$700	$600,000	--	--	$600,000
49 MGD Densadeg HRT Equipment	EA	2	$780,000	$1,560,000	$234,000	$468,000	$2,028,000
62 MGD Densadeg HRT Equipment	EA	3	$760,000	$2,280,000	$228,000	$684,000	$2,964,000
49 MGD Densadeg HRT Pipe and Pump Gallery	SF	1,000	$250	$250,000	--	--	$250,000
62 MGD Densadeg HRT Pipe and Pump Gallery	SF	2,385	$250	$600,000	--	--	$600,000
Miscellaneous Piping	LF	1,000	$100	$100,000	--	--	$100,000
Site Dewatering & Erosion Control	LS	1	$230,000	$230,000	--	--	$230,000
Painting	LS	1	$180,000	$180,000	--	--	$180,000
Miscellaneous Metals (Grating, Handrail etc.)	LS	1	$310,000	$310,000	--	--	$310,000
Site Restoration	SF	15,400	$2	$30,800	--	--	$31,000
Subtotal							**$11,200,000**
Electrical, Controls and Instrumentation (15% of Subtotal)							**$1,700,000**
General Conditions, Bonds & Insurance (5% of Subtotal)							**$600,000**
Contingency (20%)							**$3,400,000**
Total Probable Construction Cost							***$16,900,000***
Engineering, Administrative, and Legal (20%)							**$3,400,000**
Total Probable Project Cost (Rounded)							***$20,300,000***

(1) For items without installation cost, installation is included in material cost.
(2) Year 2012 Dollars.

The Engineer's Opinion of Probable Construction Cost for the new Actiflo® and Densadeg® systems are shown in Tables 7-15 and 7-16, respectively. In general, the Actiflo® system has higher equipment costs, but the Densadeg® system has higher construction costs associated with the larger dimensions.

7.4 ALTERNATIVE 3: EVALUATION OF AERATION ALTERNATIVES

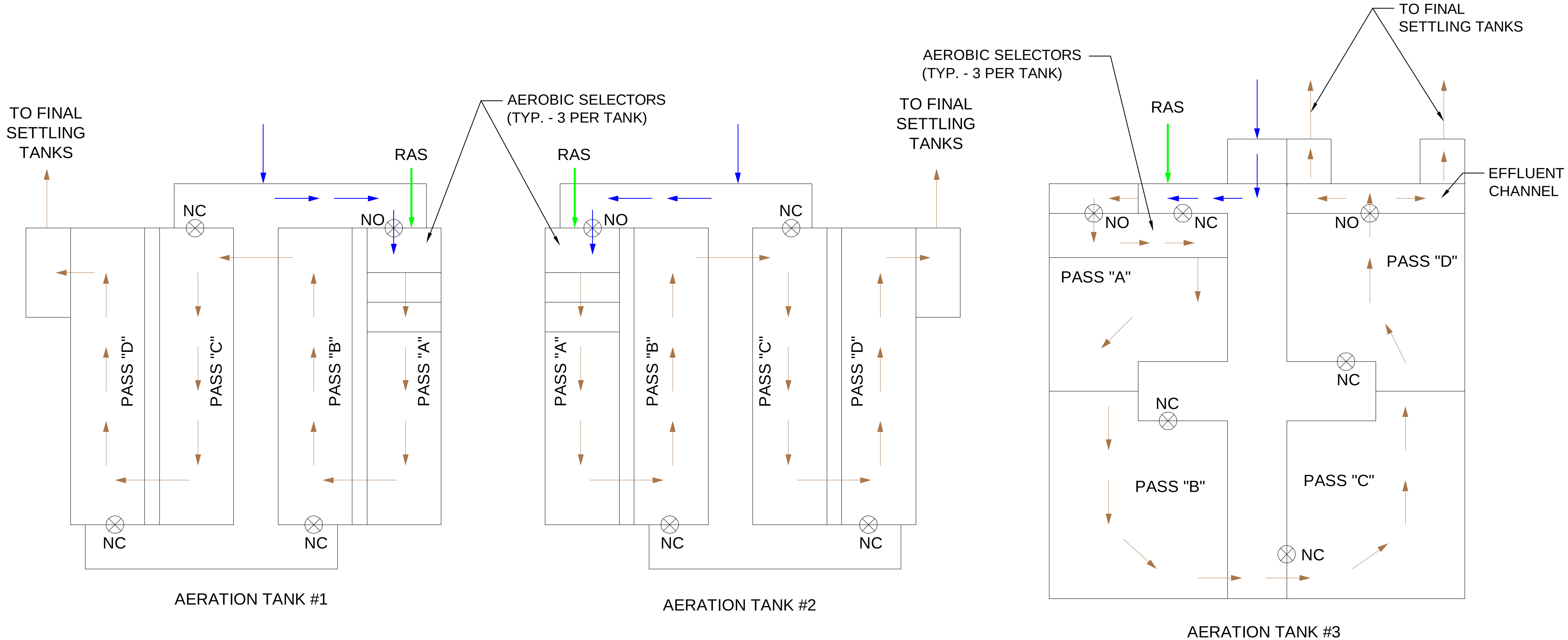
Several alternatives were evaluated to modify the existing aeration basins. The alternatives described in this Section include in-basin modifications and do not include the construction of new basins. Specifically, these alternatives include:

1. Addition of selectors in the upstream end of the basins (described in Section 7.4.1)
2. Conversion to step-feed operation (described in Section 7.4.2), and
3. Conversion to contact stabilization operation (described in Section 7.4.3).

7.4.1 Alternative 3A: Activated Sludge Selectors

Installing aerobic selector compartments at the head of the aeration basins would provide a slight increase in the process capacity of the basins without installing new tanks. Selectors are small tanks in which primary effluent is mixed with RAS. Typical selector tanks have a contact time of 20 to 60 minutes. Selectors are upstream of the activated sludge aeration basins. In the case of the WPCP, the selectors would be plug flow compartments as shown on Figure 7-19.

The selectors would have a high substrate concentration and promote the growth of floc-forming bacteria. The bacteria promoted by selectors would have better settling characteristics than filamentous bacteria. Although the installation would be straightforward, the operations of the



LEGEND

NO NORMALLY OPEN

NC NORMALLY CLOSED

FM FLOWMETER

SLIDE GATE

PRIMARY EFFLUENT

RAS FLOW

MIXED LIQUOR FLOW

WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

SELECTORS IN
AERATION BASINS

Job Number 8614782
Revision A
Date JUNE 2012

Figure 7-19

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selector zones would be somewhat complex. MLSS and F:M controls would be required to properly regulate the mixers in the selectors and the RAS rate.

The concept of selectors was reviewed with the District. The District prefers less complex modes of operation for the aeration basins and would not be interested in installing and maintaining MLSS and F:M controls associated with selectors. For this reason, the use of selectors was not evaluated in further detail.

7.4.2 Alternative 3B: Step-Feed Aeration

Converting the existing plug flow aeration basins to a step-feed configuration was evaluated. The original basis of design for the WPCP included step-feed operations at the aeration basins. Piping and isolation gates are in place for the step-feed mode, which makes the cost to convert nominal.

A schematic of step-feed operations in presented on Figure 7-20. In this mode, primary effluent would be evenly fed to all four (4) passes of each aeration basin. RAS would be pumped to the first pass of each basin only. By distributing the primary effluent, the food to mass (F:M) ratio is equalized throughout the basin and the peak oxygen demand decreases compared to a conventional plug flow reactor. The MLSS concentration is higher in the first pas, and decreases in subsequent passes as more primary effluent is introduced. With step-feed operations, a higher sludge retention time (SRT) can be maintained within the same volume.

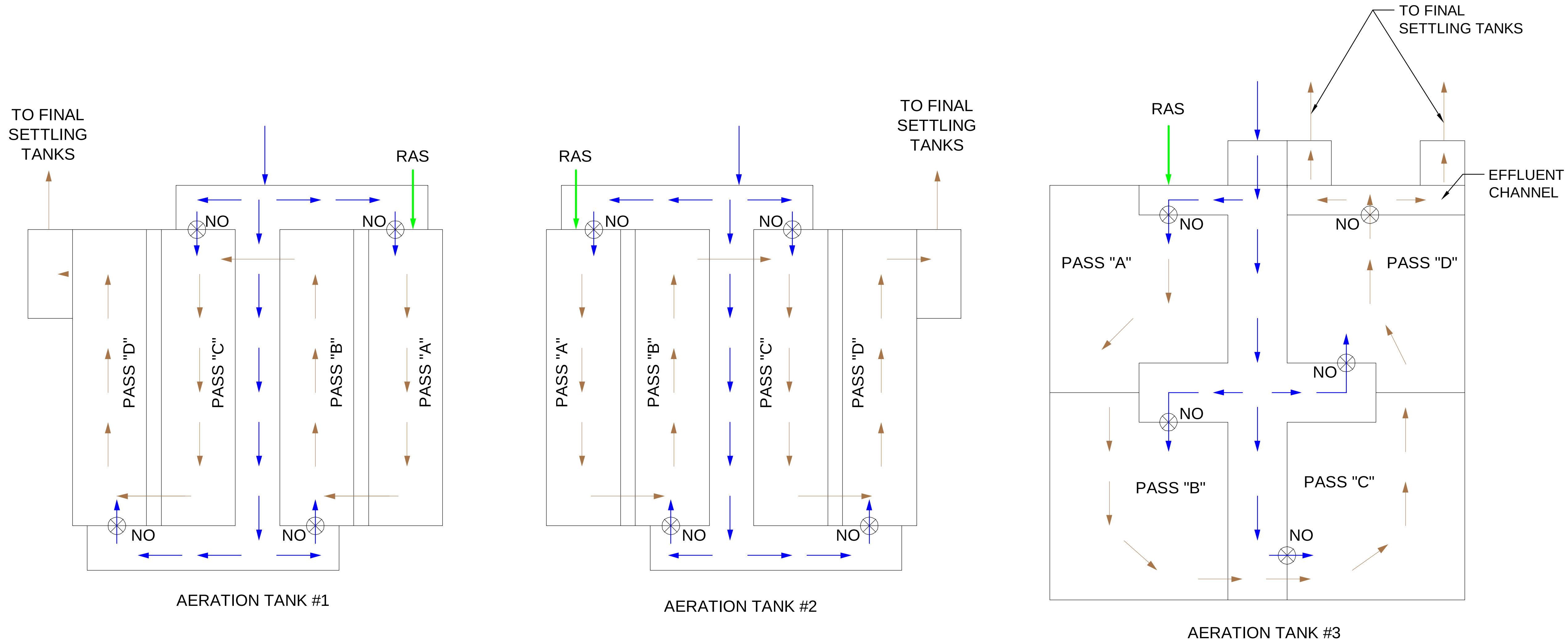
Based on the discussions presented in Section 5 of this Report, converting to step-feed operations could provide a process capacity for biochemical oxygen demand (BOD) removal and maintain an effluent NH_3 concentration of 3.0 mg/L for a peak flow of approximately 90 mgd. However, this flow rate cannot be hydraulically conveyed through the aeration basins. Even though the hydraulic capacity may not be available, converting to the step-feed mode of operation would return the plant to its original basis of design and may provide more operational flexibility for minimal cost.

7.4.3 Alternative 3C: Contact Stabilization

Contact stabilization was evaluated as an alternative to conventional or step-feed aeration. This process is common for plants which have a high degree of fluctuation in flow associated with wet weather conditions. Conversion to the contact stabilization mode could be accomplished at the WPCP by adjusting existing isolation gates, and new gates would not be required. Minimal piping would be required to redirect primary effluent. As with step-feed, switching to a contact stabilization mode would have minimal cost.

In the contact stabilization mode of operation, two (2) separate compartments are utilized. A schematic is provided on Figure 7-21. A "contact zone" is used to mix stabilized activated sludge with primary effluent. Primary effluent would be redirected to Pass D only, and Pass D of each basin could be utilized as the contact zone. Passes A through C of each basin would be utilized as the stabilization zone, where RAS is aerated and a significantly higher MLSS concentration is maintained. Since the MLSS concentration is so much higher in the stabilization zone, less aeration volume is required compared to conventional plug flow or step-feed.

Based on the capacity analysis presented in Section 5 of this Report, the contact stabilization mode could provide reliable BOD removal for a peak capacity of approximately 110 mgd. However, the contact stabilization method cannot be relied upon for NH_3 reduction due to the relatively short contact time in the contact zone. Since the WPCP does not have the hydraulic capacity for the peak flow which could be treated by contact stabilization, and since NH_3 reduction is not as achievable as plug flow or step-feed aeration, contact stabilization may not be the most preferred mode of operation.



LEGEND

NO NORMALLY OPEN

NC NORMALLY CLOSED

FM FLOWMETER

SLIDE GATE

PRIMARY EFFLUENT

RAS FLOW

MIXED LIQUOR FLOW

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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

STEP FEED AERATION

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Figure 7-20

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7.5 ALTERNATIVE 4: EVALUATION INTEGRATED FIXED FILM ACTIVATED SLUDGE (IFAS)

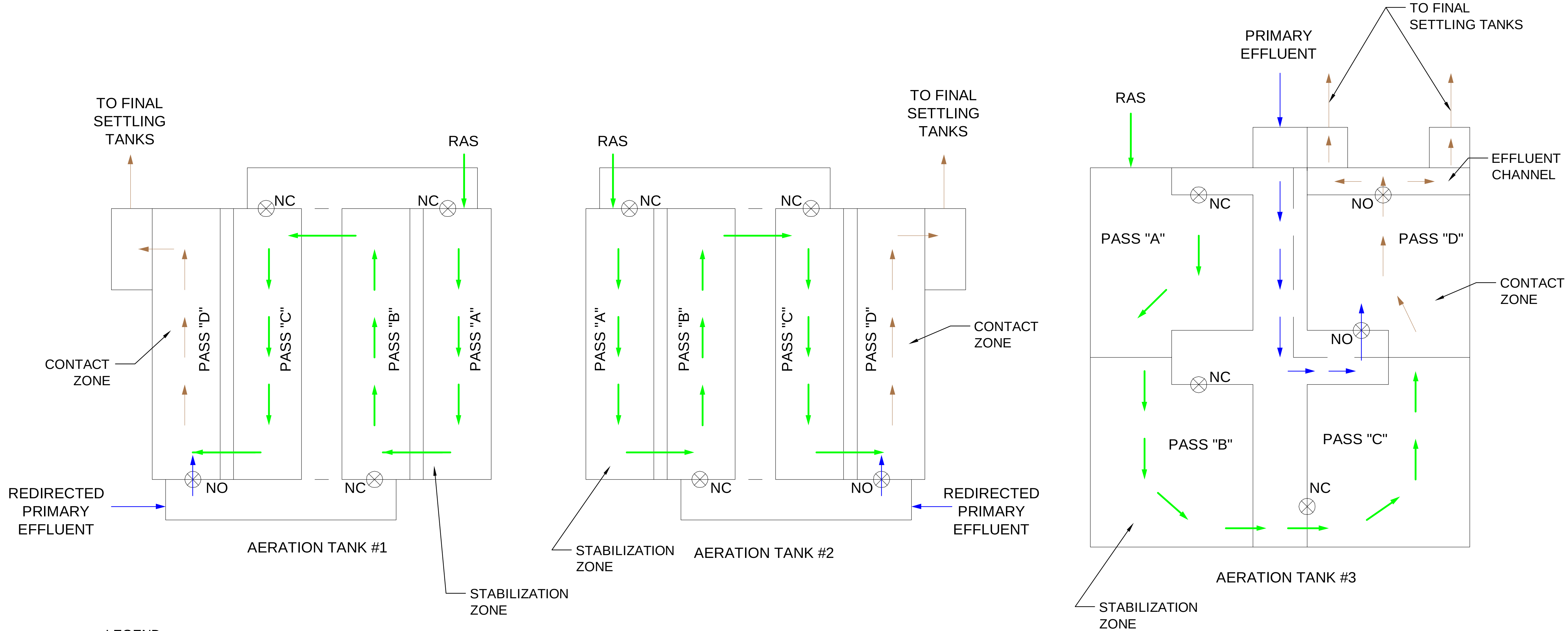
Integrated fixed film activated sludge (IFAS) was reviewed as another alternative to increase the capacity of the aeration process without constructing new tanks. The IFAS process described in this Section could be utilized with any of the aeration modes discussed in Section 7.4.

7.5.1 IFAS Process Description

IFAS is a relatively new process technology which combines attached growth treatment with conventional activated sludge. In this process, attached growth media is installed in the aeration basin to provide a surface area for biomass growth within the activated sludge basin. The attached growth surface area allows for the basin to be run at a significantly higher MLSS concentration. However, since the biomass population on the media is fixed, the solids loading to the downstream settling tanks would be lower than the MLSS concentration in the basin. At the WPCP, IFAS has the potential to increase the capacity of the aeration basins by running at a higher MLSS. At the same time, limiting the solids loading to the final setting tanks will maximize their capacity.

7.5.2 Alternative 4A: Fixed Media IFAS

IFAS media is available in fixed configurations. Plastic media, similar to trickling filter crossflow media, is common. Rope media has also utilized for some applications. For limited applications in New York State, the NYSDEC has indicated fixed plastic media is preferred to fixed rope media. Photographs of fixed plastic and rope media are provided on Figure 7-22.



LEGEND

NO NORMALLY OPEN

NC NORMALLY CLOSED

FM FLOWMETER

SLIDE GATE

PRIMARY EFFLUENT

RAS FLOW

MIXED LIQUOR FLOW

WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

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Revision A
Date JUNE 2012

CONTACT STABILIZATION

Figure 7-21

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Plastic Media, Courtesy Brentwood Industries

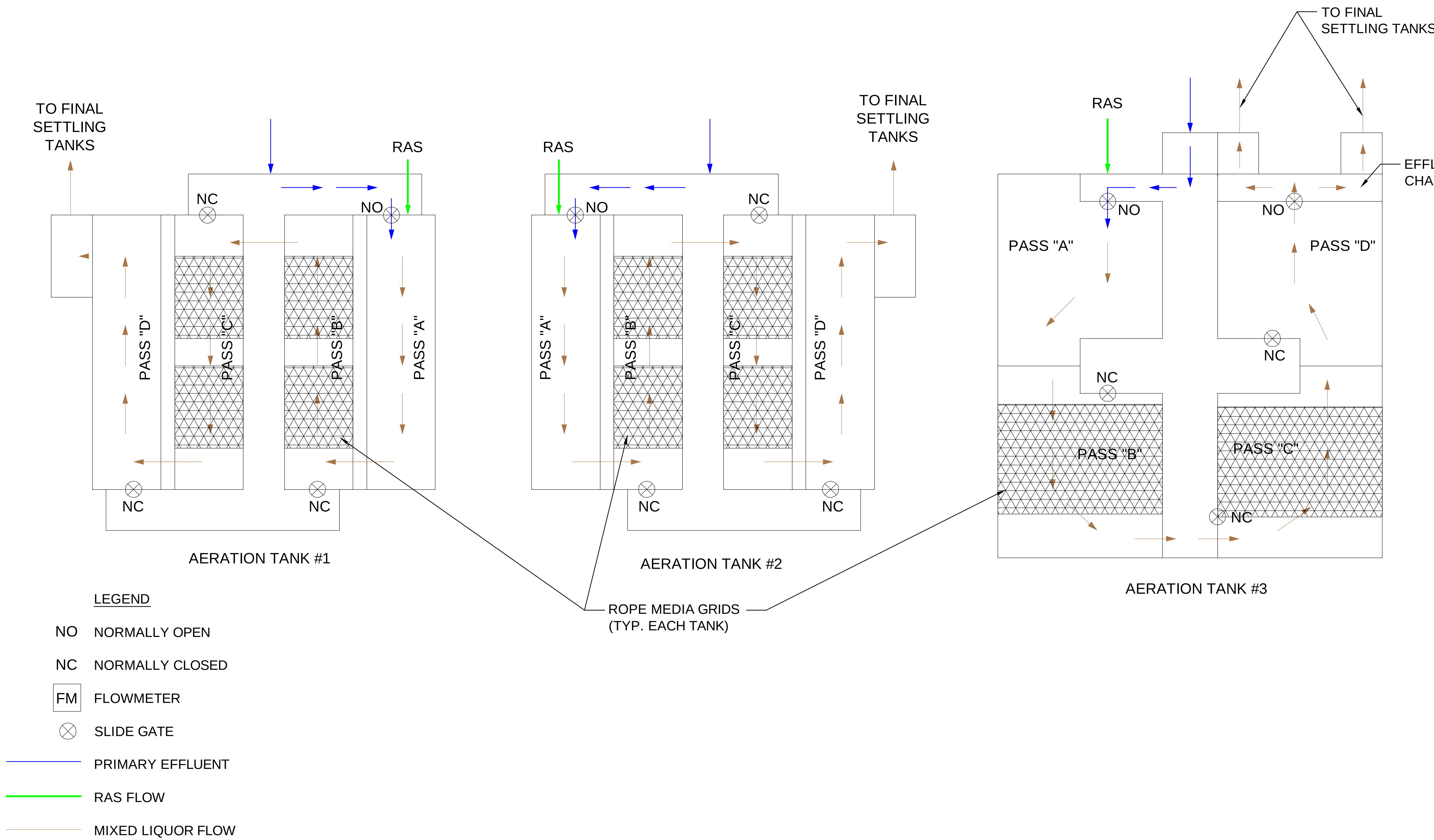
Rope Media, Courtesy Entex, Inc.

FIGURE 7-22

FIXED IFAS MEDIA

Figure 7-23 presents a potential layout for fixed media IFAS within the existing basins. Proposals for fixed media were received from manufacturers of plastic and rope units. For both types, the equipment cost is approximately $15,000,000 (including modifications to air distribution piping, media support systems, and media). When contractor installation costs are considered, the IFAS alternative could have a project cost greater than $20,000,000.

The proposals received from manufacturers for IFAS assumed in the maximum MLSS loading to the final settling tanks could be no greater than 1,800 mg/L as discussed in Section 5 of this Report. The basins would operate at approximately 3,600 mg/L MLSS, but approximately half of the biomass population would remain in the tanks, fixed to the IFAS media.



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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

FIXED MEDIA IFAS

Job Number 8614782
Revision A
Date JUNE 2012

Figure 7-23

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7.5.3 Alternative 4B: Random Media IFAS

IFAS media is also available as random plastic type media. A photograph of typical random media is provided on Figure 7-24.

Courtesy Parkson Corporation

FIGURE 7-24

RANDOM IFAS MEDIA

Random media tends to have a similar equipment cost as fixed media. However, random media requires "nets" or screens downstream of the aeration basins to prevent media units from passing to final settling tanks. In addition, random media is more prone to clogging than fixed media and requires fine screening upstream of the aeration basins. Due to these considerations, fixed media was considered the preferred IFAS alternative since the equipment costs are similar.

7.5.4 IFAS Conclusion

Both fixed and random media IFAS systems would increase the process capacity of the secondary treatment system. However, the secondary treatment system is still hydraulically limited to approximately 65 mgd. Due to this limitation, and the high capital costs associated with IFAS, it is not as practicable as the split flow alternative or aeration system modifications.

7.6 ALTERNATIVE 5: EVALUATION HIGH-RATE BALLASTED FLOCCULATION

High rate treatment (HRT) in the form of ballasted flocculation could be utilized to supplement the existing secondary treatment process. As discussed in previous sections, HRT systems are available with microsand or dense sludge as a ballast. HRT was previously evaluated as an alternative to conventional primary settling. However, HRT could also be employed downstream of conventional primary settling tanks to provide a degree BOD and TSS removal.

7.6.1 HRT Process Description

Installing HRT downstream of the primary settling tanks was not reviewed in detail for this Report. While HRT systems have demonstrated effective BOD and TSS removal similar to typical secondary treatment standards, they are not yet approved as equivalent to secondary treatment in New York State. Therefore, if HRT were to be utilized, it would be as a primary settling tank, or downstream of the combined flow primary settling tank in the split flow alternative. Refer to Section 7.3.7.3 for a more detailed description of where HRT could be employed for this project.

7.7 EVALUATION OF SLUDGE PROCESSING ALTERNATIVES

7.7.1 General

Sludge processing at the Oneida County WPCP currently includes the following:

- Gravity thickening of combined raw primary and waste activated sludge
- Belt filter press dewatering of gravity thickened sludge
- Fluidized bed incineration of dewatered sludge

- Gravity drainage and dewatering of incinerator ash in ash lagoons
- Landfill disposal of incinerator ash

To assess the impact of projected additional wastewater flow and associated pollutant loads on sludge processing facilities at the Oneida County WPCP, solids mass balances were prepared for projected future daily average and maximum month operating conditions. The solids balances, which were developed using the BioWin computer simulation model created for the WPCP, are presented in Appendix D-1.

7.7.2 Assessment of Sludge Thickening Capacity

Four 55-feet diameter gravity thickeners were constructed in 1968 to provide thickening of combined raw primary and waste activated sludge. Under current operating conditions, three gravity thickeners (Gravity Thickeners No. 1, 2, and 3) are normally in service. Based on discussions with the plant staff, Gravity Thickener No. 4 has never been used and is not currently operational.

As discussed previously in Chapter 5, the solids loading capacity of the three in-service gravity thickeners is estimated at 57,000 pounds dry solids per day. Although this capacity is adequate to handle the design maximum month mixed sludge solids loading of 50,600 pounds dry solids per day, the County may want to consider rehabilitation of Thickener No. 4 so that thickening capacity is available to handle the projected maximum month load with one unit out of service for scheduled maintenance or emergency repair. For the purpose of this report, capital cost estimates are included for replacement of internal sludge collection equipment as well as overflow weirs, grating and handrails.

7.7.3 Assessment of Sludge Dewatering Capacity

Six 2-meter belt filter presses are installed at the Oneida County WPCP for dewatering of gravity thickened sludge prior to incineration. Two belt presses are dedicated to each of the three fluidized bed incinerators. Belt Filter Presses No. 3 and No. 4, which are dedicated for use with Incinerator No. 2 are no longer operational. These units, which were installed in 1968, have been scavenged for parts. Belt Filter Presses No. 5 and 6 were installed in 1984 and are used in connection with Incinerator No. 3. Belt Filter Presses No. 1 and 2 were installed in 2005 and are used in connection with Incinerator No. 1.

According to the plant Operation and Maintenance Manual, each belt filter press is rated for a dry solids throughput capacity of 780 pounds of dry solids per hour. Under current operating conditions, two incinerators normally operate on a continuous basis (24 hours per day, 7 days per week). Based on this operating schedule, four belt filter presses are capable of dewatering 74,880 pounds dry solids per day. This capacity is more than adequate for processing the design maximum month thickened sludge solids load of 48,100 pounds dry solids per day.

7.7.4 Assessment of Sludge Incineration Capacity and Emission Regulations

The Oneida WPCP staff has successfully operated and maintained fluidized bed incinerators for many years since initial installation of Incinerators No. 1 and No. 2 in 1968, subsequent installation of Incinerator No. 3 in 1984, and upgrade of Incinerators No. 1 and No. 3 in 2005. At the time of this report, only one incinerator (Incinerator No. 1) was operational. Repairs were being made so that Incinerator No. 3 could be returned to service. Temporary sludge dewatering and post-lime sludge stabilization was being provided on a contract basis until the repairs could be made to Incinerator No. 3. Incinerator No. 2 is no longer operational and will need to be demolished. Incinerator components containing asbestos will need to be removed in accordance with applicable state and federal regulations when demolition occurs.

Based on the results of stack testing performed in October 2006, the dry solids loading capacity of each incinerator is estimated at 1,640 pounds dry solids per hour. With two incinerators operating continuously, the total sludge incineration capacity is estimated at 78,720 pounds dry solids per day. This capacity is adequate for handling the design maximum month dewatered sludge solids load of 47,100 pounds dry solids per day.

For continued long-term operation, the two operating incinerators (Incinerators No. 1 and No. 3) should be upgraded to replace components that are nearing the end of their useful life. In addition, because current and projected future sludge quantities require operation of two incinerators, the third incinerator (Incinerator No. 2), which is currently not operational, will need to be replaced in order to provide necessary redundancy for those occasions when an incinerator must be removed from service for scheduled maintenance or emergency repairs.

In addition, on March 21, 2011, USEPA enacted final regulations governing air emissions from new and existing sewage sludge incinerators. The regulations establish emission limits for nine pollutants and require stack testing, monitoring, recordkeeping and operator training for compliance.

Oneida County has not yet performed stack testing to assess the status of compliance with the new emissions limits. However, the results from stack testing conducted by Oneida County in October 2006 suggest that the incinerators appear to comply with the new emission limits for existing incinerators, including the limit for mercury. However, the capital costs developed for alternatives with rehabilitation of Incinerators Nos. 1 and 3 include budgetary pricing for mercury reduction systems on these existing units. As shown in Table 7-17, the October 2006 stack testing did not include testing for all parameters regulated under the new regulations. Stack testing is required to confirm that incinerator modifications are not required for compliance with all of the new emissions regulations.

TABLE 7-17

INCINERATOR STACK TEST RESULTS (OCTOBER 2006)

POLLUTANT	**UNITS**	**EMISSION LIMIT (40 CFR 60, MARCH 2011)**		**STACK TEST (OCT. 2006)**	**NOTES**
		EXISTING UNITS	**NEW UNITS**		
Particulate Matter	mg/dscm at 7% O2	18	9.6	0.0046 gr/dscf. 0.3 mg/dscf	1
Sulfur Dioxide	ppmv at 7% O2	15	5.3	6.65 ppm	
Carbon Monoxide	ppmvd at 7% O2	64	27	3.6 ppm	3
Nitrogen Oxides	ppmvd at 7% O2	150	30		2
Cadmium	mg/dscm at 7% O2	0.0016	0.001	0.00047 mg/dscm	
Hydrogen Chloride	ppmvd at 7% O2	0.51	0.24		2
Mercury	ug/dscm at 7% O2	0.037	0.001	0.006 mg/dscm	
Lead	mg/dscm at 7% O2	0.004	0.00062	0.00289 mg/dscm	4
Dioxins/Furans, TEQ	ug/dscm at 7% O2	0.1	0.004		2
Dioxins/Furans, TMB	ug/dscm at 7% O2	(optional) 0.10	0.013		
Opacity	%			0	

1. Reported as grains per dry standard cubic feet. Equivalent mg/dscf shown.
2. Not reported to date
3. Reported as parts per million
4. Reported as milligrams per cubic meter

The new regulations define an “existing” incinerator as an incinerator for which the cost of rehabilitation does not exceed 50% of the original installation cost (adjusted for inflation). Under the regulations, Incinerators No. 1 and No. 3 are defined as existing incinerators. If Incinerator No. 2 is replaced, the new incinerator would have to comply with the more stringent emission limits for new units. Since the October 2006 stack test results suggest that emissions from the existing incinerators exceed the new incinerator emission limit for mercury, installation of a new incinerator to replace Incinerator No. 2 would likely require a mercury reduction system. Although mercury reduction systems may not be required on Incinerator Nos. 1 and 3, capital costs are included for a mercury reduction system for alternatives with continued use of these two units in the event that revised stack testing suggests mercury removal may be required.

7.7.5 Evaluation of Alternatives

Because of the significant costs (capital cost for replacing Incinerator No. 2 as well as annual operating cost for fuel) associated with continued sludge incineration and because of the potentially significant cost and current uncertainty regarding compliance with the new sewage sludge incinerator emissions regulations, an evaluation was conducted to assess the feasibility and cost of alternatives to sludge incineration. Potential alternatives that were considered include anaerobic sludge digestion (with use of a gas-engine generator system for energy recovery from digester gas) and post-lime sludge stabilization.

The feasibility and cost of replacing the existing belt filter presses with centrifuges for sludge dewatering was also included in the evaluation of sludge processing alternatives. Centrifuge sludge dewatering is typically capable of dewatering sludge to a greater degree than belt filter press dewatering. Based on analysis of plant operating data, the belt filter presses produce dewatered sludge having a solids concentration ranging from 16 to 24 percent total solids, with an average of 20 percent total solids. In general, centrifuges are typically capable of increasing the dewatered sludge solids concentration by 4%, or more, compared to belt filter presses (i.e. 20 to 28 percent total solids with an average of 24 percent total solids).

The increase in dewatered sludge solids concentration possible with centrifuge sludge dewatering has potentially significant cost benefits including significant reduction of auxiliary fuel required for sludge incineration and significant reduction of dewatered sludge hauling and disposal costs for sludge incineration alternatives. The potential benefit of reduced fuel costs of sludge incineration are offset to some extent by increased power costs for centrifuge sludge dewatering.

In addition, for alternatives involving continued sludge incineration, potential cost savings resulting from conversion of the fluidized bed incinerators for use of natural gas in lieu of fuel oil was of interest. A 4-inch natural gas line was recently installed in the vicinity of the incinerators. Extension of the gas line appears to be possible at relatively little cost. Extension of the gas line coupled with replacement of the incinerator pre-heat burners and bed guns for use with natural gas would enable the County to take advantage of the significant cost advantage that natural gas currently has over fuel oil.

The alternatives evaluated in connection with this report are briefly described as follows:

Alternative 1 – This alternative involves continued processing and disposal of sewage sludge by thickening of combined raw primary and waste activated sludge in gravity thickeners, dewatering of thickened sludge using belt filter presses, fluidized bed incineration of dewatered sludge, and landfill disposal of incinerator ash after drainage and dewatering in existing ash lagoons. Capital costs associated with this alternative include:

- Renovations to restore Gravity Thickener No. 4 to service, including replacement of internal equipment, weirs, grating and handrail
- In-kind replacement of Belt Filter Presses No. 3, 4, 5 and 6

- Demolition of Incinerator No. 2
- Installation of a new fluidized bed incinerator, including mercury reduction system for emissions control, to replace Incinerator No. 2
- Renovations to Incinerators No. 1 and 3

Alternative 2 – This alternative is the same as Alternative 1, except for sludge dewatering. Under this alternative, all six existing belt filter presses would be demolished and three new high-solids centrifuges and appurtenant equipment would be installed for sludge dewatering. Capital costs associated with this alternative include:

- Renovations to restore Gravity Thickener No. 4 to service, including replacement of internal equipment, weirs, grating and handrail
- Demolition of six existing belt filter presses, including appurtenant equipment, instrumentation, piping and controls
- Installation of three high-solids centrifuges, including appurtenant equipment, instrumentation, piping and controls
- Demolition of Incinerator No. 2
- Installation of a new fluidized bed incinerator, including mercury reduction system for emissions control, to replace Incinerator No. 2
- Renovations to Incinerators No. 1 and 3

Alternative 3 – This alternative is the same as Alternative 1, except the three existing incinerators would be demolished and replaced with an anaerobic sludge digestion system. Capital costs associated with this alternative include:

- Renovations to restore Gravity Thickener No. 4 to service, including replacement of internal equipment, weirs, grating and handrail

- Construction of three 75-feet diameter anaerobic digesters (2 primary and 1 secondary), digester control house and cogeneration system for energy recovery from digester gas

- In-kind replacement of Belt Filter Presses No. 3, 4, 5 and 6

- Demolition of three existing fluidized bed sludge incinerators

Alternative 4 – Except for sludge dewatering, this alternative is the same as Alternative 3. Under this alternative, the six existing belt filter presses and appurtenant equipment would be replaced with three new high-solids centrifuges. Capital costs associated with this alternative include:

- Renovations to restore Gravity Thickener No. 4 to service, including replacement of internal equipment, weirs, grating and handrail

- Construction of three 75-feet diameter digesters (2 primary and 1 secondary), digester control house and cogeneration system for energy recovery from digester gas

- Demolition of six existing belt filter presses, including appurtenant equipment, instrumentation, piping and controls

- Installation of three high-solids centrifuges, including appurtenant equipment, instrumentation, piping and controls

- Demolition of three existing fluidized bed sludge incinerators

Alternative 5 – This alternative is the same as Alternative 1, except the three existing incinerators would be demolished and replaced with a post-lime sludge stabilization system. Capital costs associated with this alternative include:

- Renovations to restore Gravity Thickener No. 4 to service, including replacement of internal equipment, weirs, grating and handrail
- In-kind replacement of Belt Filter Presses No. 3, 4, 5 and 6
- Demolition of three existing fluidized bed sludge incinerators
- Construction of a post-lime sludge stabilization system

Alternative 6 – Except for sludge dewatering, this alternative is the same as Alternative 5. Under this alternative, the six existing belt filter presses and appurtenant equipment would be replaced with three new high-solids centrifuges. Capital costs associated with this alternative include:

- Renovations to restore Gravity Thickener No. 4 to service, including replacement of internal equipment, weirs, grating and handrail
- Demolition of six existing belt filter presses, including appurtenant equipment, instrumentation, piping and controls
- Installation of three high-solids centrifuges, including appurtenant equipment, instrumentation, piping and controls
- Demolition of three existing fluidized bed sludge incinerators
- Construction of a post-lime sludge stabilization system

Alternative 7 – This alternative is the same as Alternative 1, except that a post-lime sludge stabilization system would be installed for use as a backup system instead of installing a new fluidized bed incinerator to replace Incinerator No. 2. Capital costs associated with this alternative include:

- Renovations to restore Gravity Thickener No. 4 to service, including replacement of internal equipment, weirs, grating and handrail
- In-kind replacement of Belt Filter Presses No. 3, 4, 5 and 6
- Demolition of Fluidized Bed Sludge Incinerator No. 2
- Renovations to Fluidized Bed Sludge Incinerators No. 1 and 3
- Construction of a post-lime sludge stabilization system having capacity to serve as a standby system when scheduled maintenance or emergency repairs require removal of one fluidized bed incinerator from service

Alternative 8 – Except for sludge dewatering, this alternative is the same as Alternative 7. Under this alternative, the six existing belt filter presses and appurtenant equipment would be replaced with three new high-solids centrifuges. Capital costs associated with this alternative include:

- Renovations to restore Gravity Thickener No. 4 to service, including replacement of internal equipment, weirs, grating and handrail
- Demolition of six existing belt filter presses and appurtenant equipment, instrumentation, piping and controls
- Installation of three high-solids centrifuges and appurtenant equipment, instrumentation, piping and controls
- Demolition of Fluidized Bed Sludge Incinerator No. 2
- Renovations to Fluidized Bed Sludge Incinerators No. 1 and 3

- Construction of a post-lime sludge stabilization system having capacity to serve as a standby system when scheduled maintenance or emergency repairs require removal of one fluidized bed incinerator from service

In addition, two sub-alternatives were evaluated for Alternatives 1, 2, 7 and 8. These sub-alternatives involved continued use of fuel oil for sludge incineration (Alternatives 1A, 2A, 7A and 8A) and modifications for use of natural gas for sludge incineration (Alternatives 1B, 2B, 7B and 8B).

Solids mass balances were prepared for each alternative using the BioWin computer simulation model that was developed for the Oneida WPCP plant. Solids balances for design daily average and maximum month operating conditions are presented in Appendix D-1.

Based on the solids mass balances, preliminary design information was developed for each of the sludge processing alternatives. This information, which is presented in Appendix D-2, was used as the basis for development of capital and annual operation and maintenance cost estimates for each alternative.

Estimated capital and annual operating and maintenance costs for each alternative are presented in Appendix D-3 and summarized in Table 7-18. The basis for estimated annual operating and maintenance costs included the following:

- Incinerator ash/dewatered sludge disposal cost = $72.15 (current cost including hauling)
- Lime dosage = 100 pounds per wet ton of sewage sludge
- Lime cost = $0.14 per pound
- Dewatered sludge solids concentration (belt filter press) = 20%
- Dewatered sludge solids concentration (centrifuge) = 24%

- Electric cost = \$0.09 per kwh

- Natural gas cost = \$7.30 per MMBtu

For comparison of 20-year life cycle costs, annual O&M costs were converted to net present worth costs using an annual interest rate of 4 percent.

TABLE 7-18

NET PRESENT WORTH COSTS FOR SLUDGE PROCESSING ALTERNATIVES

	CAPITAL COST (1)	O&M COSTS: ANNUAL COST (1)	O&M COSTS: NET PRESENT WORTH (2)	TOTAL PRESENT WORTH COST
Alternative 1 [3]				
1A	\$ 58,100,000	\$ 1,765,000	\$ 24,000,000	\$ 82,100,000
1B	\$ 58,100,000	\$ 1,165,000	\$ 15,800,000	\$ 73,900,000
Alternative 2 [3]				
2A	\$ 59,800,000	\$ 1,055,000	\$ 14,300,000	\$ 74,100,000
2B	\$ 59,800,000	\$ 855,000	\$ 11,600,000	\$ 71,400,000
Alternative 3	\$ 36,600,000	\$ 1,110,000	\$ 15,100,000	\$ 51,700,000
Alternative 4	\$ 38,600,000	\$ 925,000	\$ 12,600,000	\$ 51,200,000
Alternative 5	\$ 15,800,000	\$ 2,745,000	\$ 37,300,000	\$ 53,100,000
Alternative 6	\$ 17,300,000	\$ 2,320,000	\$ 31,500,000	\$ 48,800,000
Alternative 7 [3]				
7A	\$ 24,400,000	\$ 1,835,000	\$ 24,900,000	\$ 49,300,000
7B	\$ 24,400,000	\$ 1,275,000	\$ 17,300,000	\$ 41,700,000
Alternative 8 [3]				
8A	\$ 26,200,000	\$ 1,185,000	\$ 16,100,000	\$ 42,300,000
8B	\$ 26,200,000	\$ 990,000	\$ 13,500,000	\$ 39,700,000

[1] See Appendix D-3 for detail

[2] Present worth factor = 13.59 (4% interest, 20 years)

[3] Alternatives 1, 2, 7, and 8 include a mercury reduction system on Incinerators 1 and 3, which may not be required depending on stack test results

As shown in Table 7-18, Alternative 8, is the least cost alternative for sludge processing. This alternative includes replacement of existing belt filter presses with centrifuges for sludge dewatering, demolition of Incinerator No. 2 and rehabilitation of Incinerators No. 1 and No. 3, and construction of a post-lime sludge stabilization system to serve as a standby system ready to be placed into service on those occasions when Incinerator No. 1 or No. 3 must be taken out of service for scheduled maintenance or emergency repair.

Although the net present worth cost for Alternative 7 has been estimated to be slightly greater than the net present worth cost for Alternative 8, a more detailed evaluation of the potential benefits and associated costs of replacing the existing belt filter presses with centrifuges for sludge dewatering is recommended before proceeding to final design. This analysis should include pilot-scale testing to demonstrate potential centrifuge sludge dewatering performance capabilities and to confirm critical assumptions used as the basis for the cost comparison presented in this report.

As shown in Table 7-18, the net present worth cost for continued use of fuel oil for sludge incineration (Alternative 8A) is slightly greater than the net present worth cost for conversion of Incinerators No. 1 and No. 3 for operation using natural gas (Alternative 8B). This suggests that conversion of Incinerators No. 1 and No. 3 for operation using natural gas is cost effective. However, the magnitude of this difference in cost is relatively small (less than 10% increase in cost). Before proceeding with final design, a more detailed evaluation is recommended to confirm the economic benefits of converting from fuel oil to natural gas for operation of the sludge incinerators.

7.8 EVALUATION OF OTHER WPCP IMPROVEMENTS

In 2010, a physical condition assessment was prepared for the WPCP. This assessment included a physical inspection of the structural, architectural, electrical, and mechanical components of the WPCP. The assessment provided an estimate on the remaining life of major facility components and approximate replacement values.

The costs identified in the condition assessment should be considered when evaluating the overall WPCP expansion, since the existing equipment and tankage must remain in service after the plant expansion.

The costs identified in the 2010 condition assessment are summarized in Table 7-19. These costs assume the WPCP will be expanded by means of the split flow concept discussed in Section 7.3. For any existing processes not modified under the split flow concept, costs to maintain existing facilities in operation must be considered. The costs are based on immediate needs (likely required in the next five (5) years), and short-term (likely required in the next ten (10) years). The immediate and short term needs would need to be addressed prior to or while expanding the WPCP.

7.9 EVALUATION OF ELECTRICAL IMPROVEMENTS

If the WPCP is expanded to accept and treat additional flows and loads, the existing electrical system will also require modifications. The existing electrical system operates near or at capacity in many areas of the WPCP. A report was prepared to review the existing electrical facilities, and identify potential upgrades to provide electrical service to new facilities proposed for the WPCP expansion. This report is provided in Appendix E. The basis for the electrical evaluation was the split flow concept discussed in Section 7.3.

7.10 OTHER CONSIDERATIONS

The costs presented in Sections 7.2 through 7.9 are broken down by individual unit processes. Additional costs would be required for piping between the new processes (including re-routing existing interceptors). The exact piping route would be determined during the final design. A probable project cost associated with new yard piping is approximately $1,500,000 based on:

- 1,000 LF of large diameter interceptor re-routing at $1,000 per LF due to the deep excavation
- 1,000 LF of new large diameter yard piping within the WPCP at $500 per LF

Constructability concerns would also add cost to the project, as temporary bypass pumping and flow diversions may be required to keep the existing WPCP in operation while new facilities are constructed. Although the sequencing plan and constructability issues would be worked out during final design of the WPCP expansion, temporary bypass pumping and flow diversions throughout construction could add $1,000,000 to the overall project cost.

8.0 EVALUATION OF SCPS

8.1 EVALUATION OF SCPS UPGRADE

This Section describes the planned upgrade of the SCPS Force Main (FM) in support of sewer overflow abatement. Previous modeling, as summarized in Section 3, has shown SSO mitigation for the 2008 monitoring period by increasing the existing capacity at SCPS from 15 MGD to 35 MGD, and I/I reduction efforts in the SCPS basin. Previous upgrades at SCPS have increased the capacity of the pumps to keep up with influent flow. Because the wet-weather flow already being pumped will be diverted from the overflow to the new FM, modifications inside SCPS will be minimal.

8.1.1 SCPS Pumps

Hydraulic analyses show the existing three (3) operating pumps can meet the required hydraulic conditions with an increase in FM capacity (see Figure 8-1). The alternatives analysis determined a parallel 36-inch DR 17 high density polyethylene (HDPE) FM will provide the best benefit over costs, as compared to adding pump capacity. However, there is room for a fourth pump should an increase in capacity be needed in the future. Additional information on the forcemains is presented in Section 8.2.

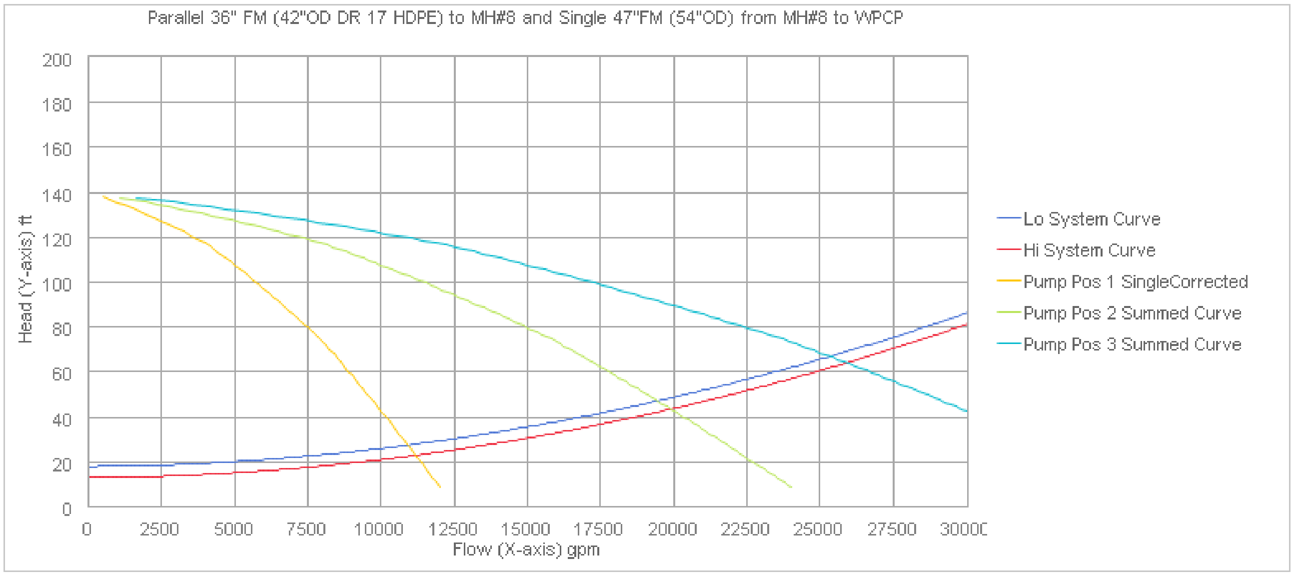


FIGURE 8-1

HYDRAULIC ANALYSIS OF EXISTING SCPS CAPACITY

8.1.2 SCPS Screens

The existing climber screen will be replaced with a new mechanical screen rated for 38 MGD. A second mechanical screen in parallel with the first will also be rated at 38 MGD. It is recommended that the bar spacing be 1/2-inch to protect the pumps and not burden the pump station with screenings removal. To prevent screening SCPS flows twice, the new SCPS forcemain will discharge to the WPCP downstream of screening facilities.

The mechanical screens at the SCPS will be on emergency power and therefore operable under all power conditions.

8.1.3 SCPS Vaults

The gate controlling overflow (and hence, flow to the WPCP) is manually set, and therefore difficult to dynamically operate to minimize overflows. Also, the existing flow totalizer does not provide the resolution of data required for the permanent metering plan. To improve operations, provide a means to connect the new forcemain to the pump discharge and to provide a more robust metering location, a new metering vault is proposed. This vault, shown in Figure 8-2, will contain hydraulically actuated valves to control flow to the parallel forcemains and downstream flow meters to provide flow monitoring.

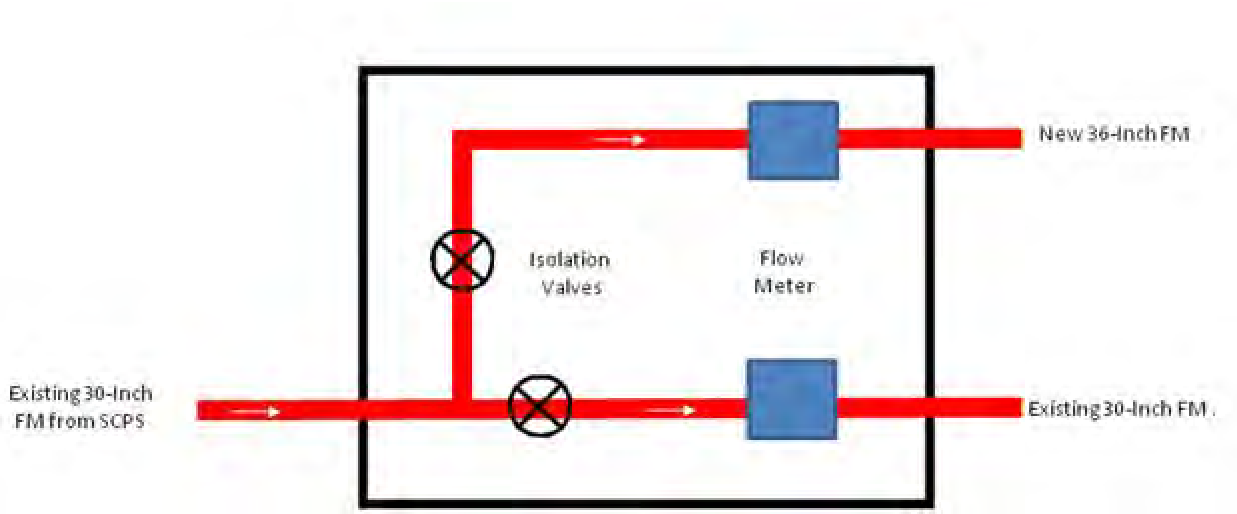


FIGURE 8-2

JUNCTION BOX AT SCPS

The flow meters will be used to regulate flow between the two (2) forcemains via the automated valves and will be programmed to maintain sufficient scouring velocities between the two (2) forcemains, isolating forcemains as required. User overrides will dictate which forcemain remains in operation.

A second vault will be needed along Leland Avenue upstream of the WPCP where the existing SCPS FM discharges into the Mohawk River Interceptor. The SCPS FM diameter will increase to 54-inches at this location. This vault will contain manual valves which will allow discharge to either the Mohawk interceptor or the new 54-inch forcemain, as shown in Figure 8-3.

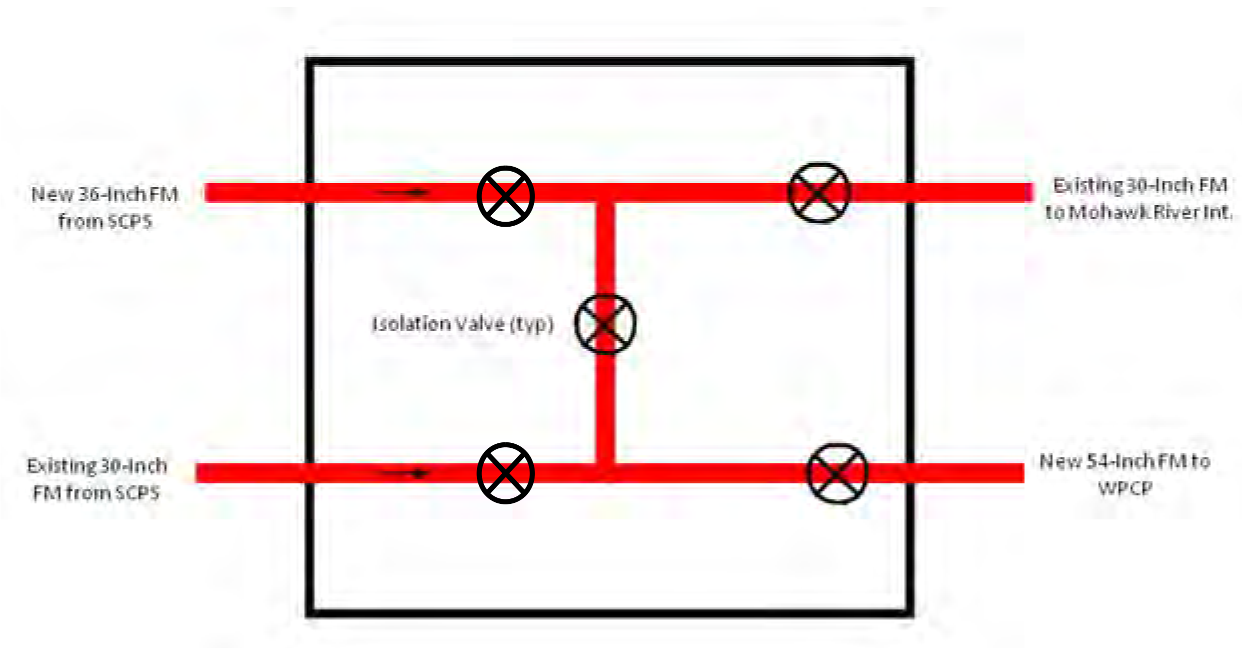


FIGURE 8-3

SCPS FM TERMINUS JUNCTION BOX

8.2 EVALUATION OF NEW FORCEMAIN TO WPCP

The alternatives analysis concluded a new force main from SCPS to the WPCP would provide the most benefit in pipe capacity and operational improvement over a more limited new FM from Barnes Avenue Pump Station to the WPCP, or replacing the existing 30-inch RCP FM.

8.2.1 FM Alignment

The most cost effective and constructible alignment for the new FM is mostly parallel to the existing FM, within the existing easement. Some challenges to constructability include crossing of a heavy rail line, major multi-lane roads, and contaminated soils. To meet these challenges, the selective use of directional drilling for pipe installation will minimize impacts and costs. Figure 8-4 provides an aerial photo of the FM alignment.

Path: P:\Oneida\133729 Oneida SPDES Permitting CSO-CSO\GIS\SCPS_New_FM_Alignment_Sheet1.mxd User: csomerlot Date Saved: 3/1/2012 11:18:07 PM

FIGURE 8-4
SCPS FM ALIGNMENT

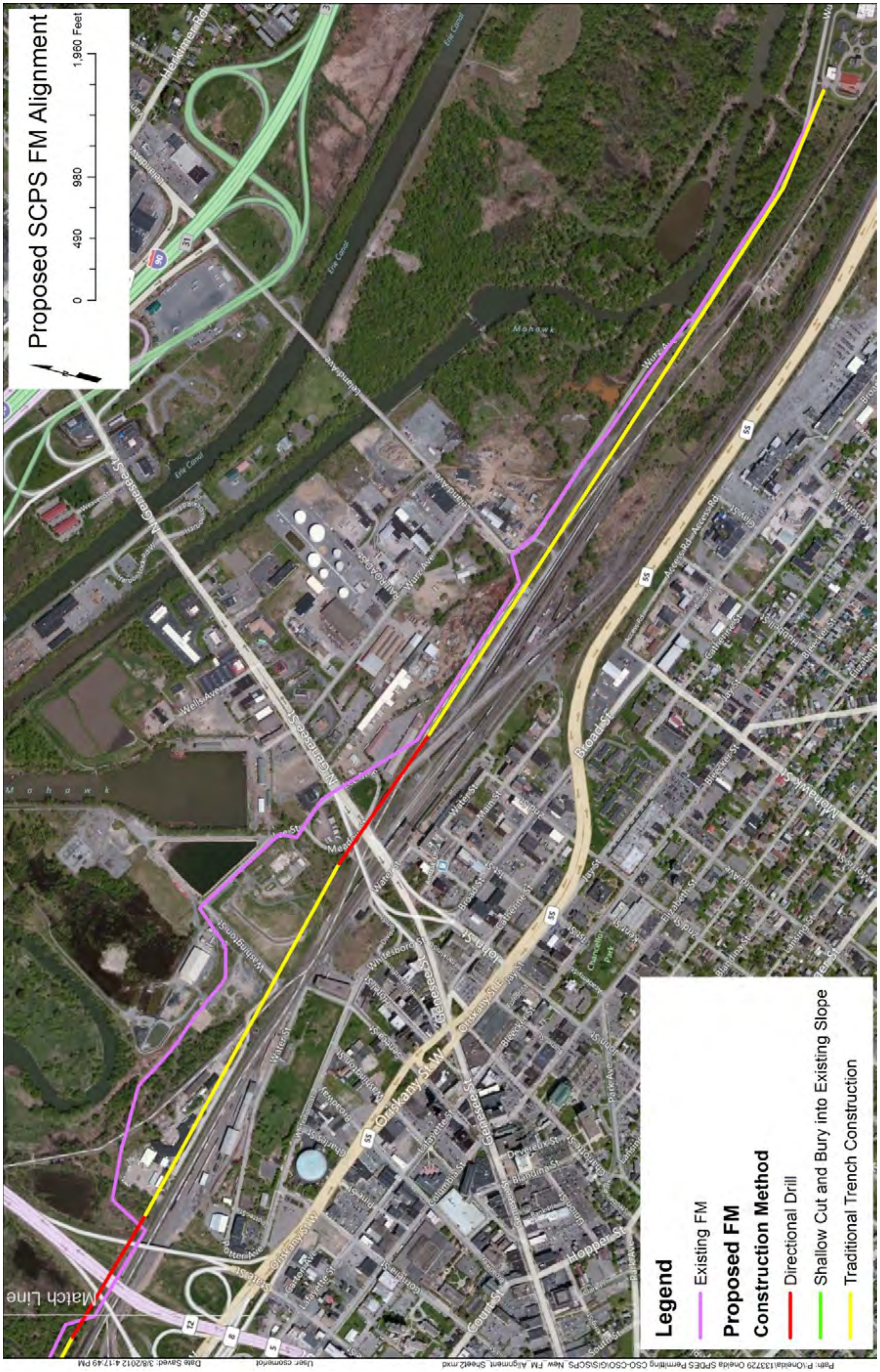


FIGURE 8-4 (Continued)
SCPS FM ALIGNMENT

Another construction challenge is installation along and through a large wetland area. In this area, the existing FM easement runs parallel to the railroad easement and embankment. To minimize cost and impact to the wetland hydrology, we propose to install the new FM near or at the existing surface grade along the toe of the embankment, and fill to cover it to a depth of a few feet. Depending on soil conditions, pipe support piles may be needed. Several H-20 rated vehicular crossings will be added to improve access to the easement for maintenance and inspection activities. See Figure 8-5 for a typical section of the proposed FM. In the remaining areas of the alignment, traditional open-trench cut and fill construction will be used.

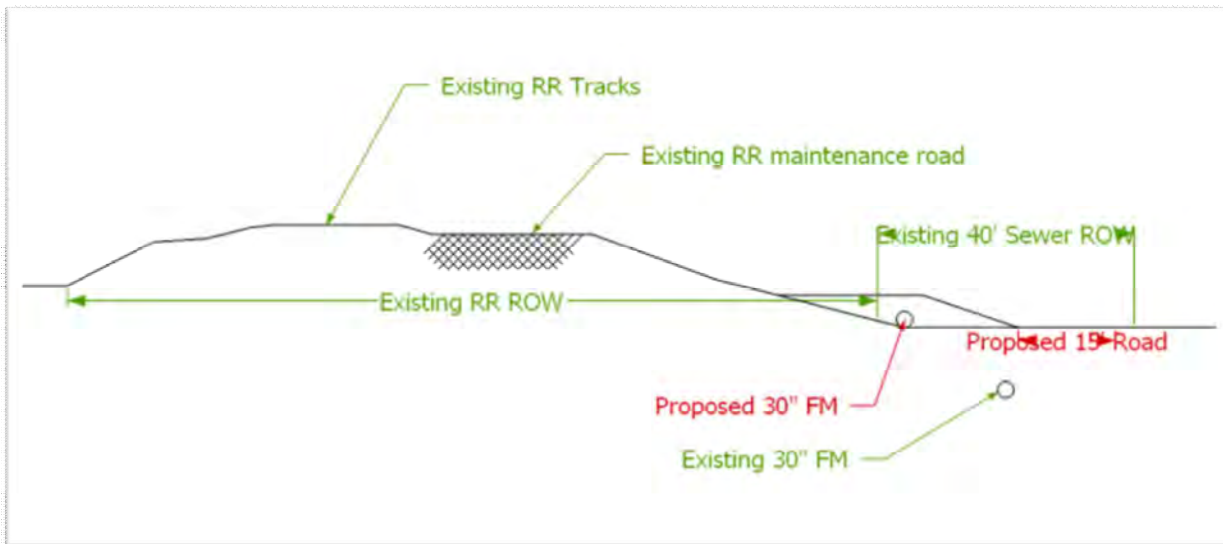


FIGURE 8-5

TYPICAL SECTION OF SCPS FM

8.3 SCPS COST

A planning-level cost estimate, with appropriate contingency was completed. This planning level cost is broken down on Table 8-1.

TABLE 8-1

ENGINEERS OPINION OF PROBABLE COST: SCPS DISCHARGE FORCEMAIN

ITEM	UNIT	ESTIMATED QUANTITY	INSTALLED COST (1)
SCPS Screens (2)	LS	1	$515,000
Traffic Control	LS	1	$5,000
Connection to WPCP Vault (2)	LS	1	$75,000
SCPS Vault (2)	LS	1	$110,000
Barnes Avenue Connection and Vault	LS	1	$20,000
Leland Avenue Connection and Vault	LS	1	$20,000
Open-cut Installation of 36-inch HDPE	LF	10,820	$4,740,000
Directional Drill of 36-inch HDPE	LF	3,230	$4,300,000
Shallow-Bury/Trench of 36-in HDPE	LF	9,270	$1,620,000
Land Acquisition/Easements	LS	1	$250,000
480 V Main Switchboard, ATS, Breakers	LS	1	$110,000
250 HP VFDs, 480 V	LS	1	$140,000
900 kW Natural Gas Generator w/ Enclosure	LS	1	$840,000
Load Bank	LS	1	$55,000
Demolition of Existing Electrical	LS	1	$25,000
Wiring	LS	1	$120,000
Rental Generator for Two (2) Weeks	LS	1	$20,000
Utility Coordination	LS	1	$50,000
13.2kV to 480 V Transformer	LS	1	$50,000
Subtotal (Including Electrical)			$13,100,000
Controls and Instrumentation (10% of Subtotal)			$1,310,000
General Conditions, Bonds & Insurance (5% of Subtotal)			$720,000
Contingency (20%)			$3,000,000
Total Probable Construction Cost			***$18,100,000***
Engineering, Administrative, and Legal (20%)			$3,500,000
Total Probable Project Cost (Rounded)			***$21,600,000***

(1) Year 2012 Dollars
(2) Includes Valves and Meters

9.0 REGULATORY CONSIDERATIONS

A critical component to evaluating alternatives for upgrading the WPCP to accept and treat additional flows will be the discharge limits the NYSDEC will impose on the District. To date, these limits are not defined, but future more stringent discharge limits for nitrogen and phosphorus have been indicated by the NYSDEC. The District has expressed their concerns that the potential limits could present a significant financial impact in the cover letter of this report.

To date, a total maximum daily load (TMDL) study of the segment of the Mohawk River at the WPCP has not been conducted by the NYSDEC. Until the NYSDEC completes this study, the future regulations may not be fully defined.

Depending on the degree to which nitrogen and phosphorus must be removed, the WPCP may have to employ costly cutting-edge technologies to remove these nutrients as the peak flow to the plant more than doubles. After a project update meeting with the NYSDEC in May 2012, the NYSDEC provided a written comment stating:

> *"The Department is currently developing new water quality criteria for nutrients which are expected to be finalized within the next few years and impact the SPDES permit. While these requirements are not yet known a preliminary assessment suggests that phosphorus reduction will likely be necessary and that total nitrogen reduction will likely not be necessary."*

Since the future regulations are not fully defined at this time, this Report can only be based upon existing regulations as summarized in Section 3.7.

The project is already a substantial financial burden on the rate payers in the District. If the NYSDEC imposes enhanced nitrogen or phosphorus removal limits, the costs will increase dramatically. Further, the possibility exists that several components of the pump station and

WPCP upgrade could be constructed to meet the Consent Order but prior to the final determination of nitrogen and phosphorus limits. In this case, the District would have to spend additional funds to upgrade recently constructed facilities that can no longer meet the more stringent permit requirements.

The District has requested an accelerated determination of discharge permit requirements. Given the magnitude of costs, the District will be required to obtain funding. Knowing the permit limits as soon as possible would assist in obtaining the proper funding for the project.

10.0 CONCLUSIONS

The existing WPCP operates within its SPDES permit for flows up to 55 mgd. Additional flows and loads will be conveyed to the WPCP. The new flows and loads will be the result of:

- CSO mitigation in the City of Utica
- SSO mitigation in the SCPS basin
- Industrial growth and associated spin-off

The existing WPCP is hydraulically limited at flows greater than 55 mgd due to restrictions between the primary settling tanks and aeration basins. The process and hydraulic capacity of the secondary treatment system is approximately 65 mgd.

Several alternatives were evaluated to expand the WPCP, including:

- Split flow during wet weather
- Alternate modes of aeration
- IFAS
- High rate ballasted flocculation

Several alternatives were evaluated for expanding the solids handling and disposal facilities, including:

- Gravity thickening
- Belt filter press dewatering
- Centrifuge dewatering
- Continued use of incineration
- Anaerobic digestion
- Lime stabilization
- Incineration with lime stabilization backup

The split flow alternative represents the most cost effective approach to expanding the WPCP and should be the basis for WPCP expansion. Under this alternative, all combined flows from the City of Utica receive primary settling and disinfection during wet weather and secondary treatment during dry weather. All sanitary flows receive secondary treatment at all times. The capacity of the existing secondary treatment facility is maximized during wet weather. In addition, preliminary treatment would be upgraded to provide screening and grit removal of all flows.

The existing primary settling tanks have significant structural issues, and could be replaced with rectangular settling tanks in conjunction with the split flow alternative. Conventional primary settling tanks are more cost effective than CEPT for ballasted flocculation.

The process capacity of the aeration basins could potentially be expanded by implementing the step-feed or contact stabilization modes of operation.

Placing the fourth thickener back into service would provide adequate thickening capacity for future flows and loads. Replacing belt filter presses with centrifuges would provide a drier sludge cake and reduce the fuel consumption at the incinerators.

Rehabilitating Incinerator Nos. 1 and 3, with a backup lime stabilization system to replace Incinerator No. 2, is the most cost effective sludge disposal alternative. Converting the incinerator fuel source from fuel oil to natural gas would decrease the annual fuel cost.

The physical condition of the WPCP requires significant upgrades to sustain operations into the future. Existing electrical facilities (switchgear, power distribution, and emergency generator) will need to be expanded.

11.0 RECOMMENDED ALTERNATIVE

Based on a comprehensive evaluation of several alternatives to expand the WPCP to accept and treat additional flows and loads, the split flow concept is the most cost effective solution for upgrading the WPCP. The District should proceed with the construction of split flow facilities, including the construction of several major new facilities at the WPCP:

- New sanitary screen facility and pump station
- New grit removal
- New split flow distribution structure
- New conventional primary settling tanks
- New high rate disinfection

In addition to increasing the capacity of the WPCP through the construction of new facilities, WPCP operators should experiment with alternate modes of operation, including step-feed and contact stabilization. These operational adjustments can be implemented with minimal modifications to the existing basins, and may improve the capacity of the basins.

Hydraulic bottlenecks at the WPCP should be alleviated by constructing new aeration basin influent pumping and weir modifications throughout the WPCP.

The SCPS should be upgraded to provide increased capacity. The capacity can be increased by installing a new, larger diameter, forcemain to the WPCP. The forcemain should be installed via a combination of open cut, directional drilling, and shallow-bury/trenching.

To maintain the long-term viability of the WPCP, the District must address the physical condition of the plant based on the immediate and short term needs identified in the previously prepared physical condition assessment. In addition, the the WPCPs existing electrical facilities must be upgraded and expanded to support future electrical demands. .

11.1 COST OF RECOMMENDED ALTERNATIVE

The probable project cost for the entire upgrade, including facilities at the WPCP and SCPS, is presented in Table 11-1.

TABLE 11-1

ENGINEERS OPINION OF PROBABLE COST:

WPCP EXPANSION AND SCPS UPGRADE

RECOMMENDED PROJECT COMPONENT	PROBABLE PROJECT COST (1)
New Screens at SCPS and Forcemain to WPCP	$21,600,000
New Sanitary Bar Screen Facility	$7,600,000
New Sanitary Pump Station	$7,100,000
New Grit Removal	$3,800,000
New Primary Settling Tanks	$22,300,000
New Split Flow Distribution Box	$1,000,000
New High Rate Disinfection Facilities	$3,000,000
Raise WPCP Weirs	$250,000
New Aeration Basin Influent Piping	$500,000
Solids Handling & Incinerators	$26,200,000
Physical Condition Upgrades at WPCP	$33,900,000
Electrical Switchgear, Distribution, and Generator Modifications	$8,500,000
Re-route Existing Interceptors and New Yard Piping	$1,500,000
Construction Staging (Flow Diversions and Bypass Pumping)	$1,000,000
Total Project Cost (Rounded)	**$138,000,000**

(1) Year 2012 Dollars

12.0 IMPLEMENTATION SCHEDULE

12.1 PROJECT IMPLEMENTATION SCHEDULE

The Consent Order requires construction of the expanded WPCP facilities and SCPS to be completed by December 31, 2021. A project of this magnitude could take several years to design, and physical construction could also be expected to take several years.

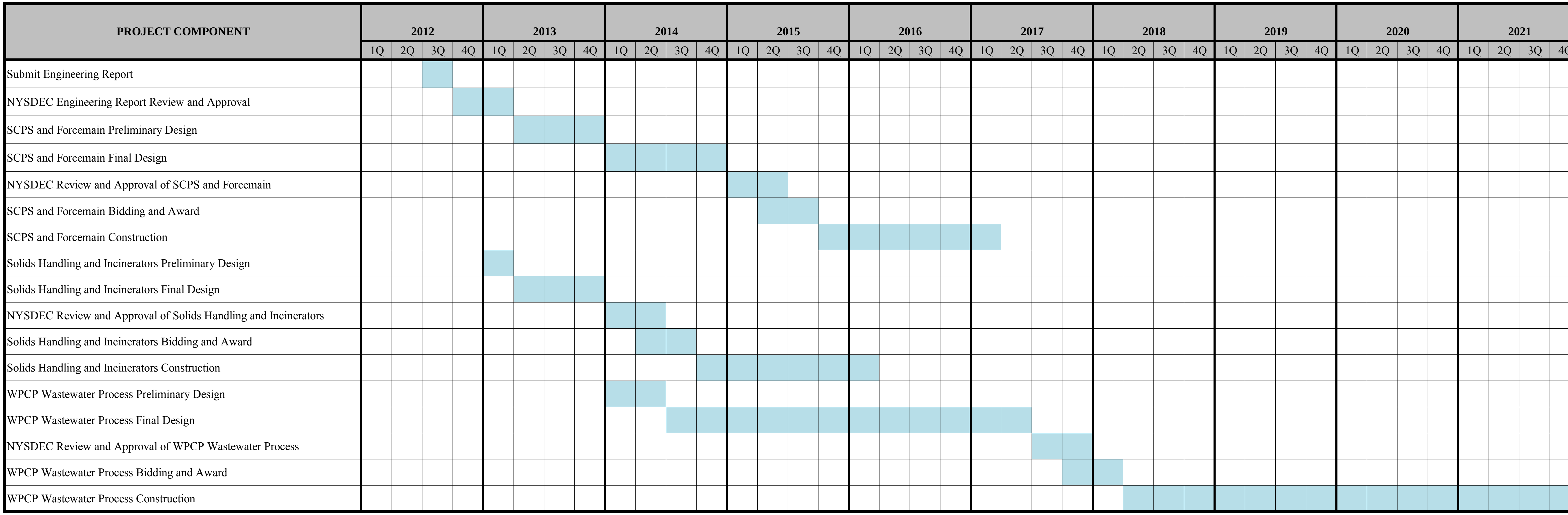
The project would likely be phased into a series of contracts. The work associated with the SCPS upgrades and new discharge forcemain could be constructed on a separate track from improvements at the WPCP. At the WPCP, the solids handling and incineration upgrades would need to be constructed by March 2016 according to incinerator emissions guidelines recently enacted by the NYSDEC[2]. The wastewater treatment upgrades could be constructed following the upgrades to the solids handling and incineration facilities.

An implementation schedule based on the above constraints is provided on Figure 12-1.

[2] 6 NYCRR Subpart 219-9: Emission Guidelines and Compliance Schedules for Existing Sewage Sludge Incineration Units, Effective May 12, 2012.

FIGURE12-1

IMPLEMENTATION SCHEDULE



APPENDIX A

AERATION SYSTEM MODEL CALIBRATION FILES

GHD	Oneida County WPCP Report	6/27/12	8614782
Clients	Project	Date	Job No.
People	Plant Flow and loads analysis	SES	
Performance	Subject	Comp. By	Checked By

For Reference Only: Metcalf & Eddy Wastewater Characteristics

Wastewater strength	Flow	BOD (mg/L)	TSS (mg/L)	NH3 (mg/L)	TKN (mg/L)	TP (mg/L)	
Medium	-	190	210	25	40	7	
Maximum Month to Average Day Peaking Factors	1.20	1.30	1.30	1.25	1.25	1.25	WEF MOP-8 Table 3-4; M&E Figure 3-8

Table 1: Current WPCP Flows

Condition	Influent Flow (mgd)
Average Month	42.00
Maximum Month	54.00
Peak Hour	55.00

Note:
1. From Table 3.1 in the July 2012 draft report

Table 2: Current WPCP Loads

	Flow (MGD)	BOD (lb/d)	TSS (lb/d)	NH4 (lb/d)	TKN (lb/d)	TP (lbs/d)
Average Month	42.00	31,200	21,600	1,500	3,900	1,944
Maximum Month	54.00	49,900	31,700	4,500	5,000	2,853

Notes:
1. TP load is estimated based on an assumed an TP to TSS ratio of 0.09 based on the existing calibration period
2. Loads are per Table 3-1 in the Water Pollution Control Plant Report Draft dated July 2012

Table 3: Current WPCP Concentrations

	Flow (MGD)	BOD (mg/L)	TSS (mg/L)	NH4 (mg/L)	TKN (mg/L)	TP (mg/L)
Average Month	42.00	89	62	4.3	11.1	5.55
Maximum Month	54.00	111	70	10.0	11.1	6.33

Notes:
1. Concentrations = Load/8.34/ Flow

Table 4: Design Dry Weather WPCP Flows

Condition	Sanitary Flow Peaking Factor	Sanitary Flow (mgd)	Chip Plant MFG Flow (mgd)	Chip Plant Residential Flow (mgd)	Total Flow (mgd)
Average Month	1.00	42.00	6	3	51.00
Maximum Month	1.20	50.40	6	3.6	60.00

Note:
1. Design Average sanitary flow is equal to the current sanitary flow
2. Design maximum month sanitary flow is equal to the design average sanitary flow Times a peaking factor of 1.2 (WEF MOP-8)
3. Chip plant flow is set at a constant flow of 6.0 mgd (see 7-17-2012 email Story to Schwetschenau)
4. Chip Plant average day residential flow is equal to 3.0 mgd. (see 7-17-2012 email Story to Schwetschenau)
5. Chip Plant maximum month residential flow is equal to the average day flow times a peaking factor of 1.2
6. Total dry weather plant flow is the sum of the sanitary flow, the chip plant mfg flow and the chip plant residential flow

Table 5: Design Wet Weather WPCP Flows

Total Allowable Flow to the Plant, mgd	111
Max Flow to Biological Treatment Train, mgd	65

Condition	Total Dry Weather Flow (mgd)	CSO Flow (mgd)	Total Flow to Plant (mgd)	Flow to Biological System (mgd)	CSO Flow to Biological System (mgd)	CSO Flow to Wet Weather Treatment System (mgd)
Average Month	51.00	60.00	111	65	14.00	46.00
Maximum Month	60.00	51.00	111	65	5.00	46.00

Notes:
1. The peak flow to the facility is capped at 111 mgd.
2. CSO flow = Total Plant Flow - Total Dry Weather Flow
3. Flow to the biological system is capped at 65 mgd by the final clarifiers
4. CSO flow is diverted from the wet weather treatment system to the biological treatemtn system up to 65 mgd
   CSO Flow to biological system = Flow to biological system - Total Dry Weather Flow
5. CSO To wet weather flow treatment = CSO Flow - CSO Flow to Biological System

Table 6: Design Sanitary Sewer WPCP Loads

	Flow (MGD)	BOD (lb/d)	TSS (lb/d)	NH4 (lb/d)	TKN (lb/d)	TP (lbs/d)
Average Month	42.00	31,200	21,600	1,500	3,900	1,944
Maximum Month	50.40	40,560	28,080	1,875	4,875	2,430

Notes:
1. Average sanitary sewer WPCP concentrations are the same as current and summarized in Table 3-1 from the Draft July 2012 report

Table 7: Chip Fab Plant Loads

BOD Concentration, mg/L	0	process is inorganic
TSS Concentration, mg/L	50	
NH3, mg/L	20	

	Flow (MGD)	BOD (lb/d)	TSS (lb/d)	NH4 (lb/d)	TKN (lb/d)	TP (lbs/d)
Average Month	6.00	0	2,502	1,001	1,001	0
Maximum Month	6.00	0	2,502	1,001	1,001	0

Notes:

1. Assume the TP concentration is 0 since it was not specified
2. Since the process is organice assume the organic nitrogen concentration is 0. TKN = NH3
3. There is assumed to be no flucctuation is flows and loads from the chip plant average and MM are the same load

Table 8: Chip Fab Plant Residential Loads

	Flow (MGD)	BOD (lb/d)	TSS (lb/d)	NH4 (lb/d)	TKN (lb/d)	TP (lbs/d)
Average Month	3	2,229	1,543	107	279	139
Maximum Month	3.6	2,897	2,006	134	348	174

Note:

1. Residential component of the chip fab plant is assumed to have the same concentrations as the existing influent.
   See Table 3 for concentrations used

Table 9: Design Dry Weather WPCP Loads

	Flow (MGD)	BOD (lb/d)	TSS (lb/d)	NH4 (lb/d)	TKN (lb/d)	TP (lbs/d)
Average Month	51.00	33,429	25,645	2,608	5,179	2,083
Maximum Month	60.00	43,457	32,588	3,010	6,224	2,604

Notes:

1. Loads are the sum of the Sanitary sewer load + Chip Plant Load + Chip Plant Residential Load

Table 10: Design Dry Weather WPCP Concentrations

	Flow (MGD)	BOD (mg/L)	TSS (mg/L)	NH4 (mg/L)	TKN (mg/L)	TP (mg/L)
Average Month	51.00	78.6	60.3	6.1	12.2	4.9
Maximum Month	60.00	86.8	65.1	6.0	12.4	5.2

Use these in the model for the Sanitary flow input

Notes:

1. Concentrations = Load/8.34/ Flow
2. Assume a VSS to TSS ratio of 0.75

Table 11: Design CSO Concentrations

	Flow (MGD)	BOD (mg/L)	TSS (mg/L)	NH4 (mg/L)	TKN (mg/L)	TP (mg/L)
Average Month	60.00	60	50	4	8	2
Maximum Month	51.00	60	50	4	8	2

Note:
1. Concentrations are per published resources and are the same for Maximum month and average conditions
2. Assume a VSS to TSS ratio of 0.8
3. Assume an NH4 to TKN ratio of 0.505

Use these in the model for the CSO flow input

Table 12: Design CSO Loads

	Flow (MGD)	BOD (lb/d)	TSS (lb/d)	NH4 (lb/d)	TKN (lb/d)	TP (lbs/d)
Average Month	60.00	30,024	25,020	2,022	4,003	1,001
Maximum Month	51.00	25,520	21,267	1,718	3,403	851

Notes:
1. Loads = concentration * 8.34* Flow

BioWin Models

Steady State Results for Design Models

Condition 1: Average - Existing Solids Train	Temp	SRT
Winter	8.8	8.4
Sanitary Influent Flow	51	mgd
CSO Flow	60	
Model Influent Flow	111	

	Sewer Influent	CSO Influent	
BOD:TKN Ratio	6.46	-	
AOB Growth Rate Coefficient	0.76	-	default is 0.9
RAS Flow, mgd	15.30	-	30% of influent
Fbs	0.22	0.16	
Fxsp	0.60	0.03	
Fus	0.14	0.05	
Fup	0.10	0.40	
P.Clarifier Removal	47.00%	47.00%	default is 60%
OP/TP Fraction	0.50	0.50	
NH4/TKN Fraction	0.50	0.51	

Model Notes:

Model Name: Design CSO bypass Avg Dry Existing Solids.bwc

Elements	Total COD [mg/L]	Total Carbonaceous BOD [mg/L]	Total Kjeldahl Nitrogen [mgN/L]	Total N [mgN/L]	Total suspended solids [mgTSS/L]	Volatile suspended solids [mgVSS/L]	Ammonia N [mgN/L]	Nitrate N [mgN/L]
Influent	164.89	78.61	12.2	12.8	60.31	45.2	6.1	0.6
Sanitary Sewer Primaries	129.69	62.87	9.73	10.35	38.25	24.6	5.53	0.62
1A	2772.86	837.34	163.93	165.85	2,740	1,847	3.94	1.61
1B	2761.35	829.59	163.05	165.77	2,738	1,845	3	2.29
1C	2750.84	822.69	162.23	165.69	2,735	1,841	2.16	2.99
1D	2741.39	816.47	161.49	165.62	2,731	1,837	1.5	3.68
Effluent	49.57	12.4	4.23	8.36	21.94	14.76	1.5	3.68
Stormwater	125.49	59.54	6.61	7.21	26.72	21.37	4.04	0.6

Solids Train Results

Elements	Total suspended solids [mgTSS/L]	Total suspended solids [lb TSS/d]	Volatile suspended solids [mgVSS/L]	Volatile suspended solids [lb VSS/d]	Flow [mgd]	Flow [lb /d]
Influent	60.31	25,671	45.2	19,238	51	-----
Sanitary Sewer Primaries	38.25	21,353	24.6	13,734	67	-----
Sanitary Sewer Primaries (U)	4,493	18,936	2,890	12,179	1	-----
Stormwater Primaries (U)	3011.95	9,024	2408.91	7,217	0.36	-----
Stormwater Primaries	27	10,176	21	8,138	46	-----
Secoondary Clarifier	22	12,137	15	8,164	66	-----
Secoondary Clarifier (U)	14,467	1,847,167	9,732	1,242,572	15	-----
WAS RAS Splitter	2,731	1,859,303	1,837	1,250,736	82	-----
WAS RAS Splitter (U)	2,731	14,298	1,837	9,618	1	-----
Belt Filter Press	1,333	803	915	551	0	-----
Belt Filter Press (U)	206,763	39,342	141,966	27,013	0	-----
Gravity Thickner	181.3	2,113	124.5	1,451	1.4	-----
Gravity Thickner (U)	50,636	40,145	34,767	27,564	0	-----
Sludge	206,763	39,342	141,966	27,013	0	-----

Condition 2: Max Month - Existing Solids Train	Temp	SRT
Winter	8.8	8.65
Sanitary Influent Flow	60	mgd
CSO Flow	51	
Model Influent Flow	111	

	Sewer Influent	CSO Influent	
BOD:TKN Ratio	6.74	-	
AOB Growth Rate Coefficient	0.76	-	default is 0.9
RAS Flow, mgd	18.00	-	30% of influent
Fbs	0.22	0.16	
Fxsp	0.60	0.03	
Fus	0.14	0.05	
Fup	0.10	0.40	
P.Clarifier Removal	47.00%	47.00%	default is 60%
OP/TP Fraction	0.50	0.50	
NH4/TKN Fraction	0.49	0.51	

Model Notes:

Load to Gravity Thickener	50,602

Model Name: Design CSO bypass MM Dry Existing Solids.bwc

Elements	Total COD [mg/L]	Total Carbonaceous BOD [mg/L]	Total Kjeldahl Nitrogen [mgN/L]	Total N [mgN/L]	Total suspended solids [mgTSS/L]	Volatile suspended solids [mgVSS/L]	Ammonia N [mgN/L]	Nitrate N [mgN/L]
Influent	181.53	86.81	12.4	13	65.12	48.8	6.08	0.6
Sanitary Sewer Primaries	143.16	70.07	10.4	11.04	41.68	26.96	5.77	0.64
1A	2967.05	968.44	181.39	183.63	2,989	1,990	3.94	1.93
1B	2954.2	959.75	180.39	183.54	2,986	1,987	2.91	2.74
1C	2942.63	952.12	179.47	183.45	2,982	1,983	1.99	3.55
1D	2932.31	945.36	178.66	183.38	2,978	1,978	1.31	4.35
Effluent	44.04	9.59	3.54	8.26	12.95	8.6	1.31	4.35
Stormwater	125.49	59.54	6.61	7.21	26.72	21.37	4.04	0.6

Solids Train Results

Elements	Total suspended solids [mgTSS/L]	Total suspended solids [lb TSS/d]	Volatile suspended solids [mgVSS/L]	Volatile suspended solids [lb VSS/d]	Flow [mgd]	Flow [lb /d]
Influent	65.12	32,605	48.8	24,435	60	-----
Sanitary Sewer Primaries	41.68	23,342	26.96	15,096	67	-----
Sanitary Sewer Primaries (U)	4,912	20,700	3,176	13,387	1	-----
Secoondary Clarifier	13	7,160	9	4,756	66	-----
Secoondary Clarifier (U)	13,895	2,087,247	9,230	1,386,470	18	-----
WAS RAS Splitter	2978.26	2,094,406	1978.33	1,391,226	84.27	-----
WAS RAS Splitter (U)	2,978	20,878	1,978	13,868	1	-----
Belt Filter Press	1,345	961	916	655	0	-----
Belt Filter Press (U)	192,499	47,110	131,141	32,094	0	-----
Gravity Thickner	191	2,530	130	1,724	2	-----
Gravity Thickner (U)	50,089	48,071	34,123	32,749	0	-----
Sludge	192,498.8	47,110	131,140.7	32,094	0.0	-----
Stormwater Primaries	27	10,176	21	8,138	46	-----
Stormwater Primaries (U)	3,012	9,024	2,409	7,217	0	-----

Condition 5: Average - Lime Stab Solids Train	Temp	SRT
Winter	8.8	10
Sanitary Influent Flow	51	mgd
CSO Flow	60	
Model Influent Flow	111	

	Sewer Influent	CSO Influent	
BOD:TKN Ratio	6.47	-	
AOB Growth Rate Coefficient	0.76	-	default is 0.9
RAS Flow, mgd	15.30	-	30% of influent
Fbs	0.22	0.16	
Fxsp	0.60	0.03	
Fus	0.14	0.05	
Fup	0.10	0.40	
P.Clarifier Removal	47.00%	47.00%	default is 60%
OP/TP Fraction	0.50	0.50	
NH4/TKN Fraction	0.50	0.51	

Model Notes:

Model Name: Design CSO bypass Avg Dry Existing Solids no incin.bwc

Elements	Total COD [mg/L]	Total Carbonaceous BOD [mg/L]	Total Kjeldahl Nitrogen [mgN/L]	Total N [mgN/L]	Total suspended solids [mgTSS/L]	Volatile suspended solids [mgVSS/L]	Ammonia N [mgN/L]	Nitrate N [mgN/L]
Influent	164.89	78.61	12.2	12.8	60.31	45.2	6.1	0.6
Sanitary Sewer Primaries	131.63	63.85	9.87	10.5	32.79	24.91	5.61	0.63
1A	3178.38	910.5	186.45	188.77	2,773	2,121	3.64	2.03
1B	3166.1	902.19	185.35	188.68	2,770	2,118	2.48	2.97
1C	3155.08	894.93	184.37	188.6	2,767	2,114	1.53	3.9
1D	3145.24	888.48	183.58	188.53	2,762	2,109	0.88	4.73
Effluent	51.62	12.1	3.75	8.69	21.84	16.68	0.88	4.73
Stormwater	125.49	59.54	6.61	7.21	26.72	21.37	4.04	0.6

Solids Train Results

Elements	Total suspended solids [mgTSS/L]	Total suspended solids [lb TSS/d]	Volatile suspended solids [mgVSS/L]	Volatile suspended solids [lb VSS/d]	Flow [mgd]	Flow [lb /d]
Influent	60.31	25,671	45.2	19,238	51	-----
Sanitary Sewer Primaries	32.79	18,003	24.91	13,676	66	-----
Sanitary Sewer Primaries (U)	3,788	15,965	2,878	12,128	1	-----
Secoondary Clarifier	21.84	11,909	16.68	9,093	65.34	-----
Secoondary Clarifier (U)	14,466	1,847,144	11,046	1,410,438	15	-----
WAS RAS Splitter	2,762	1,859,052	2,109	1,419,531	81	-----
WAS RAS Splitter (U)	2,762	10,582	2,109	8,080	0	-----
Belt Filter Press	1,403	676	1,081	521	0	-----
Belt Filter Press (U)	200,795	33,116	154,812	25,532	0	-----
Gravity Thickner	171	1,779	132	1,371	1	-----
Gravity Thickner (U)	52,248	33,792	40,283	26,054	0	-----
Sludge	200,795.1	33,116	154,811.6	25,532	0.0	-----
Stormwater Primaries	27	10,176	21	8,138	46	-----
Stormwater Primaries (U)	3,012	9,024	2,409	7,217	0	-----

Condition 6: Max Month - Lime Stab Solids Train	Temp	SRT
Winter	8.8	10.5
Sanitary Influent Flow	60	mgd
CSO Flow	51	
Model Influent Flow	111	

	Sewer Influent	CSO Influent	
BOD:TKN Ratio	6.74	-	
AOB Growth Rate Coefficient	0.76	-	default is 0.9
RAS Flow, mgd	18.00	-	30% of influent
Fbs	0.22	0.16	
Fxsp	0.60	0.03	
Fus	0.14	0.05	
Fup	0.10	0.40	
P.Clarifier Removal	47.00%	47.00%	default is 60%
OP/TP Fraction	0.50	0.50	
NH4/TKN Fraction	0.49	0.51	

Model Notes:

Load to Gravity Thickener 43,626

Model Name: Design CSO bypass MM Dry Existing Solids no incin.bwc

Elements	Total COD [mg/L]	Total Carbonaceous BOD [mg/L]	Total Kjeldahl Nitrogen [mgN/L]	Total N [mgN/L]	Total suspended solids [mgTSS/L]	Volatile suspended solids [mgVSS/L]	Ammonia N [mgN/L]	Nitrate N [mgN/L]
Influent	181.53	86.81	12.4	13	65.12	48.8	6.08	0.6
Sanitary Sewer Primaries	145.36	71.17	10.56	11.2	36.28	27.32	5.86	0.65
1A	3427.03	1054.98	207.94	210.6	3,077	2,302	3.63	2.37
1B	3413.34	1045.68	206.71	210.5	3,074	2,299	2.39	3.44
1C	3401.22	1037.7	205.66	210.41	3,070	2,294	1.39	4.47
1D	3390.46	1030.65	204.83	210.34	3,065	2,289	0.76	5.33
Effluent	44.79	9.09	3.05	8.56	12.79	9.55	0.76	5.33
Stormwater	125.49	59.54	6.61	7.21	26.72	21.37	4.04	0.6

Solids Train Results

Elements	Total suspended solids [mgTSS/L]	Total suspended solids [lb TSS/d]	Volatile suspended solids [mgVSS/L]	Volatile suspended solids [lb VSS/d]	Flow [mgd]	Flow [lb /d]
Influent	65.12	32,605	48.8	24,435	60	-----
Sanitary Sewer Primaries	36.28	19,982	27.32	15,048	66	-----
Sanitary Sewer Primaries (U)	4204.6	17,720	3166.45	13,345	1	-----
Secoondary Clarifier	12.79	6,973	9.55	5,207	65	-----
Secoondary Clarifier (U)	14,143	2,124,591	10,563	1,586,688	18	-----
WAS RAS Splitter	3065.01	2,131,564	2289.01	1,591,896	83.33	-----
WAS RAS Splitter (U)	3,065	16,882	2,289	12,608	1	-----
Belt Filter Press	1,333	829	1,014	630	0	-----
Belt Filter Press (U)	190,856	40,616	145,112	30,881	0	-----
Gravity Thickner	184	2,181	140	1,658	1	-----
Gravity Thickner (U)	49,661	41,444	37,759	31,511	0	-----
Sludge	190,855.7	40,616	145,112.0	30,881	0.0	-----
Stormwater Primaries	27	10,176	21	8,138	46	-----
Stormwater Primaries (U)	3,012	9,024	2,409	7,217	0	-----

Condition 7: Average - Digester Solids Train	Temp	SRT
Winter	8.8	10
Sanitary Influent Flow	51	mgd
CSO Flow	60	
Model Influent Flow	111	

	Sewer Influent	CSO Influent	
BOD:TKN Ratio	5.84	-	
AOB Growth Rate Coefficient	0.76	-	default is 0.9
RAS Flow, mgd	15.30	-	30% of influent
Fbs	0.22	0.16	
Fxsp	0.60	0.03	
Fus	0.14	0.05	
Fup	0.10	0.40	
P.Clarifier Removal	47.00%	47.00%	default is 60%
OP/TP Fraction	0.50	0.50	
NH4/TKN Fraction	0.50	0.51	

Model Notes:

Model Name:

Elements	Total COD [mg/L]	Total Carbonaceous BOD [mg/L]	Total Kjeldahl Nitrogen [mgN/L]	Total N [mgN/L]	Total suspended solids [mgTSS/L]	Volatile suspended solids [mgVSS/L]	Ammonia N [mgN/L]	Nitrate N [mgN/L]
Influent	164.89	78.61	12.2	12.8	60.34	45.2	6.1	0.6
Sanitary Sewer Primaries	131.18	63.68	10.91	11.55	32.48	24.62	6.68	0.63
1A	3176.18	916.49	187.18	189.99	2,772	2,120	4.21	2.48
1B	3163.99	908.25	185.79	189.9	2,770	2,117	2.76	3.71
1C	3153.05	901.05	184.58	189.82	2,766	2,113	1.57	4.9
1D	3143.27	894.62	183.66	189.75	2,762	2,109	0.78	5.9
Effluent	51.55	12.14	3.65	9.74	21.78	16.62	0.78	5.9
Stormwater	125.55	59.54	6.61	7.21	26.77	21.41	4.04	0.6

Solids Train Results

Elements	Total suspended solids [mgTSS/L]	Total suspended solids [lb TSS/d]	Volatile suspended solids [mgVSS/L]	Volatile suspended solids [lb VSS/d]	Flow [mgd]	Flow [lb /d]
Influent	60.34	25,683	45.2	19,238	51	-----
Sanitary Sewer Primaries	32.48	17,839	24.62	13,522	66	-----
Sanitary Sewer Primaries (U)	3,754	15,820	2,845	11,991	1	-----
Secoondary Clarifier	21.78	11,876	16.62	9,066	65.35	-----
Secoondary Clarifier (U)	14466.46	1,847,146	11043.88	1,410,135	15.3	-----
WAS RAS Splitter	2762.16	1,859,022	2108.67	1,419,201	80.65	-----
WAS RAS Splitter (U)	2762.16	10,581	2108.67	8,077	0.46	-----
Belt Filter Press	856.93	450	585.81	307	0.06	-----
Belt Filter Press (U)	212,494	22,033	145,263	15,062	0	-----
Gravity Thickner	162	1,683	124	1,294	1	-----
Gravity Thickner (U)	51,087	31,976	39,270	24,580	0	-----
Sludge	212,494	22,033	145,263	15,062	0	-----
Primary Digester	37,033	23,272	25,674	16,134	0	-----
Secondary Digester	35,777	22,483	24,457	15,369	0	-----
Stormwater Primaries	27	8,185	21	6,546	37	-----
Stormwater Primaries (U)	2,423	7,258	1,938	5,805	0	-----

Condition 8: Maximum Month - Digester Solids Train	Temp	SRT
Winter	8.8	10.3
Sanitary Influent Flow	60	mgd
CSO Flow	51	
Model Influent Flow	111	

	Sewer Influent	CSO Influent	
BOD:TKN Ratio	7.00	-	
AOB Growth Rate Coefficient	0.76	-	default is 0.9
RAS Flow, mgd	18.00	-	30% of influent
Fbs	0.22	0.16	
Fxsp	0.60	0.03	
Fus	0.14	0.05	
Fup	0.10	0.40	
P.Clarifier Removal	47.00%	47.00%	default is 60%
OP/TP Fraction	0.50	0.50	
NH4/TKN Fraction	0.49	0.51	

Model Notes:

Load to Gravity Thickener	43,634

Model Name:

Elements	Total COD [mg/L]	Total Carbonaceous BOD [mg/L]	Total Kjeldahl Nitrogen [mgN/L]	Total N [mgN/L]	Total suspended solids [mgTSS/L]	Volatile suspended solids [mgVSS/L]	Ammonia N [mgN/L]	Nitrate N [mgN/L]
Influent	181.53	86.81	12.4	13	65.12	48.8	6.08	0.6
Sanitary Sewer Primaries	144.97	70.94	11.88	12.54	36.03	27.08	7.2	0.66
1A	3394.62	1055.21	206.85	210.13	3,045	2,280	4.35	2.94
1B	3381.09	1046.04	205.28	210.03	3,042	2,277	2.75	4.34
1C	3369.1	1038.13	203.95	209.95	3,038	2,272	1.47	5.68
1D	3358.42	1031.14	202.98	209.87	3,033	2,267	0.68	6.74
Effluent	44.85	9.18	2.98	9.87	12.75	9.53	0.68	6.74
Stormwater	125.49	59.54	6.61	7.21	26.72	21.37	4.04	0.6

Solids Train Results

Elements	Total suspended solids [mgTSS/L]	Total suspended solids [lb TSS/d]	Volatile suspended solids [mgVSS/L]	Volatile suspended solids [lb VSS/d]	Flow [mgd]	Flow [lb /d]
Influent	65.12	32,605	48.8	24,435	60	-----
Sanitary Sewer Primaries	36.03	19,848	27.08	14,918	66	-----
Sanitary Sewer Primaries (U)	4,176	17,601	3,139	13,229	1	-----
Secoondary Clarifier	12.75	6,954	9.53	5,198	65.34	-----
Secoondary Clarifier (U)	13,997	2,102,587	10,464	1,571,893	18	-----
WAS RAS Splitter	3,033	2,109,540	2,267	1,577,091	83	-----
WAS RAS Splitter (U)	3,033	17,009	2,267	12,716	1	-----
Belt Filter Press	821	574	550	385	0	-----
Belt Filter Press (U)	203,516	28,150	136,383	18,864	0	-----
Gravity Thickner	182	2,182	138	1,658	1	-----
Gravity Thickner (U)	49,671	41,452	37,751	31,505	0	-----
Sludge	203,515.6	28,150	136,382.5	18,864	0.0	-----
Primary Digester	35,643	29,879	24,286	20,359	0	-----
Secondary Digester	34,265	28,725	22,962	19,249	0	-----
Stormwater Primaries	27	10,176	21	8,138	46	-----
Stormwater Primaries (U)	3,012	9,024	2,409	7,217	0	-----

BioWin Models
Steady State Results for Design Models

Condition 1: Average - Existing Solids Train	Temp	SRT
Winter	8.8	8.4
Sanitary Influent Flow	51	mgd
CSO Flow	60	
Model Influent Flow	111	

	Sewer Influent	CSO Influent	
BOD:TKN Ratio	6.46	-	
AOB Growth Rate Coefficient	0.76	-	default is 0.9
RAS Flow, mgd	15.30	-	30% of influent
Fbs	0.22	0.16	
Fxsp	0.60	0.03	
Fus	0.14	0.05	
Fup	0.10	0.40	
P.Clarifier Removal	47.00%	47.00%	default is 60%
OP/TP Fraction	0.50	0.50	
NH4/TKN Fraction	0.50	0.51	

Model Notes:

Model Name: Design CSO bypass Avg Dry Existing Solids.bwc

Elements	Total COD [mg/L]	Total Carbonaceous BOD [mg/L]	Total Kjeldahl Nitrogen [mgN/L]	Total N [mgN/L]	Total suspended solids [mgTSS/L]	Volatile suspended solids [mgVSS/L]	Ammonia N [mgN/L]	Nitrate N [mgN/L]
Influent	164.89	78.61	12.2	12.8	60.31	45.2	6.1	0.6
Sanitary Sewer Primaries	129.69	62.87	9.73	10.35	38.24	24.6	5.53	0.62
1A	2772.7	837.33	163.93	165.84	2,740	1,847	3.94	1.61
1B	2761.19	829.58	163.05	165.76	2,738	1,845	3	2.29
1C	2750.69	822.68	162.22	165.68	2,735	1,841	2.16	2.99
1D	2741.24	816.46	161.48	165.61	2,731	1,837	1.5	3.68
Effluent	49.58	12.4	4.23	8.36	21.95	14.76	1.5	3.68
Stormwater	125.49	59.54	6.61	7.21	26.72	21.37	4.04	0.6

Solids Train Results

Elements	Total suspended solids [mgTSS/L]	Total suspended solids [lb TSS/d]	Volatile suspended solids [mgVSS/L]	Volatile suspended solids [lb VSS/d]	Flow [mgd]	Flow [lb /d]
Influent	60.31	25,671	45.2	19,238	51	-----
Sanitary Sewer Primaries	38.24	21,353	24.6	13,734	67	-----
Sanitary Sewer Primaries (U)	4,493	18,935	2,890	12,179	1	-----
Stormwater Primaries (U)	3011.95	9,024	2408.91	7,217	0.36	-----
Stormwater Primaries	27	10,176	21	8,138	46	-----
Secoondary Clarifier	22	12,138	15	8,165	66	-----
Secoondary Clarifier (U)	14,467	1,847,167	9,732	1,242,574	15	-----
WAS RAS Splitter	2,731	1,859,305	1,837	1,250,740	82	-----
WAS RAS Splitter (U)	2,731	14,297	1,837	9,618	1	-----
Belt Filter Press	1,250	803	858	551	0	-----
Belt Filter Press (U)	261,169	39,341	179,322	27,012	0	-----
Gravity Thickner	181.3	2,113	124.5	1,451	1.4	-----
Gravity Thickner (U)	50,635	40,144	34,767	27,563	0	-----
Sludge	261,169	39,341	179,322	27,012	0	-----

Condition 2: Maximum Month - Existing Solids Train	Temp	SRT
Winter	8.8	8.65
Sanitary Influent Flow	60	mgd
CSO Flow	51	
Model Influent Flow	111	

	Sewer Influent	CSO Influent	
BOD:TKN Ratio	6.74	-	
AOB Growth Rate Coefficient	0.76	-	default is 0.9
RAS Flow, mgd	18.00	-	30% of influent
Fbs	0.22	0.16	
Fxsp	0.60	0.03	
Fus	0.14	0.05	
Fup	0.10	0.40	
P.Clarifier Removal	47.00%	47.00%	default is 60%
OP/TP Fraction	0.50	0.50	
NH4/TKN Fraction	0.49	0.51	

Model Notes:

Model Name: Design CSO bypass MM Dry Existing Solids.bwc

Elements	Total COD [mg/L]	Total Carbonaceous BOD [mg/L]	Total Kjeldahl Nitrogen [mgN/L]	Total N [mgN/L]	Total suspended solids [mgTSS/L]	Volatile suspended solids [mgVSS/L]	Ammonia N [mgN/L]	Nitrate N [mgN/L]
Influent	181.53	86.81	12.4	13	65.12	48.8	6.08	0.6
Sanitary Sewer Primaries	143.15	70.07	10.4	11.04	41.68	26.95	5.77	0.64
1A	2966.86	968.42	181.38	183.62	2,988	1,989	3.94	1.93
1B	2954.01	959.73	180.38	183.53	2,986	1,987	2.91	2.73
1C	2942.44	952.1	179.46	183.44	2,982	1,983	2	3.55
1D	2932.12	945.34	178.65	183.37	2,978	1,978	1.31	4.35
Effluent	44.04	9.59	3.54	8.26	12.95	8.6	1.31	4.35
Stormwater	125.49	59.54	6.61	7.21	26.72	21.37	4.04	0.6

Solids Train Results

Elements	Total suspended solids [mgTSS/L]	Total suspended solids [lb TSS/d]	Volatile suspended solids [mgVSS/L]	Volatile suspended solids [lb VSS/d]	Flow [mgd]	Flow [lb /d]
Influent	65.12	32,605	48.8	24,435	60	-----
Sanitary Sewer Primaries	41.68	23,342	26.95	15,096	67	-----
Sanitary Sewer Primaries (U)	4,912	20,700	3,176	13,387	1	-----
Secoondary Clarifier	12.95	7,162	8.6	4,757	66.27	-----
Secoondary Clarifier (U)	13,895	2,087,259	9,230	1,386,501	18	-----
WAS RAS Splitter	2,978	2,094,421	1,978	1,391,259	84	-----
WAS RAS Splitter (U)	2,978	20,876	1,978	13,867	1	-----
Belt Filter Press	1,237	961	843	655	0	-----
Belt Filter Press (U)	258,345	47,108	176,000	32,093	0	-----
Gravity Thickner	191	2,530	130	1,724	2	-----
Gravity Thickner (U)	50,087	48,070	34,122	32,748	0	-----
Sludge	258,344.6	47,108	175,999.8	32,093	0.0	-----
Stormwater Primaries	27	10,176	21	8,138	46	-----
Stormwater Primaries (U)	3,012	9,024	2,409	7,217	0	-----

Condition 5: Average - Lime Stab Solids Train	Temp	SRT
Winter	8.8	10
Sanitary Influent Flow	51	mgd
CSO Flow	60	
Model Influent Flow	111	

	Sewer Influent	CSO Influent	
BOD:TKN Ratio	6.47	-	
AOB Growth Rate Coefficient	0.76	-	default is 0.9
RAS Flow, mgd	15.30	-	30% of influent
Fbs	0.22	0.16	
Fxsp	0.60	0.03	
Fus	0.14	0.05	
Fup	0.10	0.40	
P.Clarifier Removal	47.00%	47.00%	default is 60%
OP/TP Fraction	0.50	0.50	
NH4/TKN Fraction	0.50	0.51	

Model Notes:

Model Name: Design CSO bypass Avg Dry Existing Solids no incin.bwc

Elements	Total COD [mg/L]	Total Carbonaceous BOD [mg/L]	Total Kjeldahl Nitrogen [mgN/L]	Total N [mgN/L]	Total suspended solids [mgTSS/L]	Volatile suspended solids [mgVSS/L]	Ammonia N [mgN/L]	Nitrate N [mgN/L]
Influent	164.89	78.61	12.2	12.8	60.31	45.2	6.1	0.6
Sanitary Sewer Primaries	131.63	63.85	9.87	10.5	32.78	24.9	5.61	0.63
1A	3178.24	910.48	186.45	188.76	2,773	2,120	3.64	2.03
1B	3165.96	902.18	185.34	188.67	2,770	2,118	2.49	2.97
1C	3154.94	894.92	184.36	188.59	2,767	2,114	1.53	3.9
1D	3145.11	888.47	183.57	188.52	2,762	2,109	0.88	4.73
Effluent	51.62	12.1	3.75	8.69	21.84	16.68	0.88	4.73
Stormwater	125.49	59.54	6.61	7.21	26.72	21.37	4.04	0.6

Solids Train Results

Elements	Total suspended solids [mgTSS/L]	Total suspended solids [lb TSS/d]	Volatile suspended solids [mgVSS/L]	Volatile suspended solids [lb VSS/d]	Flow [mgd]	Flow [lb /d]
Influent	60.31	25,671	45.2	19,238	51	-----
Sanitary Sewer Primaries	32.78	18,003	24.9	13,676	66	-----
Sanitary Sewer Primaries (U)	3788.21	15,965	2877.65	12,128	1	-----
Secoondary Clarifier	21.84	11,909	16.68	9,094	65	-----
Secoondary Clarifier (U)	14,466	1,847,144	11,046	1,410,451	15	-----
WAS RAS Splitter	2762.33	1,859,053	2109.27	1,419,545	80.64	-----
WAS RAS Splitter (U)	2,762	10,581	2,109	8,080	0	-----
Belt Filter Press	1,306	676	1,007	521	0	-----
Belt Filter Press (U)	256,010	33,116	197,382	25,532	0	-----
Gravity Thickner	171	1,779	132	1,371	1	-----
Gravity Thickner (U)	52,247	33,792	40,282	26,053	0	-----
Sludge	256,009.7	33,116	197,382.2	25,532	0.0	-----
Stormwater Primaries	27	10,176	21	8,138	46	-----
Stormwater Primaries (U)	3,012	9,024	2,409	7,217	0	-----

Condition 6: Max Month - Lime Stab Solids Train	Temp	SRT
Winter	8.8	10.46
Sanitary Influent Flow	60	mgd
CSO Flow	51	
Model Influent Flow	111	

	Sewer Influent	CSO Influent	
BOD:TKN Ratio	6.74	-	
AOB Growth Rate Coefficient	0.76	-	default is 0.9
RAS Flow, mgd	18.00	-	30% of influent
Fbs	0.22	0.16	
Fxsp	0.60	0.03	
Fus	0.14	0.05	
Fup	0.10	0.40	
P.Clarifier Removal	47.00%	47.00%	default is 60%
OP/TP Fraction	0.50	0.50	
NH4/TKN Fraction	0.49	0.51	

Model Notes:

Model Name: Design CSO bypass MM Dry Existing Solids no incin.bwc

Elements	Total COD [mg/L]	Total Carbonaceous BOD [mg/L]	Total Kjeldahl Nitrogen [mgN/L]	Total N [mgN/L]	Total suspended solids [mgTSS/L]	Volatile suspended solids [mgVSS/L]	Ammonia N [mgN/L]	Nitrate N [mgN/L]
Influent	181.53	86.81	12.4	13	65.12	48.8	6.08	0.6
Sanitary Sewer Primaries	145.35	71.17	10.56	11.2	36.28	27.32	5.86	0.65
1A	3426.92	1054.98	207.94	210.6	3,077	2,302	3.63	2.37
1B	3413.22	1045.69	206.71	210.5	3,074	2,298	2.39	3.44
1C	3401.1	1037.7	205.65	210.41	3,070	2,294	1.39	4.47
1D	3390.34	1030.66	204.83	210.33	3,065	2,289	0.76	5.33
Effluent	44.8	9.09	3.05	8.56	12.79	9.55	0.76	5.33
Stormwater	125.49	59.54	6.61	7.21	26.72	21.37	4.04	0.6

Solids Train Results

Elements	Total suspended solids [mgTSS/L]	Total suspended solids [lb TSS/d]	Volatile suspended solids [mgVSS/L]	Volatile suspended solids [lb VSS/d]	Flow [mgd]	Flow [lb /d]
Influent	65.12	32,605	48.8	24,435	60	-----
Sanitary Sewer Primaries	36.28	19,982	27.32	15,048	66	-----
Sanitary Sewer Primaries (U)	4204.6	17,720	3166.44	13,345	1	-----
Secoondary Clarifier	12.79	6,974	9.55	5,209	65	-----
Secoondary Clarifier (U)	14,144	2,124,666	10,563	1,586,760	18	-----
WAS RAS Splitter	3064.88	2,131,640	2288.94	1,591,968	83.34	-----
WAS RAS Splitter (U)	3,065	16,881	2,289	12,607	1	-----
Belt Filter Press	1,226	829	932	630	0	-----
Belt Filter Press (U)	256,144	40,615	194,753	30,881	0	-----
Gravity Thickner	184	2,181	140	1,658	1	-----
Gravity Thickner (U)	49,661	41,444	37,758	31,511	0	-----
Sludge	256,144.1	40,615	194,753.0	30,881	0.0	-----
Stormwater Primaries	27	10,176	21	8,138	46	-----
Stormwater Primaries (U)	3,012	9,024	2,409	7,217	0	-----

Condition 7: Average - Digester Solids Train	Temp	SRT
Winter	8.8	10
Sanitary Influent Flow	51	mgd
CSO Flow	60	
Model Influent Flow	111	

	Sewer Influent	CSO Influent	
BOD:TKN Ratio	5.82	-	
AOB Growth Rate Coefficient	0.76	-	default is 0.9
RAS Flow, mgd	15.30	-	30% of influent
Fbs	0.22	0.16	
Fxsp	0.60	0.03	
Fus	0.14	0.05	
Fup	0.10	0.40	
P.Clarifier Removal	47.00%	47.00%	default is 60%
OP/TP Fraction	0.50	0.50	
NH4/TKN Fraction	0.50	0.51	

Model Notes:

Model Name:

Elements	Total COD [mg/L]	Total Carbonaceous BOD [mg/L]	Total Kjeldahl Nitrogen [mgN/L]	Total N [mgN/L]	Total suspended solids [mgTSS/L]	Volatile suspended solids [mgVSS/L]	Ammonia N [mgN/L]	Nitrate N [mgN/L]
Influent	164.89	78.61	12.2	12.8	60.34	45.2	6.1	0.6
Sanitary Sewer Primaries	131.18	63.68	10.94	11.58	32.48	24.62	6.71	0.63
1A	3176.18	916.71	187.2	190.02	2,772	2,120	4.22	2.5
1B	3163.99	908.48	185.8	189.93	2,770	2,117	2.77	3.73
1C	3153.04	901.27	184.59	189.85	2,766	2,113	1.57	4.93
1D	3143.26	894.84	183.66	189.78	2,762	2,109	0.78	5.93
Effluent	51.56	12.14	3.65	9.77	21.78	16.63	0.78	5.93
Stormwater	125.55	59.54	6.61	7.21	26.77	21.41	4.04	0.6

Solids Train Results

Elements	Total suspended solids [mgTSS/L]	Total suspended solids [lb TSS/d]	Volatile suspended solids [mgVSS/L]	Volatile suspended solids [lb VSS/d]	Flow [mgd]	Flow [lb /d]
Influent	60.34	25,683	45.2	19,238	51	-----
Sanitary Sewer Primaries	32.48	17,839	24.62	13,522	66	-----
Sanitary Sewer Primaries (U)	3,754	15,820	2,845	11,991	1	-----
Secoondary Clarifier	22	11,879	17	9,069	65	-----
Secoondary Clarifier (U)	14,466	1,847,146	11,044	1,410,172	15	-----
WAS RAS Splitter	2762.1	1,859,025	2108.68	1,419,241	80.65	-----
WAS RAS Splitter (U)	2762.1	10,580	2108.68	8,077	0.46	-----
Belt Filter Press	832	450	568.77	307	0.06	-----
Belt Filter Press (U)	250,432	22,032	171,198	15,062	0	-----
Gravity Thickner	162	1,683	124	1,294	1	-----
Gravity Thickner (U)	51,087	31,976	39,270	24,580	0	-----
Sludge	250,432	22,032	171,198	15,062	0	-----
Primary Digester	37,032	23,271	25,674	16,134	0	-----
Secondary Digester	35,776	22,482	24,457	15,369	0	-----
Stormwater Primaries	27	8,185	21	6,546	37	-----
Stormwater Primaries (U)	2,423	7,258	1,938	5,805	0	-----

Condition 8: Maximum Month - Digester Solids Train	Temp	SRT
Winter	8.8	10.3
Sanitary Influent Flow	60	mgd
CSO Flow	51	
Model Influent Flow	111	

	Sewer Influent	CSO Influent	
BOD:TKN Ratio	5.95	-	
AOB Growth Rate Coefficient	0.76	-	default is 0.9
RAS Flow, mgd	18.00	-	30% of influent
Fbs	0.22	0.16	
Fxsp	0.60	0.03	
Fus	0.14	0.05	
Fup	0.10	0.40	
P.Clarifier Removal	47.00%	47.00%	default is 60%
OP/TP Fraction	0.50	0.50	
NH4/TKN Fraction	0.49	0.51	

Model Notes:

Model Name:

Elements	Total COD [mg/L]	Total Carbonaceous BOD [mg/L]	Total Kjeldahl Nitrogen [mgN/L]	Total N [mgN/L]	Total suspended solids [mgTSS/L]	Volatile suspended solids [mgVSS/L]	Ammonia N [mgN/L]	Nitrate N [mgN/L]
Influent	181.53	86.81	12.4	13	65.12	48.8	6.08	0.6
Sanitary Sewer Primaries	144.96	70.94	11.93	12.59	36.02	27.08	7.25	0.66
1A	3395.24	1055.65	206.92	210.23	3,045	2,280	4.37	2.97
1B	3381.71	1046.48	205.34	210.13	3,042	2,277	2.77	4.38
1C	3369.72	1038.57	204	210.04	3,038	2,273	1.47	5.72
1D	3359.05	1031.58	203.02	209.96	3,033	2,268	0.68	6.79
Effluent	44.85	9.18	2.98	9.92	12.75	9.53	0.68	6.79
Stormwater	125.49	59.54	6.61	7.21	26.72	21.37	4.04	0.6

Solids Train Results

Elements	Total suspended solids [mgTSS/L]	Total suspended solids [lb TSS/d]	Volatile suspended solids [mgVSS/L]	Volatile suspended solids [lb VSS/d]	Flow [mgd]	Flow [lb /d]
Influent	65.12	32,605	48.8	24,435	60	-----
Sanitary Sewer Primaries	36.02	19,848	27.08	14,918	66	-----
Sanitary Sewer Primaries (U)	4,176	17,601	3,139	13,229	1	-----
Secoondary Clarifier	12.75	6,955	9.53	5,200	65.35	-----
Secoondary Clarifier (U)	14,000	2,102,994	10,467	1,572,255	18	-----
WAS RAS Splitter	3,033	2,109,949	2,268	1,577,454	83	-----
WAS RAS Splitter (U)	3,033	17,012	2,268	12,719	1	-----
Belt Filter Press	792	575	531	385	0	-----
Belt Filter Press (U)	248,752	28,151	166,700	18,865	0	-----
Gravity Thickner	182	2,182	138	1,658	1	-----
Gravity Thickner (U)	49,674	41,455	37,754	31,507	0	-----
Sludge	248,751.7	28,151	166,699.6	18,865	0.0	-----
Primary Digester	35,645	29,881	24,288	20,360	0	-----
Secondary Digester	34,267	28,726	22,964	19,250	0	-----
Stormwater Primaries	27	10,176	21	8,138	46	-----
Stormwater Primaries (U)	3,012	9,024	2,409	7,217	0	-----

APPENDIX B

FINAL SETTLING TANK CFD MODEL CALIBRATION FILES

DRAFT

Estimation of Capacity of Final Settling Tanks No. 4 and 5 for the Oneida WWTP

Alex McCorquodale, P.E.

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December 29, 2011

Model Validation

The models for both FSTs No. 4 and 5 were validated using data collected by Brown and Caldwell on November 1,, 2011.

A Vesilind test was conducted that gave the settling velocity as:

[1] $Vs = 12.618.e^{-0.254.x}$

where x = conc. in g/L.

The flocculation parameters were $Ka = 5.1*10^{-8}$ L/mg; $Ka = 3.5*10^{-9}$ s.

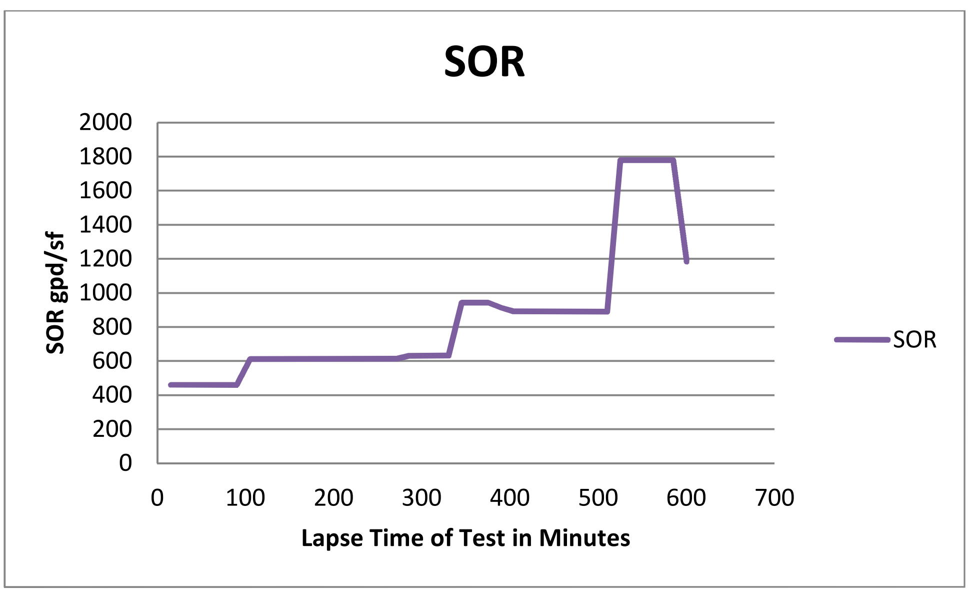

Figure 1. SOR used in the Validation Testing

The MLSS was sampled and estimated in both the FST 4 and FST 5 streams. A few of the MLSS values appear to be abnormally low, especially in the stream of FST No. 5. The mean value of the MLSS for FST No. 4 was 1803 mg/L while No. 5 had values in the range of 1100 to 1400 mg/L. Table 1 shows observed and modeled blanket depths at the end of the stress test. A reasonably good agreement was found for FST No. 4. With the mean MLSS of 1100 mg/L, the model significantly under-estimated the blanket depth in No. 5; a better agreement was found when the normal plant MLSS of about 1400 was used in the model.

Table 1. Modeled and Measured Sludge Depths at End of Validation Testing

FST	Location 1	Location 3	Location 3	Comment
#4 Obs SBD ft	1.5	2.0	1.0	End of Test
#4 Modeled	2.0	2.0	1.2	MLSS 1803
#5 Obs SBD ft	2.0	2.8	2.0	End of Test
#5 Modeled	2.2	2.4	1.0	MLSS 1400
#5 Modeled	1.0	2.0	0.8	MLSS 1100

The comparison of the observed and modeled ESS for FST No. 4 is shown in Figure 2. The model tracks the trend of the ESS very well. The solids distribution and flow pattern near the end of the stress test is given in Figure 3. The blanket is partially scoured at the head of the tank. The blanket depth increases towards the cross-collector and then decreases to a minimum near the end wall.

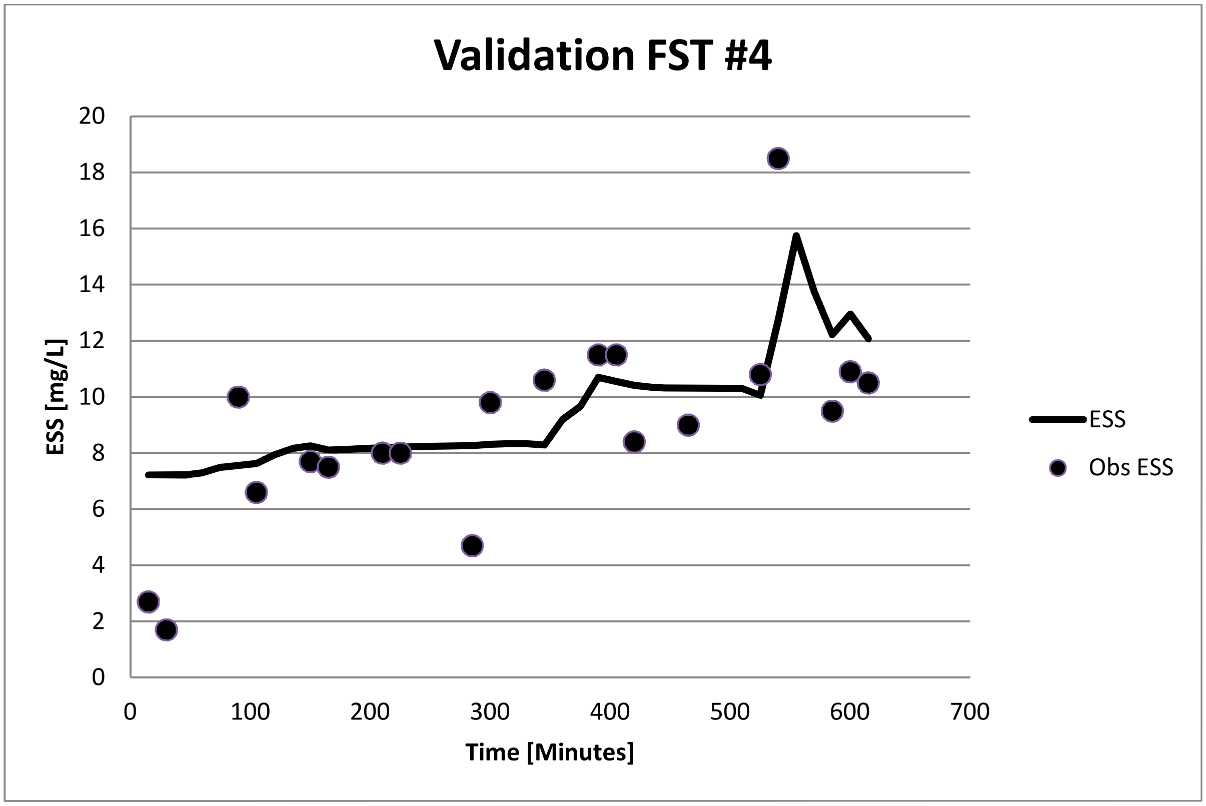

Figure 2. ESS Validation of the Model for FST 4

Validation of FST No. 4

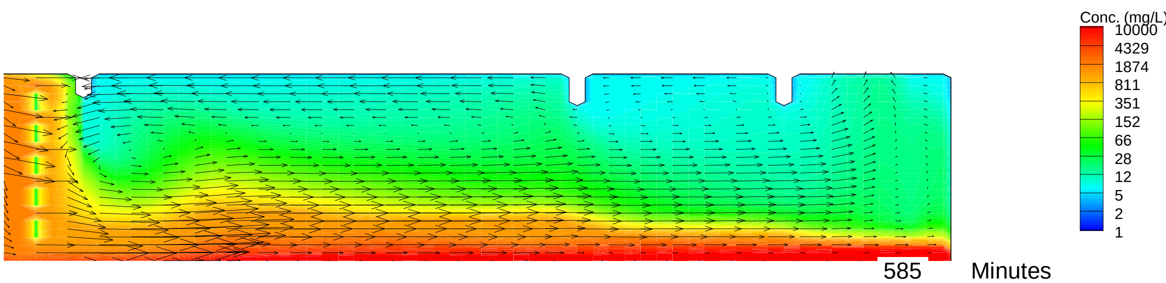

Figure 3. Solids and flow distribution near the end of the stress test for FST No. 4.

The comparison of the observed and modeled ESS for FST No. 5 is shown in Figure 4. The model tracks the trend of the ESS reasonably well. The solids distribution and flow pattern near the end of the stress test is given in Figure 5. The blanket depth shows a similar pattern to that in FST No. 4.

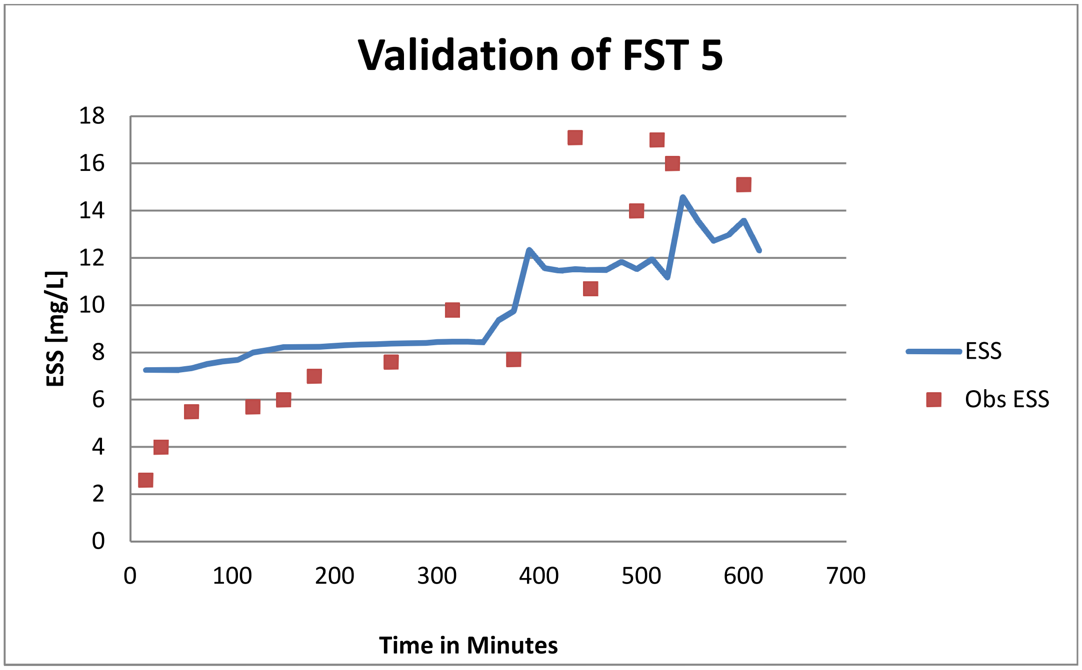

Figure 4. ESS Validation of the Model for FST 5

Validation of FST No. 5

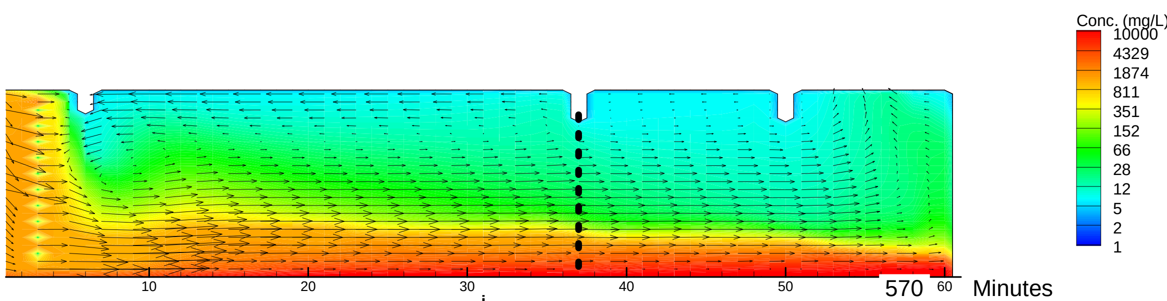

Figure 5. Of the flow and solids pattern in FST No. 5 with MLSS = 1400 mg/L

Estimation of Vesilind Parameters from SVI for Capacity Study

The Wahlberg-Keinath (1988a, b) correlated Vo and K with SSVI. Their regressions were compared with the observed Vo and K for the Oneida data for the calibration and validation tests and the mean of several other regression equations given in Dimosthenis et al (2003). The conversion of SVI to SSVI is approximately,

[2] SSVI=0.8*SVI

The best fit to the observed data is:

[3] $Vo = 15.3 - 0.0615 * SSVI$

[4] $K = 0.80\{0.426 - 0.00384 * SSVI + 5.43 * 10^{-5} * SSVI^2\}$

Figures 6 and 7 compare the Vo and K from Equations 3 and 4 with the Wahlberg-Keinath Equation and the mean of several regression equations from Dimosthenis et al (2003).

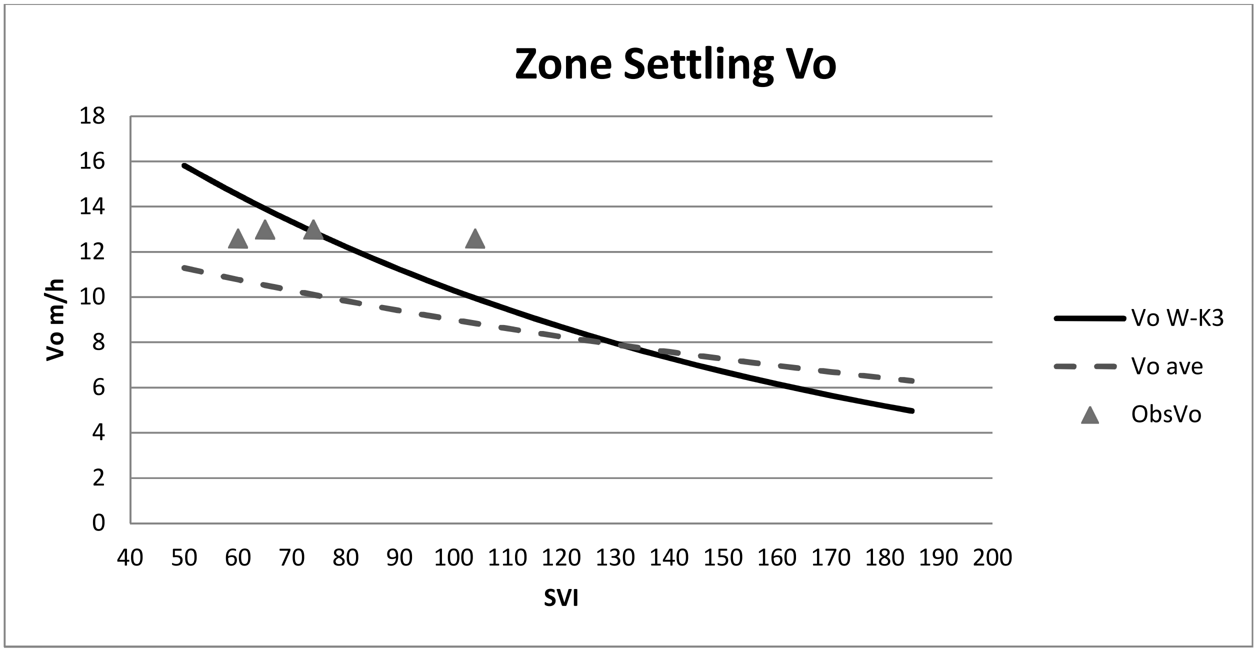

Figure 6. Estimation of Vo based on the re-calibration of the Wahlberg-Keinath Equation
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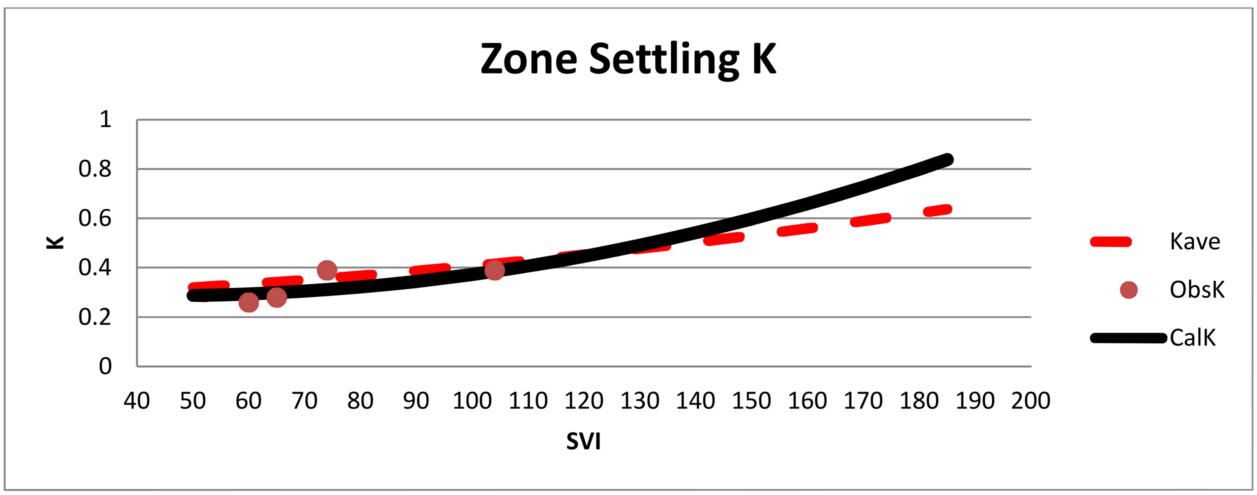

Figure 7. Estimation of K based on the re-calibration of the Wahlberg-Keinath Equation

Capacity Simulations

The 90-%tile SVI is estimated to be 125 and Equations 3 and 4 give Vo = 9.15 m/h and K = 0.47.

Figure 8 shows the variation of ESS with SOR for SVI = 125 and MLSS = 3000 mg/L with RAS flow of 100 L/s. RAS flows were varied from 46 to 101 L/s. The tank failed due to a rising blanket at RAS flows less than 90 L/s.

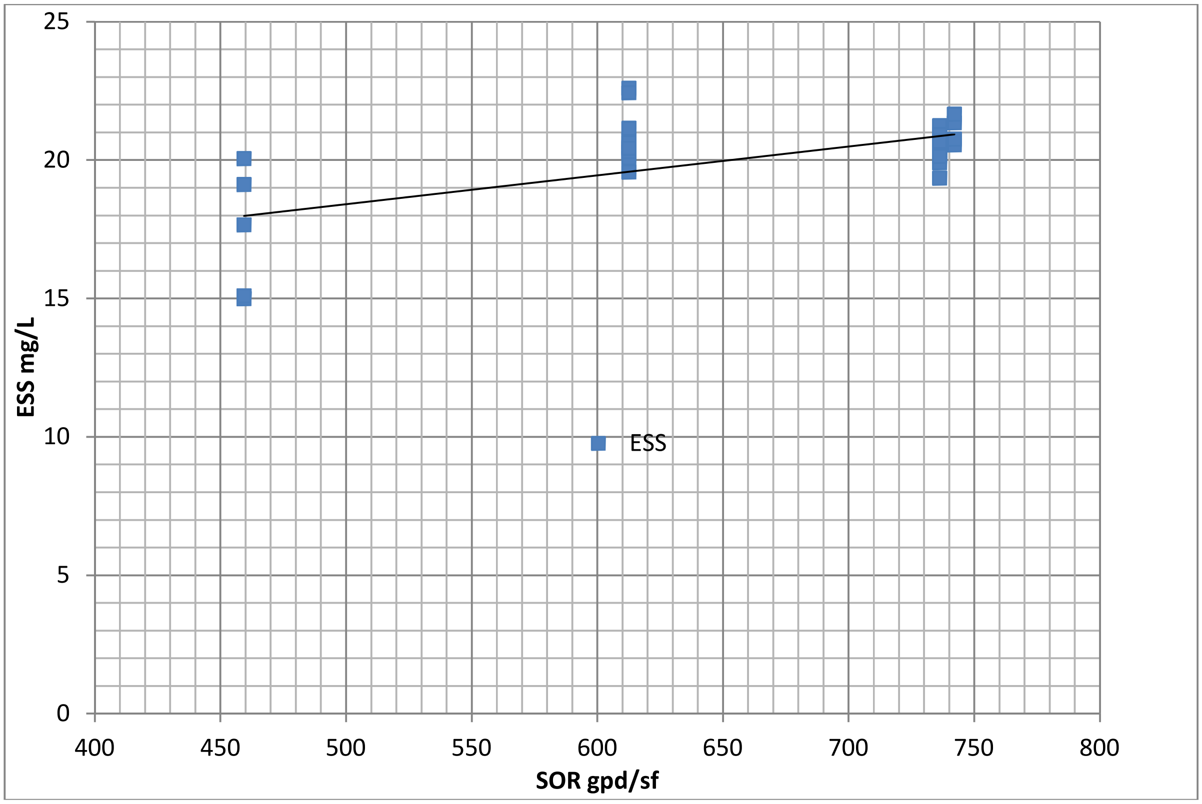

Figure 9. FST No. 5 ESS versus SOR for SVI = 125 and MLSS 3000 mg/L with RAS Flow = 101 L/s

Figure 9 shows the performance of FST No. 5 with the 90-%tile SVI and MLSS = 3000 mg/L. The limiting SOR is approximately 740 gpd/sf with an ESS of 21 mg/L+/-. The blanket depth is approximately 50% of the tank depth.

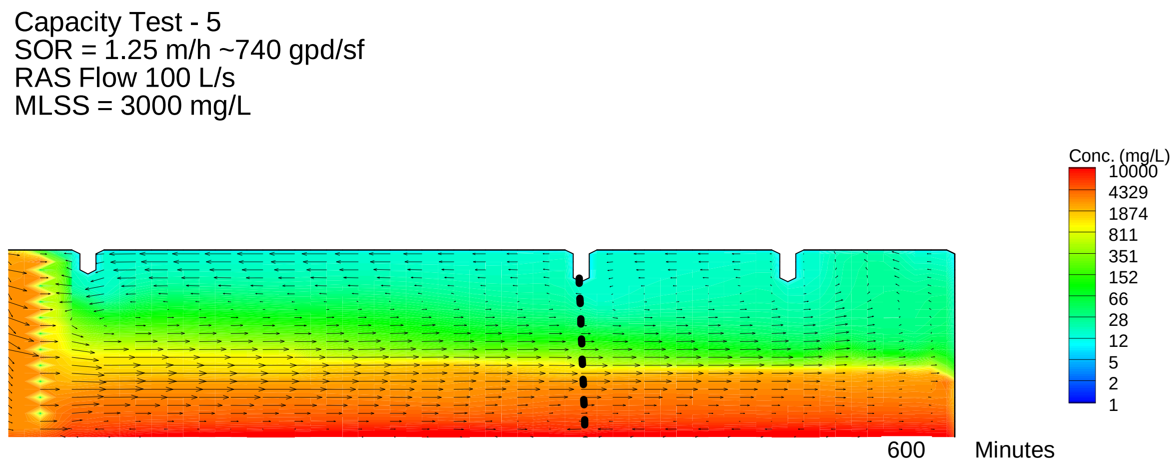

Figure 9. FST No. 5 near Capacity of 740 gpd/sf with SVI = 125 and MLSS = 3000 mg/L with RAS flow = 100 L/s (2.3 mgd).

The model was used to estimate the maximum SOR for FST No 5. Figure 10 shows the results for an SOR of 830 gpd/sf; failure occurs after 10 h due to a rising blanket.

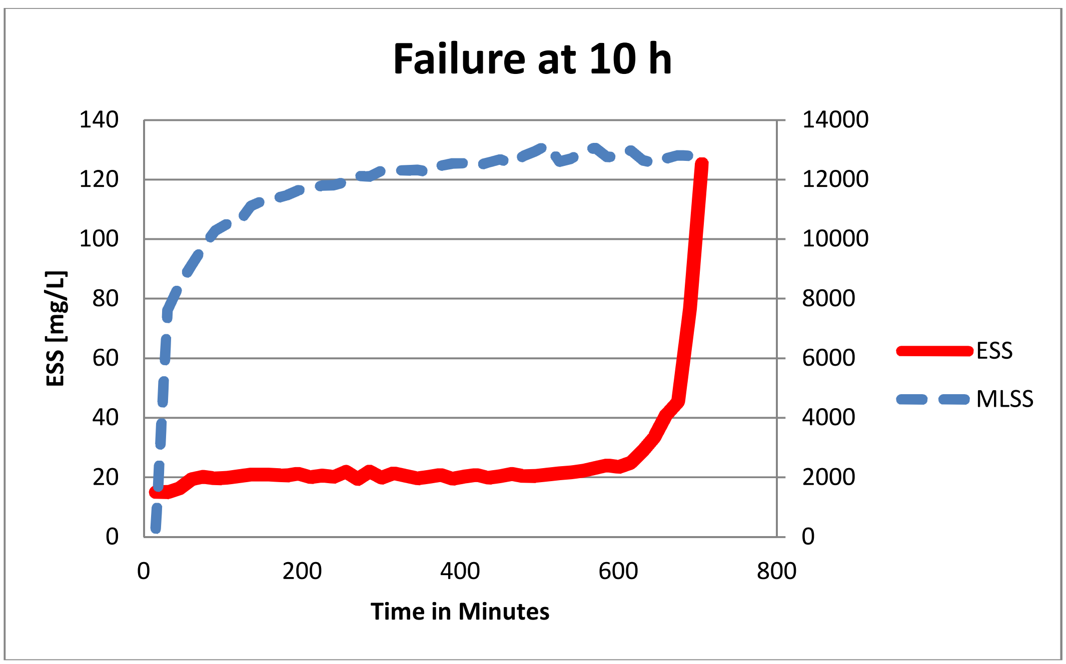

Figure 10. Failure in FST No. 5 at 830 gpd/sf with RAS flow = 100 L/s (2.3 mgd)..

Figure 11 shows the ESS and RAS SS when the SOR was decreased to 800 gpd/sf. The blanket depth for this case is presented in Figure 12.

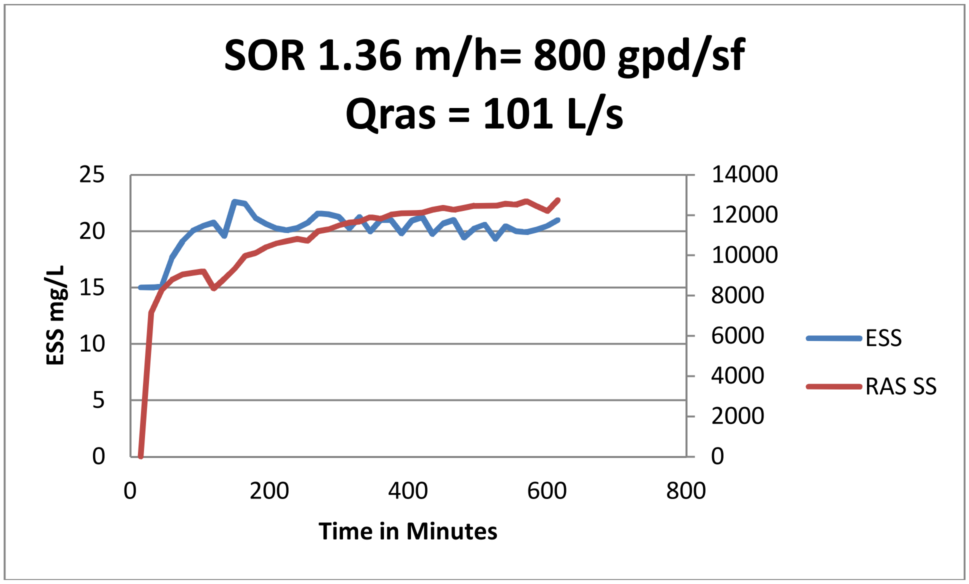

Figure 11. FST No. 5 near failure at SOR = 800 gpd/sf with RAS flow = 101 L/s (2.3 mgd).

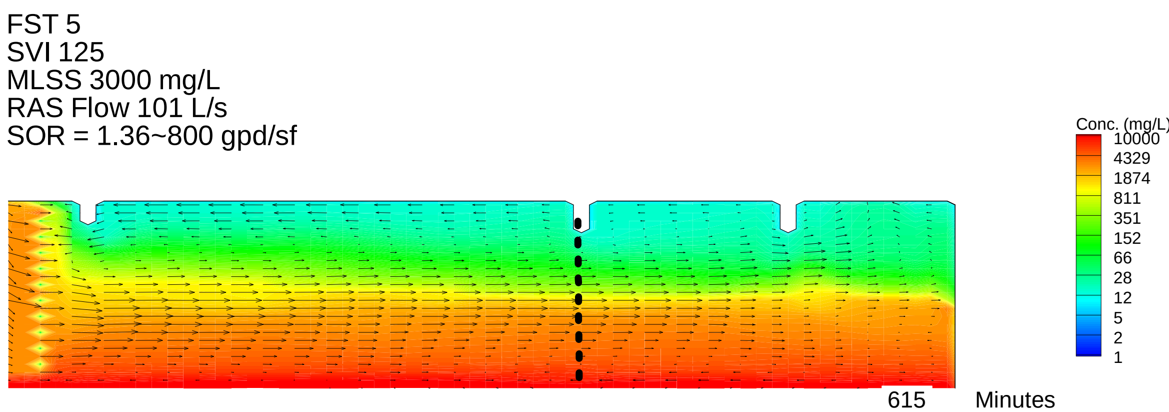

Figure 12. FST No. 5 near failure with RAS flow = 101 L/s (2.3 mgd).

The capacity of FST No. was found to vary with the RAS flow. Figures 13 and 14 show the results of the simulation with a RAS flow of 85 L/s (1.9 mgd)

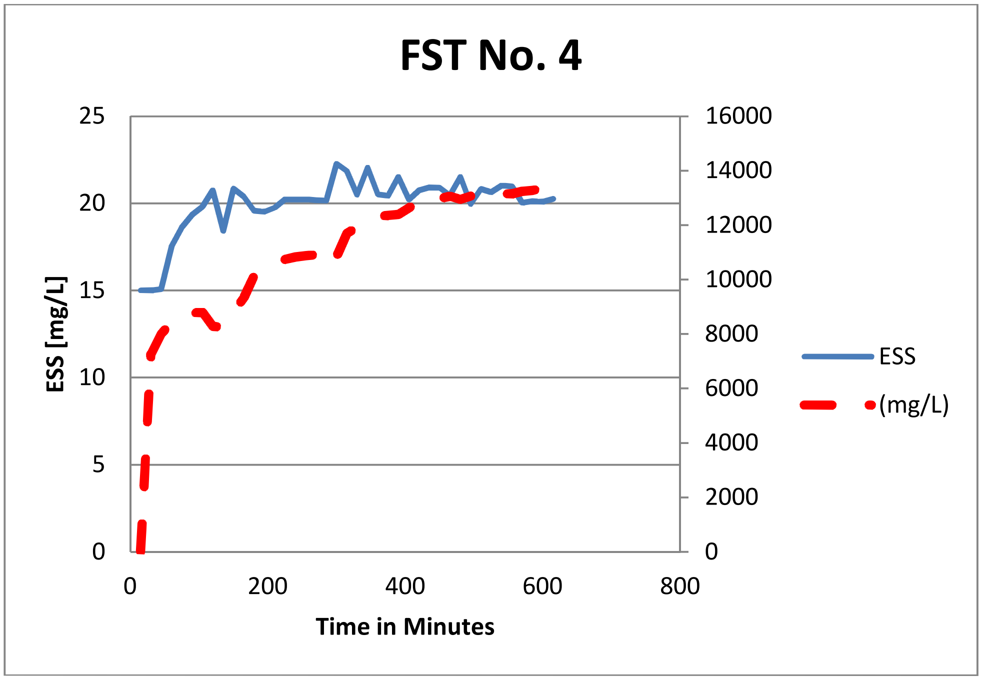

Figure 13. FST No. 4 near maximum capacity at SOR = 890 gpd/sf with RAS Flow = 85 L/s (1.9 mgd).

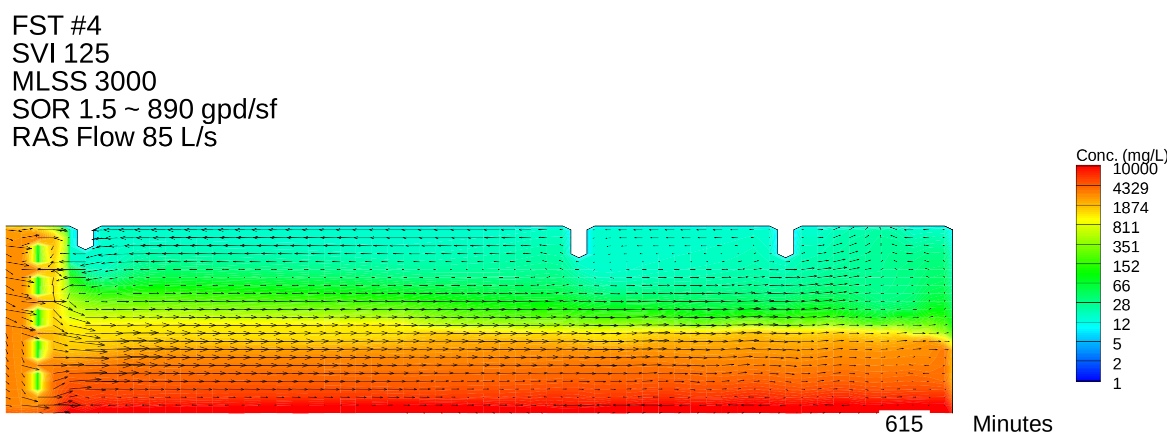

Figure 14. FST No. 4 near maximum capacity of 890 gpd/sf with SVI = 125 and MLSS = 3000 mg/L (1.9 mgd).

Conclusions

The calibrated and validated model was applied to FST No. 4 and FST No. 5 to estimate the capacity of the secondary clarifiers at 90-%tile SVI. The SVI was determined to be 125.

The Wahlberg-Keinath Equation was re-calibrated using the calibration and validation settling column data. The estimated Vo was 9.15 m/h and the K = 0.47.

The model was run for the equivalent of 10 h at various SORs and RAS flows to determine the points of failure. It was found that FST No. 4 could be operated for over 10 h at an SOR of 890 mgd at a RAS flow of 1.9 mgd per clarifier. FST No. 5 started to fail at approximately 800 mgd and RAS flow of 2.3 mgd. In general solids overflow started when the blanket exceeded 50% of the tank depth.

The tank capacities were sensitive to the RAS flow. It was also noted that the relatively short length of the launder resulted in an 'updraft' of solids that may have contributed to the failure.

References

Wahlberg, EJ, and Keinath, T.M. (1988b)   *Development of settling flux curves using SVI: an addendum.* Water Environ Res. 67(5):872–4.

Wahlberg, EJ, and Keinath, TM. (1988a) *Development of settling flux curves using SVI.* J Water Pollut Control Fed. 60(12):2095–100.

Dimosthenis et al (2003) *Comparison and evaluation of empirical zone settling velocity parameters based on sludge volume index using a unified settling characteristics database,* Water Research 37, 3821–3836. See Table below.

Table 2
Settleability tests and correlation-based equations proposed by different studies

Reference	Test	Number of data	Sludge Volume Index range (ml/g)	Correlation for V_o (m/h)	Correlation for K (m^3/Kg)	Abbreviation used in this study
Mines et al. [27]	SVI		27–236	7.27	0.0281 + 0.00229 SVI	V_0
Hartel and Pöpel [22]	SVI	—	—	$17.4e^{-0.0113\ SVI}$	$1.043\text{-}0.983e^{-0.00581\ SVI}$	V_1
Daigger and Roper [13]	SVI	236	36–402	7.8	0.148 +0.0021 SVI	V_2
Pitman [12]	SVI	697	45–360	$(V_o/k) = 37480\ e^{-0.00359\ SVI}$	0.00088-0.000393 log [0.001x (Vo/k)]	V_3
Akca et al. [23]	SVI	—	—	$28.1\ SVI^{-0.2667}$	0.177 +0.0014 SVI	V4
Wahlberg and Keinath [24]	SVI	185	48–235	$18.2\ e^{-0.00602\ SVI}$	0.351 +0.00058 SVI	V_5
Daigger [18]	SVI	> 1500	36–402	6.5	0.165 +0.001586 SVI	V_6
Ozinsky and Ekama [15]	SVI	—	—	$8.53094\ e^{-0.00165\ SVI}$	0.20036+ 0.00091 SVI	V_7
Wahlberg and Keinath [25]	SSVI	185	35–220	15.3-0.0615 SSVI	0.426-0.00384 SSVI + $5.43 \times 10^{-5}\ SSVI^2$	V_8
Wahlberg and Keinath [24]	SSVI	185	35–220	$24.3\ e^{-0.01073\ SSVI}$	0.245 +0.00296 SSVI	V_9
Renko [26]				$V = 100\ v/SSVI\ e^{-nX}$		Renko 2
Bye and Dold [16]	Various	—	—	$V_i = (H_o - (H_o\ SVI\ MLSS)/1000)/t$	where H_o = column height and t = settling time	V_{10}

V, n parameters describing zone settling velocity. In order to facilitate the direct comparison, v was arbitrarily assigned as V_o and n as k. Their values were estimated from the respective V_o and k values of equations V_8 and V_9. It was observed that the emerging settling velocities have to be multiplied by 100 instead of 1000 to bring the estimated values within the dynamic measuring range of the settling tests. Alternatively, the estimation of these parameters, results in different values as proposed by Renko [26] which does not enable direct comparison with the other equations.

D.L. Giokas et al. / Water Research 37 (2003) 3821–3836

Addendum to Report on Capacity of Final Settling Tanks No. 4 and 5 at Oneida, NY

February 26, 2012

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Introduction

The purposes of this addendum is to re-evaluate the capacities of Final Settling Tanks No. 4 and No. 5 at the Oneida WWTP with higher RAS flows (up to 4.8 mgd) and a lower MLSS of 1800 mg/L. The calibrated/validated models for FSTs 4 and 5 were used with RAS flows from 2.1 to 4.8 mgd.

Simulations

Table 1 shows the simulations that were completed in this re-evaluation. The capacity in terms of the peak SOR was found to improve when the MLSS was reduced from 3000 mg/L to 1800 mg/L. In addition, the capacity increases as the RAS flow increases. Figure 1 shows the limiting SORs as a function of the RAS flow for an MLSS of 1800 mg/L. It should be noted that the ESS is slightly degraded at normal SORs when the RAS flows are increased. For example in FST No. 4, at an SOR of 1190 gpd/sf, the ESS at a RAS flow of 2.1 mgd, the ESS is 13.8 mg/L while at a RAS flow of 4.8 mgd, the ESS increases to 16.4 mg/L. Table 1 also shows the modeled RAS SS and the RSSe which represent the RAS SS needed for mass balance with the SOR, MLSS and RAS flow that is used in the simulation.

Table 1. Simulations and Results in the Re-evaluation of Oneida Final Settling Tanks

FST	SOR gpd/sf	MLSS mg/L	RAS Q mgd	ESS mg/L	RAS SS mg/L	RSSe mg/L	RAS Ratio	Status
4	928	1800	2.1	13	9120	9078	25%	OK
5	833	1800	2.4	13	8870	8925	25%	OK
4	**1190**	**1800**	**2.1**	**13.8**	11100	11134	19%	OK
5	1066	1800	2.4	13.9	10753	10914	20%	OK
4	1302	1800	2.1	14.8	11800	12012	18%	OK
5	1190	1800	2.4	14.5	11390	11971	18%	OK
4	459	1800	2.1	12	5400	5404	50%	OK
5	1419	1800	2.4	228	12100	13935	15%	Fails
4	**1419**	1800	2.1	**16**	12400	12936	16%	**Marginal**
5	**1308**	1800	2.4	**20.2**	11900	12978	16%	**Marginal**
4	928	1800	4.8	14	4900	4929	58%	OK
5	833	1800	4.8	14	5340	5362	51%	OK
4	987	1800	4.8	14.6	5100	5128	54%	OK
5	925	1800	4.8	15.1	5730	5753	46%	OK
4	**1184**	**1800**	**4.8**	**16.4**	5780	5794	45%	OK
5	1066	1800	4.8	15.4	6330	6357	40%	OK
4	1302	1800	4.8	16	6170	6191	41%	OK
5	1184	1800	4.8	15.7	6830	6860	36%	OK
4	1419	1800	4.8	16.4	6563	6588	38%	OK
5	**1302**	1800	**4.8**	**15.7**	7330	7364	32%	OK
4	1419	1800	3.6	15.6	8150	8185	28%	OK
5	1302	1800	3.6	14.6	9165	9218	24%	OK
4	1537	1800	3.6	16	8670	8714	26%	OK
5	1540	1800	3.6	118	10133	10575	21%	Fails
4	**1620**	1800	3.6	**16.9**	9012	9085	25%	**Marginal**
5	**1419**	1800	3.6	16	9780	9890	22%	**Marginal**

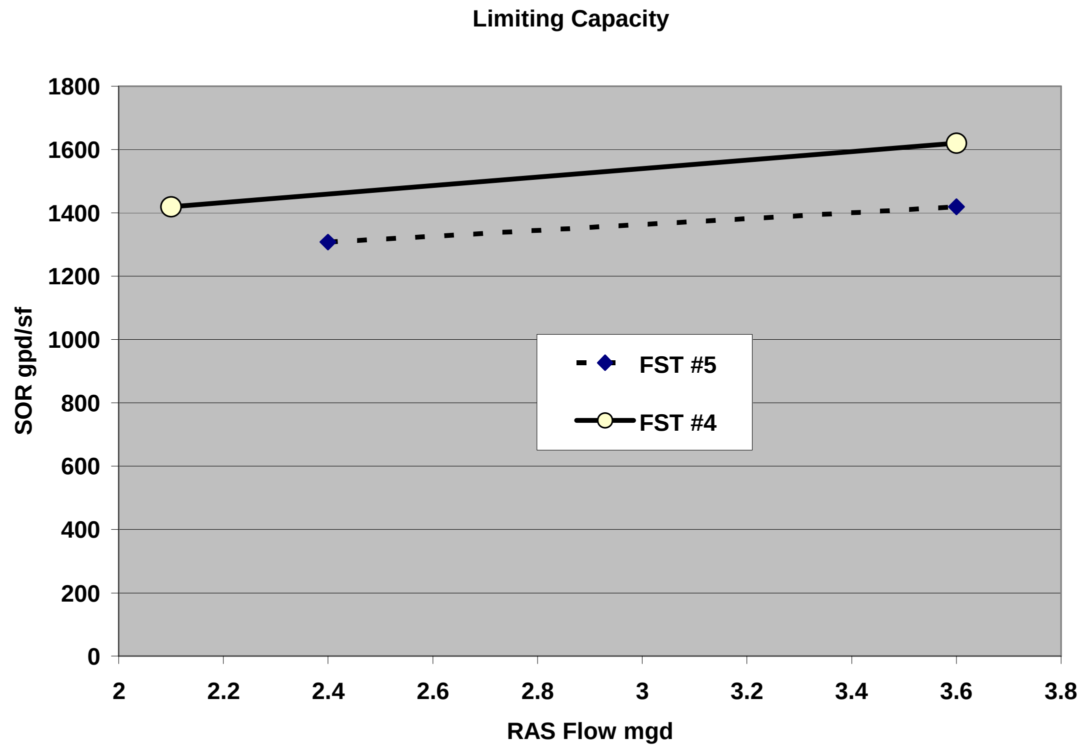

Figure 1. Limiting SOR for FSTs 4 and 5 with MLSS of 1800 mg/L

Conclusions

1. The capacity of FST No. 4 is approximately 1600 gpd/sf for a RAS Flow of 3.6 mgd for an MLSS of 1800 mg/L. A slightly higher SOR could be achieved for a RAS flow of 4.8 mgd; however, at this RAS flow, there would be degradation in the ESS for lower SORs. The expected ESS at the limiting SOR is approximately 16 mg/L.

2. The capacity of FST No. 5 is approximately 1450 gpd/sf for a RAS Flow of 3.6 mgd for an MLSS of 1800 mg/L. A slightly higher SOR could be achieved for a RAS flow of 4.8 mgd; however, at this RAS flow, the ESS would be degraded at lower SORs. The expected ESS at the limiting SOR is approximately 16 mg/L.

3. These results suggest that a proportional RAS flow might be helpful in maintaining good removal at both high and low SORs.

Estimation of Capacity of Final Settling Tanks No. 4 and 5 at Oneida, NY

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December 29, 2011 Revised February 4, 2012

Introduction

The rectangular version of the model 2Dc had previously been calibrated based on field testing conducted by Brown and Caldwell (See Appendix A). A validation test was conducted on November 1, 2011. This report includes the model validation and the estimation of the clarifier capacity for the 90 percentile SVI. FST No. 4 and FST No. 5 were field tested as representatives of the older and the new secondary clarifiers. Validations were completed for both of these tanks.

Model Validation

The models for both FSTs No. 4 and 5 were validated using data collected by Brown and Caldwell on November 1, 2011. The SOR for the stress test is shown in Figure 1.

A Vesilind test was conducted that gave the settling velocity as:

[1] $Vs = 12.618.e^{-0.254.x}$

where x = conc. in g/L.

The flocculation parameters were Ka = $5.1*10^{-8}$ L/mg; Ka =$3.\ 5*10^{-9}$ s.

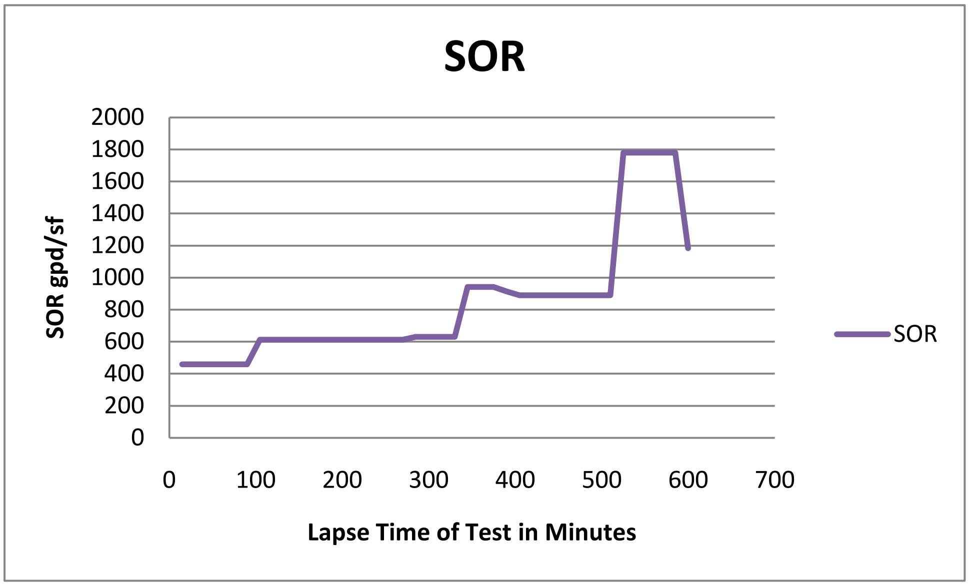

Figure 1. SOR used in the Validation Testing

The MLSS was sampled and estimated in both the FST 4 and FST 5 streams. A few of the MLSS values appear to be abnormally low, especially in the stream of FST No. 5. The mean value of the MLSS for FST No. 4 was 1803 mg/L while No. 5 had values in the range of 1100 to 1400 mg/L. Table 1 shows observed and modeled blanket depths at the end of the stress test. A reasonably good agreement was found for FST No. 4. With the mean MLSS of 1100 mg/L, the model significantly under-estimated the blanket depth in No. 5; a better agreement was found when the normal plant MLSS of about 1400 was used in the model.

Table 1. Modeled and Measured Sludge Depths at End of Validation Testing

FST	Location 1	Location 3	Location 3	Comment
#4 Obs SBD ft	1.5	2.0	1.0	End of Test
#4 Modeled	2.0	2.0	1.2	MLSS 1803
#5 Obs SBD ft	2.0	2.8	2.0	End of Test
#5 Modeled	2.2	2.4	1.0	MLSS 1400
#5 Modeled	1.0	2.0	0.8	MLSS 1100

The comparison of the observed and modeled ESS for FST No. 4 is shown in Figure 2. The model tracks the trend of the ESS very well. The solids distribution and flow pattern near the end of the stress test is given in Figure 3. The blanket is partially scoured at the head of the tank. The blanket depth increases towards the cross-collector and then decreases to a minimum near the end wall.

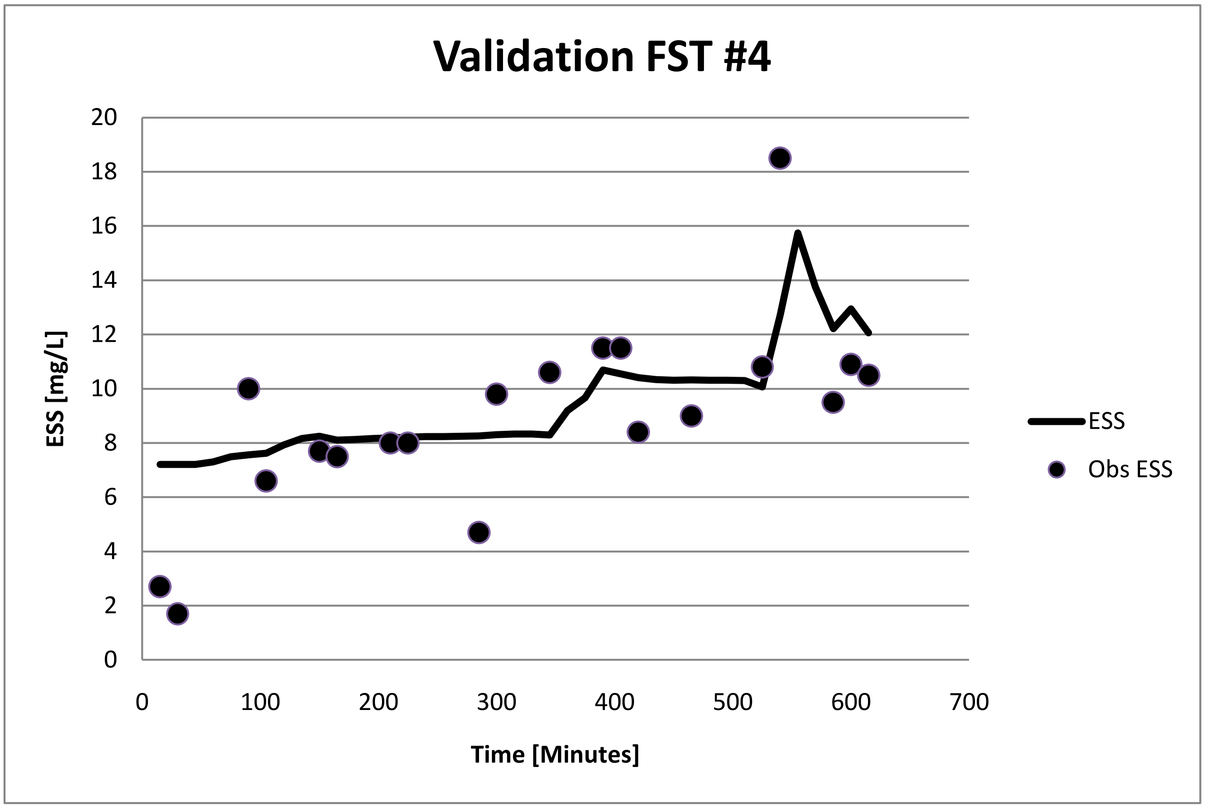

Figure 2. ESS Validation of the Model for FST No. 4

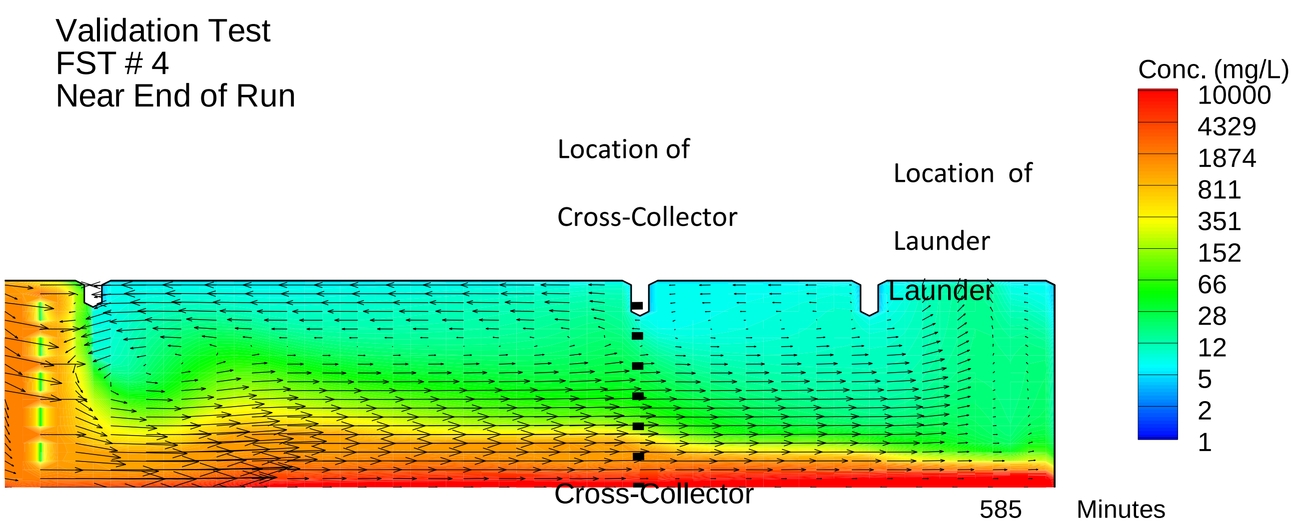

Figure 3. Solids and flow distribution near the end of the stress test for FST No. 4.

The comparison of the observed and modeled ESS for FST No. 5 is shown in Figure 4. The model tracks the trend of the ESS reasonably well. The solids distribution and flow pattern near the end of the stress test is given in Figure 5. The blanket depth shows a similar pattern to that in FST No. 4.

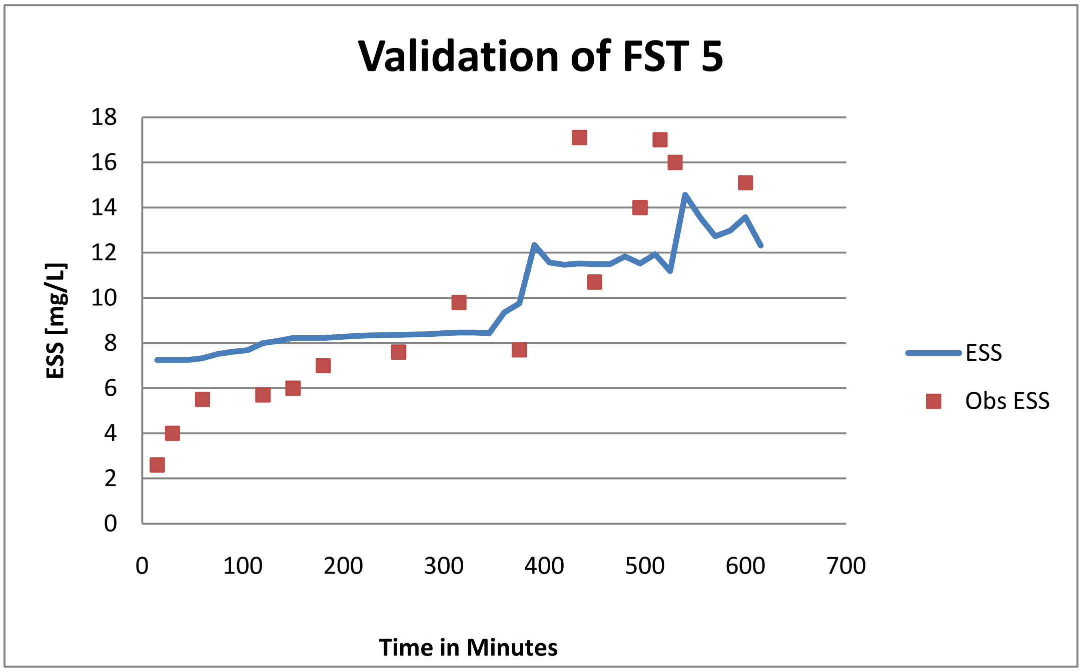

Figure 4. ESS Validation of the Model for FST No. 5

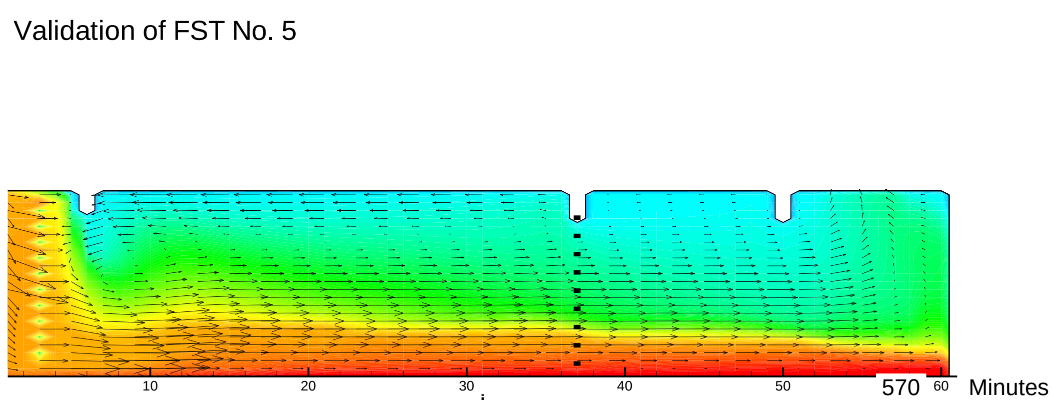

Figure 5. Of the flow and solids pattern in FST No. 5 with MLSS = 1400 mg/L

Estimation of Vesilind Parameters from SVI for Capacity Study

The Wahlberg-Keinath (1988a, b) correlated V_o and K with SSVI. Their regressions were compared with the observed V_o and K for the Oneida data for the calibration and validation tests and the mean of several other regression equations given in Dimosthenis et al (2003). The conversion of SVI to SSVI is approximately,

[2] $SSVI=0.8*SVI$

The best fit to the observed data is:

[3] $V_o =15.3-0.0615*SSVI$

[4] $K = 0.80\{0.426-0.00384*SSVI+5.43*10^{-5}*SSVI^2\}$

Figures 6 and 7 compare the Vo and K from Equations 3 and 4 with the Wahlberg-Keinath Equation and the mean of several regression equations from Dimosthenis et al (2003).

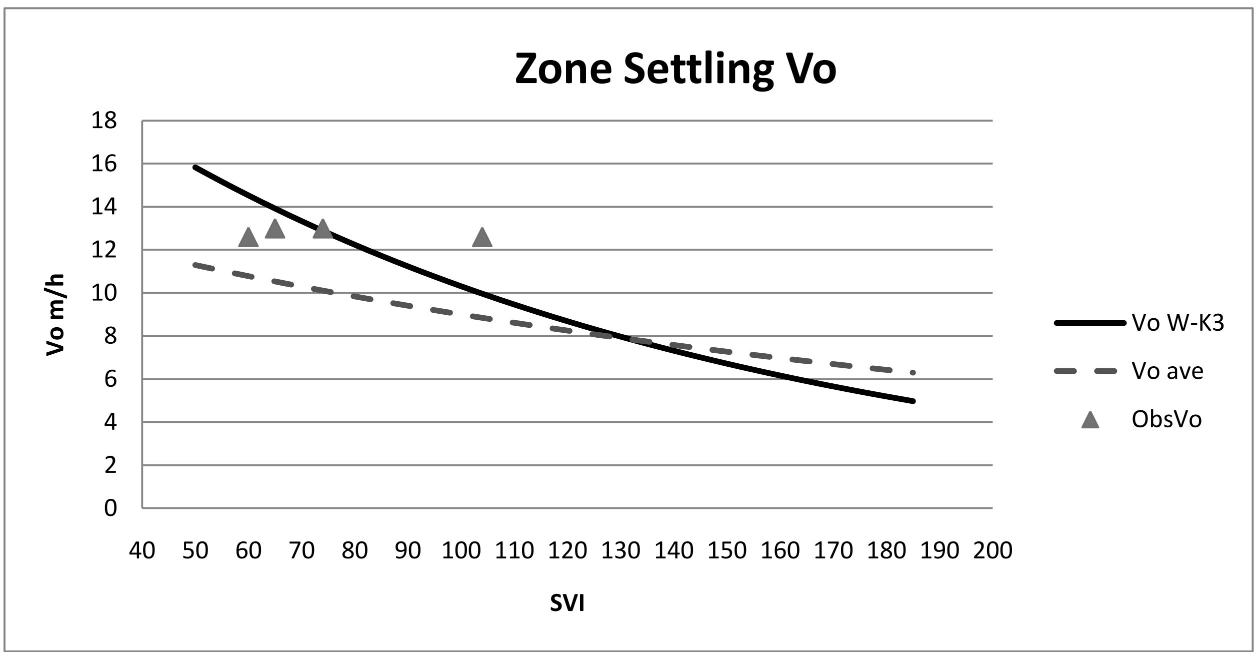

Figure 6. Estimation of V_o based on the re-calibration of the Wahlberg-Keinath Equation

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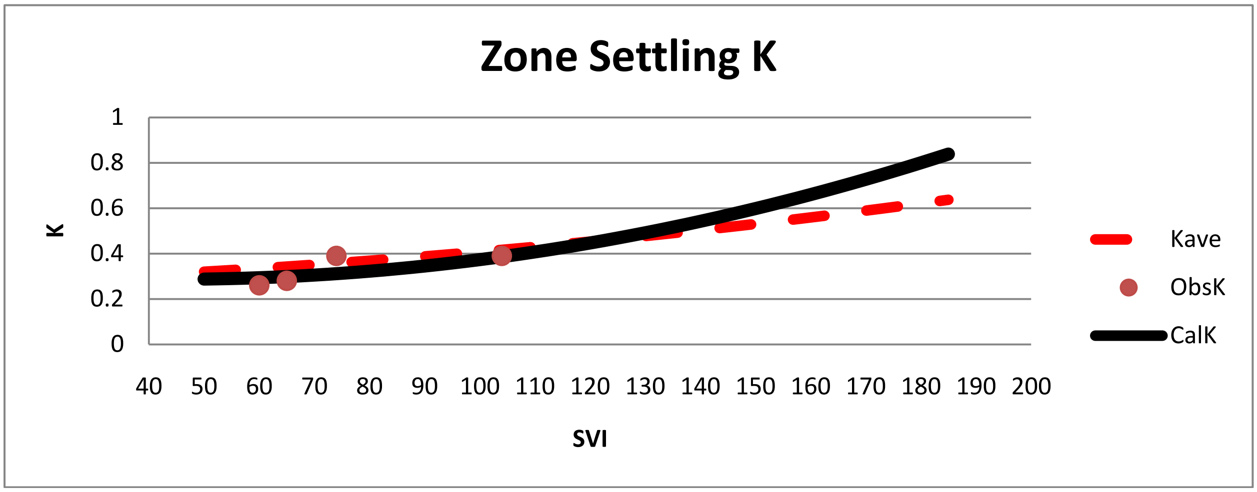


Figure 7. Estimation of K based on the re-calibration of the Wahlberg-Keinath Equation

Capacity Simulations

The 90 percentile SVI is estimated to be 125 and Equations 3 and 4 give $V_o = 9.15$ m/h and K = 0.47. Figure 8 shows the variation of ESS with SOR for SVI = 125 and MLSS = 3000 mg/L with RAS flow of 100 L/s. RAS flows were varied from 46 to 101 L/s. The tank failed due to rising blanket at RAS flows less than 90 L/s.

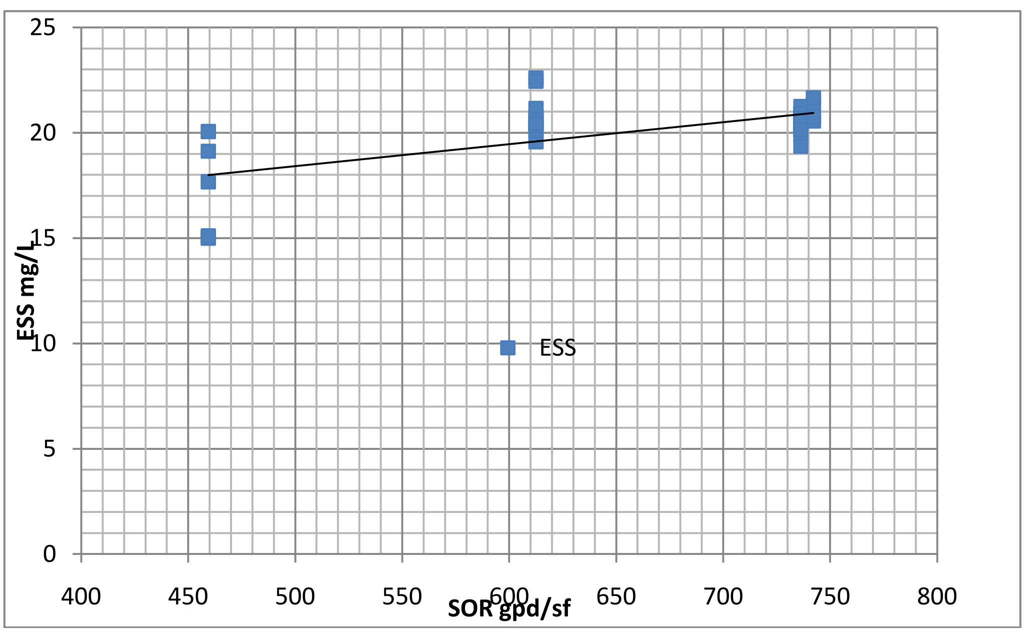


Figure 8. FST No. 5 ESS versus SOR for SVI = 125 and MLSS 3000 mg/L with RAS Flow = 101 L/s

Figure 9 shows the performance of FST No. 5 with the 90-%tile SVI and MLSS = 3000 mg/L. The limiting SOR is approximately 740 gpd/sf with an ESS of 21 mg/L+/-. The blanket depth is approximately 50% of the tank depth.

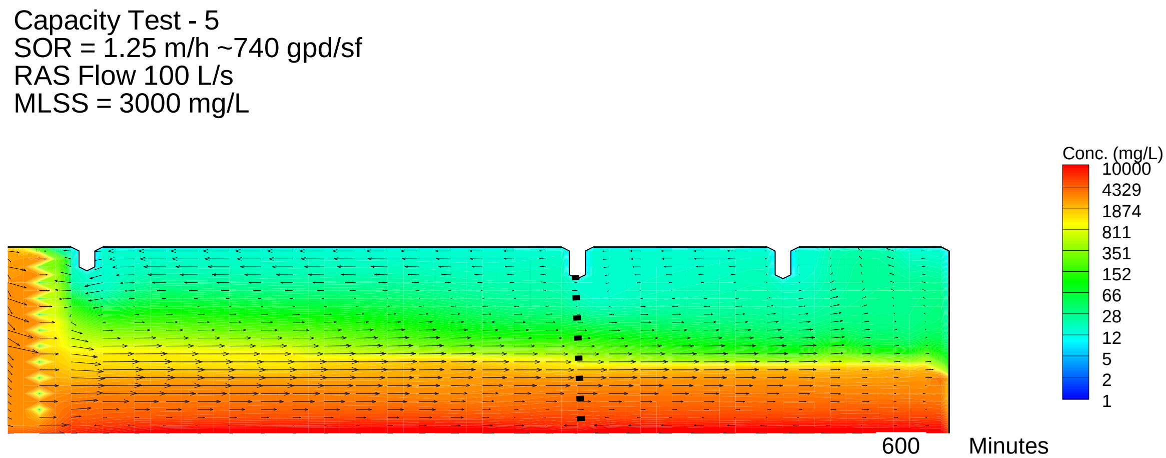


Figure 9. FST No. 5 near Capacity of 740 gpd/sf with SVI = 125 and MLSS = 3000 mg/L with RAS flow = 100 L/s (2.3 mgd).

The model was used to estimate the maximum SOR for FST No 5. Figure 10 shows the results for an SOR of 830 gpd/sf; failure occurs after 10 h due to a rising blanket.

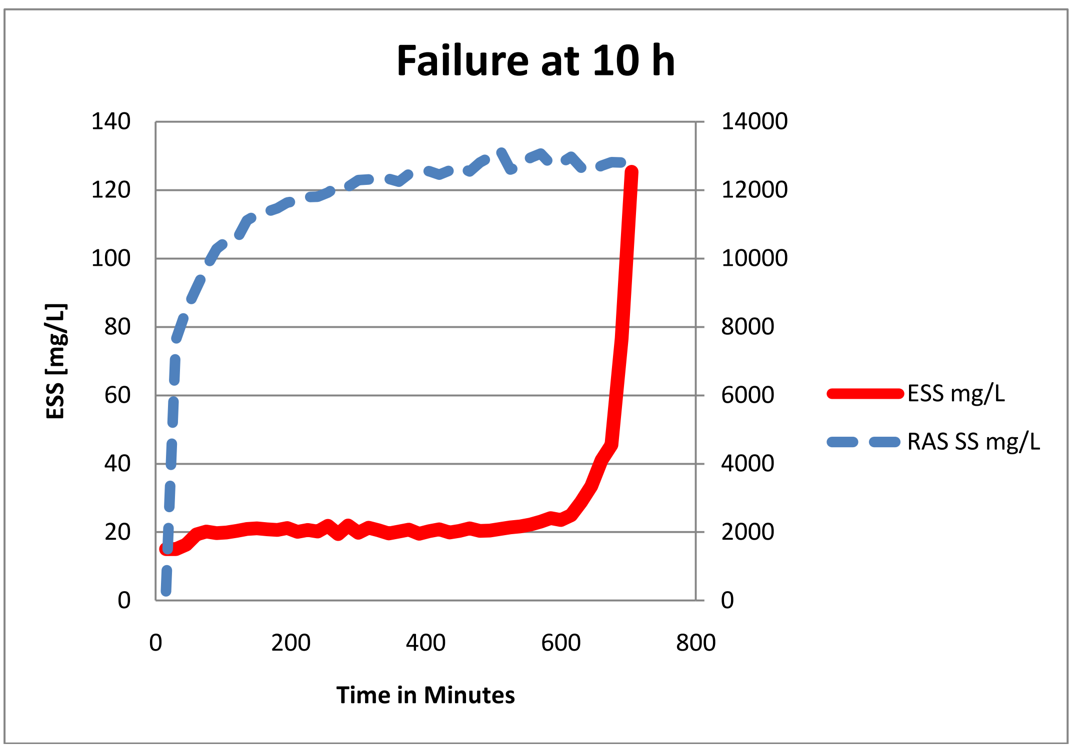


Figure 10. Failure in FST No. 5 at 830 gpd/sf with RAS flow = 100 L/s (2.3 mgd).

Figure 11 shows the ESS and RAS SS when the SOR was decreased to 800 gpd/sf. The blanket for this case is presented in Figure 12.

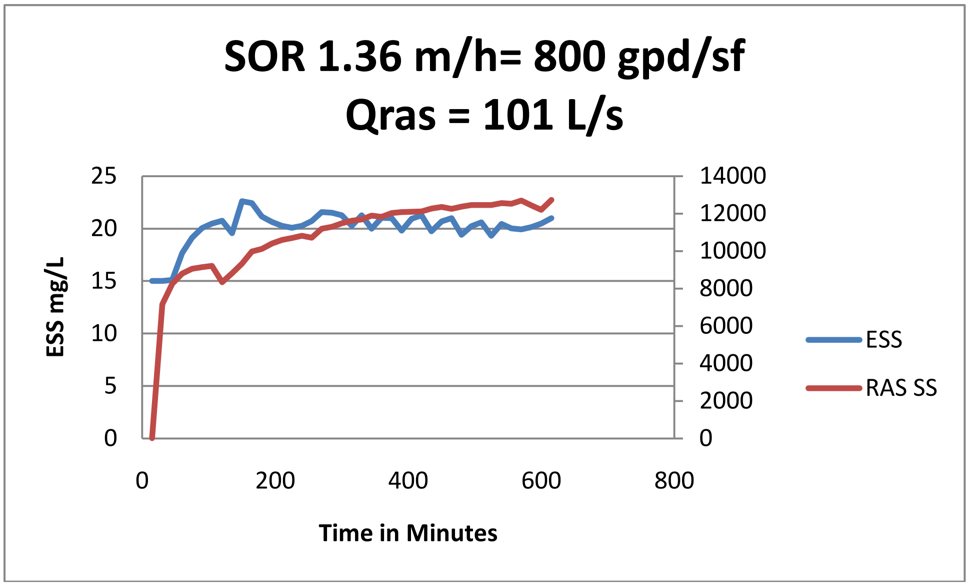


Figure 11. FST No. 5 near failure at SOR = 800 gpd/sf with RAS flow = 101 L/s (RAS flow = 2.3 mgd).

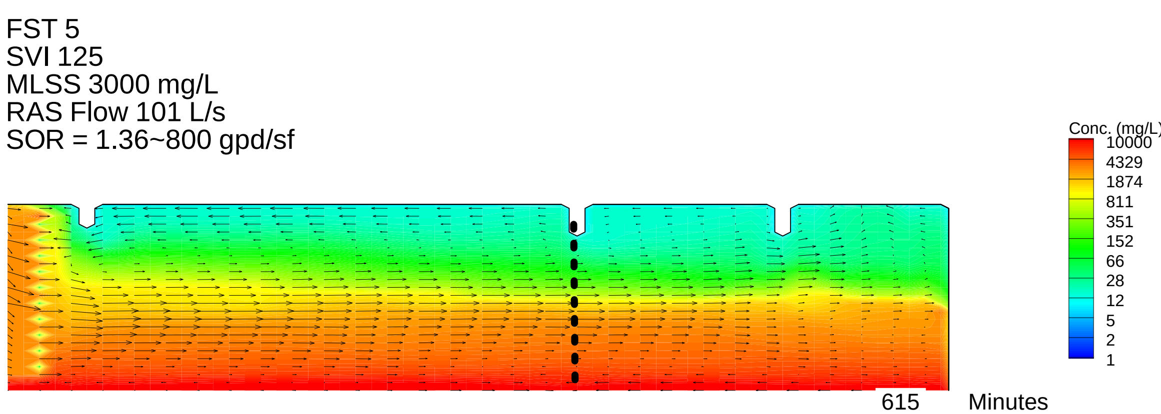


Figure 12. FST No. 5 near Failure with RAS flow = 101 L/s (2.3 mgd).

The capacity of FST No. 4 was found to vary with the RAS flow. Figures 13 and 14 show the results of the simulation with a RAS flow of 85 L/s (1.9 mgd)

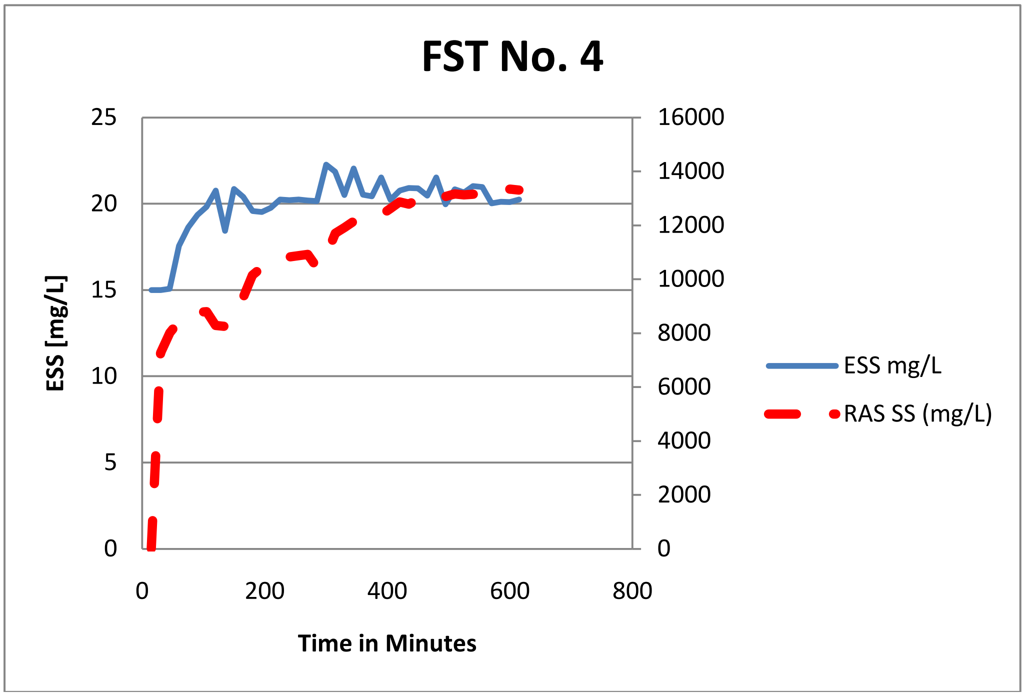


Figure 13. FST No. 4 near maximum capacity at SOR = 890 gpd/sf with RAS Flow = 85 L/s (1.9 mgd).

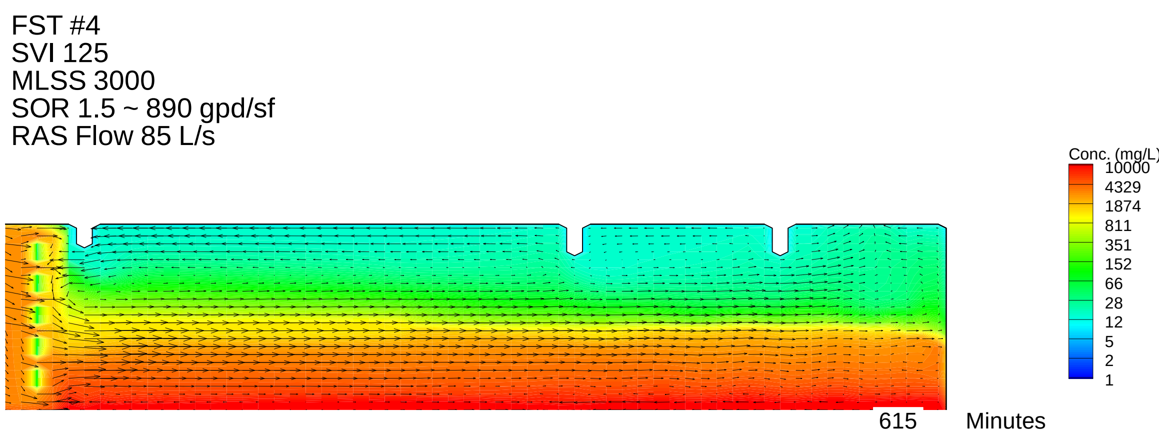


Figure 14. FST No. 4 near maximum capacity of 890 gpd/sf with SVI = 125 and MLSS = 3000 mg/L (RAS Flow = 1.9 mgd).

Conclusions

The calibrated and validated model was applied to FST No. 4 and FST No. 5 to estimate the capacity of the secondary clarifiers at 90-percentile SVI. The SVI was determined to be 125.

The Wahlberg-Keinath Equation was re-calibrated using the calibration and validation settling column data. The estimated V_o was 9.15 m/h and the K = 0.47.

The model was run for the equivalent of 10 h at various SORs and RAS flows to determine the points of failure. It was found that FST No. 4 could be operated for over 10 h at an SOR of 890 gpd/sf at a RAS flow of 1.9 mgd per clarifier. FST No. 5 started to fail at approximately 800 gpd/sf and a RAS flow of 2.3 mgd. In general, solids overflow started when the blanket exceeded 50% of the tank depth.

The tank capacities were sensitive to the RAS flow. It was also noted that the relatively short length of the launder resulted in an 'updraft' of solids that may have contributed to the failure.

References

Wahlberg, EJ, and Keinath, T.M. (1988b) *Development of settling flux curves using SVI: an addendum.* Water Environ Res. 67(5):872–4.

Wahlberg, EJ, and Keinath, TM. (1988a) *Development of settling flux curves using SVI.* J Water Pollut Control Fed. 60(12):2095–100.

Dimosthenis et al (2003) *Comparison and evaluation of empirical zone settling velocity parameters based on sludge volume index using a unified settling characteristics database*, Water Research 37, 3821–3836. See Table 2 below.

Table 2
Settleability tests and correlation-based equations proposed by different studies

Reference	Test	Number of data	Sludge Volume Index range (ml/g)	Correlation for V_o (m/h)	Correlation for K (m^3/Kg)	Abbreviation used in this study
Mines et al. [27]	SVI		27–236	7.27	0.0281 + 0.00229 SVI	V_0
Hartel and Pöpel [22]	SVI	—	—	$17.4e^{-0.0113\ SVI}$	$1.043\text{-}0.983e^{-0.00581 SVI}$	V_1
Daigger and Roper [13]	SVI	236	36–402	7.8	0.148 + 0.0021 SVI	V_2
Pitman [12]	SVI	697	45–360	$(V_o/k) = 37480\ e^{-0.00395 SVI}$	0.00088-0.000393 log [0.001x (Vo/k)]	V_3
Akca et al. [23]	SVI	—	—	$28.1\ SVI^{-0.2667}$	0.177 + 0.0014 SVI	V4
Wahlberg and Keinath [24]	SVI	185	48–235	$18.2\ e^{-0.00602\ SVI}$	0.351 + 0.00058 SVI	V_5
Daigger [18]	SVI	> 1500	36–402	6.5	0.165 + 0.001586 SVI	V_6
Ozinsky and Ekama [15]	SVI	—	—	$8.53094\ e^{-0.00165\ SVI}$	0.20036 + 0.00091 SVI	V_7
Wahlberg and Keinath [25]	SSVI	185	35–220	15.3-0.0615 SSVI	$0.426\text{-}0.00384\ SSVI + 5.43 \times 10^{-5}\ SSVI^2$	V_8
Wahlberg and Keinath [24]	SSVI	185	35–220	$24.3\ e^{-0.01073\ SSVI}$	0.245 + 0.00296 SSVI	V_9
Renko [26]				$V = 100\ v/SSVI\ e^{-nX}$		Renko 2
Bye and Dold [16]	Various	—	—	$V_t = (H_o - (H_o\ SVI\ MLSS)/1000)/t$	where H_o = column height and t = settling time	V_{10}

V, n parameters describing zone settling velocity. In order to facilitate the direct comparison, v was arbitrarily assigned as V_o and n as k. Their values were estimated from the respective V_o and k values of equations V_8 and V_9. It was observed that the emerging settling velocities have to be multiplied by 100 instead of 1000 to bring the estimated values within the dynamic measuring range of the settling tests. Alternatively, the estimation of these parameters, results in different values as proposed by Renko [26] which does not enable direct comparison with the other equations.

APPENDIX A

Calibration Results for No. 4

Simulations for the Calibration of FST No. 4

The FST No.4 was calibrated to reproduce the field data collected by Brown and Caldwell. The calibration involved adjusting the fraction (f3 and f4) of poorly flocculated solids in the influent. After 25 trials, the attached *Settling Properties* gave best results. Figure 1 shows the solids distribution near the end of the stress test. Figure 2 compares the ESS of the model and the field data. The model tended to over-estimate the blanket depth, RAS SS and ESS. The agreement with the composite data was better.

Calibration Results

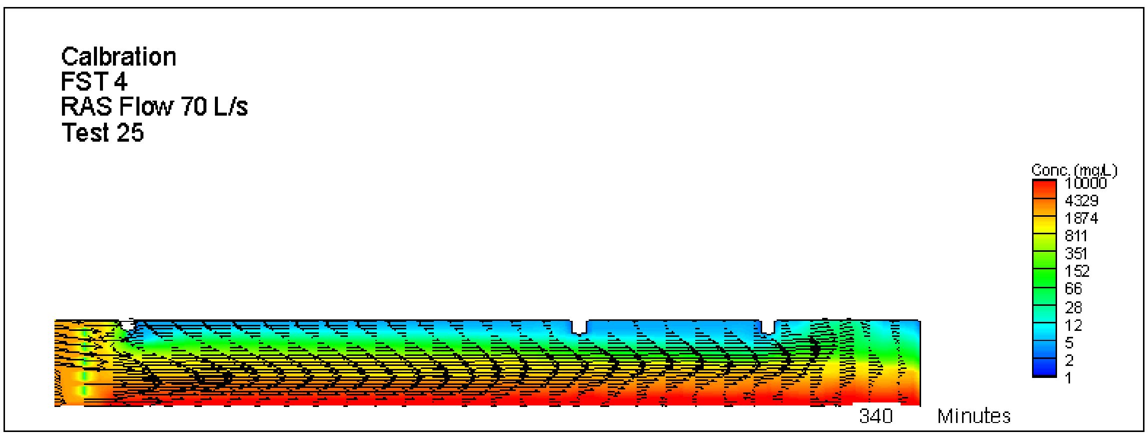


Figure 1. Solids Distribution Calibration of FST No. 4. Run 25.

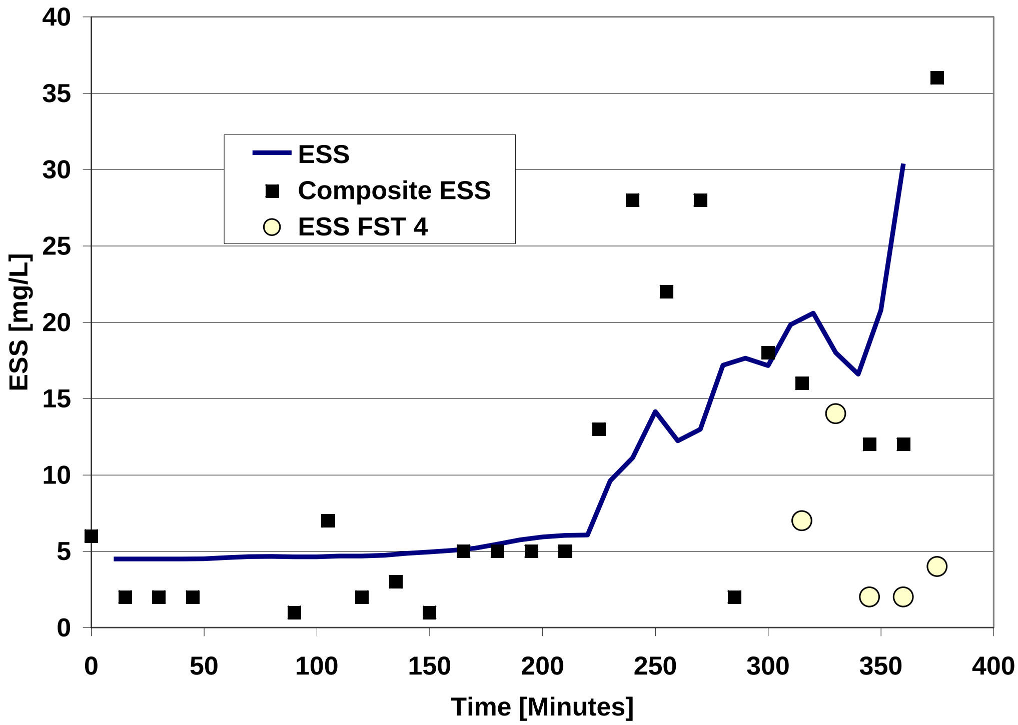


Figure 2. ESS Calibration of FST No. 4. Run 25.

Calibrated Inputs

Settling Properties

13. ! !Maximum Settling Velocity (m/h) = Vmax

0.39 ! !Floc Settling Parameter (m^3/Kg) = Fsp

15.0 !Colloids Settling Parameter (m^3/Kg) = Csp NOT USED

0.00020 !Concentration of nonsettling floc Kg/m3 = Cmin

7.9975 ! !Compression Settling velocity (m/h) = Vcom (Vo/2)

0.15675 ! !Compression Settling Parameter (m^3/Kg) = Kcom

1 !Is flocculation submodel used? Yes =1; No = 2

2.50 !Flocculation Constant for Differential Settling (turbulence)

4.E-8 !Flocculation Constant for Aggregation, Ka (L/g)

1.07E-8 !Flocculation Constant for Breakup, Kb (sec)

2.00 !Floc Breakup exponent

1200.0 !Threshold for hindered Settling, mg/L (Threshold = 0 when running Takacs Model)

800 ! !Threshold for discrete particle settling, mg/L (Threshold = 0 when running Takacs Model)

4 !Number of fractions for discrete particles, Limitation now = 5 (Make it equal to 1 when running Takacs)

0.80 ! !Fraction in class 1

21.50 ! Settling Velocity for Fraction in class 1 (m/h)

0.1925 !Fraction in class 2

13.75 ! Settling Velocity for Fraction in class 2 (m/h)

0.005 !Fraction in class 3

3.7755 ! !Settling Velocity for Fraction in class 3 (m/h)

0.002500 !Fraction in class 4

0.899502 !Settling Velocity for Fraction in class 4 (m/h)

0.000 !Fraction in class 5

-100.0 !Settling Velocity for Fraction in class 5 (m/h)

Geometry

53.00 !Length of the tank(m)= xl

15. !Width of the tank (m) = wz

3.07 !Depth of the tank (m)= hy

0.0 !1.0 Bottom slope (%)= Slope

0 !Porous or Solid Inlet Wall (Porous=0, Solid=1)

0.5 !Porosity of the Inlet Wall

2. ! !Inlet Depth

1 ! Modeling Skirt 1 (yes=1, no=0)'

5.0 ! Skirt 1 Distance from Inlet (m)

0.4 ! Skirt 1 Depth (m)

1 !Modeling Skirt 2 (yes=1, no=0)'

32.5 !Skirt 2 Distance from Inlet (m)

0.6 !Skirt 2 Depth (m)

1 !Modeling Skirt 3 (yes=1, no=0)'

44. !Skirt 3 Distance from Inlet (m)

0.6 !Skirt 3 Depth (m)

Time Series

t min	SOR m/h	MLSS g/L	ToC
0	0.57353655	1.8	20.9
15	0.72315478	2.1	20.9
30	0.946917156	2.2	20.9
45	0.828552511	2.3	20.9
60	0.769370189	3.0	20.9
75	0.769370189	2.1	20.9
90	0.739779028	2.1	20.9
105	0.739779028	1.7	20.9
120	1.109668542	2.3	20.9
135	1.109668542	2.5	20.9
150	1.154055284	2.2	20.9
165	1.109668542	2.3	20.9
180	1.154055284	1.9	20.9
195	1.33160225	2.9	20.9
210	1.420375734	2.8	20.9
225	3.107071918	2.6	20.9
240	3.284618884	2.4	20.9
255	3.284618884	3.2	20.9
270	3.284618884	3.2	20.9
285	3.195845401	2.2	20.9
300	3.462165851	2.1	20.9
315	3.373392368	1.9	20.9
330	3.195845401	2.5	20.9
345	3.284618884	2.1	20.9
360	3.284618884	1.8	20.9

Output

Time(min)	ESS	RSS SS	SOR m/s	SOR gpd/sf
10	4.5	4.5	0.57	336

20	4.5	25.75	0.72	424
30	4.5	5118.48	0.95	560
40	4.5	6615.55	0.95	560
50	4.51	8413.03	0.83	489
60	4.58	8322.4	0.77	454
70	4.65	8220.1	0.77	454
80	4.66	9142.07	0.77	454
90	4.64	9020.74	0.74	436
100	4.64	7880.04	0.74	436
110	4.68	7297.75	0.74	436
120	4.68	6985.49	1.11	654
130	4.74	6216.17	1.11	654
140	4.86	9058.31	1.11	654
150	4.95	9986.5	1.15	677
160	5.06	10697.81	1.15	677
170	5.2	10501.49	1.11	654
180	5.46	10297.08	1.15	677
190	5.74	10266.35	1.15	677
200	5.94	9648.02	1.33	783
210	6.04	10629.04	1.42	836
220	6.07	11489.02	1.42	836
230	9.62	11060.14	3.11	1832
240	11.12	11607.88	3.28	1932
250	14.14	14053.71	3.28	1932
260	12.23	16435.08	3.28	1932
270	12.99	16951.72	3.28	1932
280	17.19	17795.63	3.28	1932

290	17.65	18290.89	3.2	1885
300	17.16	18590.11	3.46	2038
310	19.84	18805.31	3.46	2038
320	20.6	18820.68	3.37	1985
330	18	18876.55	3.2	1885
340	16.6	18452.2	3.2	1885
350	20.78	18193.31	3.28	1932
360	30.39	18602.21	3.28	1932

Calibration Results for No. 5

Simulations for the Calibration of FST No. 5

The FST No.5 was calibrated to reproduce the field data collected by Brown and Caldwell. The calibration involved adjusting the fraction (f3 and f4) of poorly flocculated solids in the influent. After 25 trials, the attached *Settling Properties* gave best results. Figures 3a and 3b shows the solids distribution near the end of the stress test. Figure 4 compares the ESS of the model and the field data. Generally the blanket and ESS were in good agreement. The Results of FST No. 5 were used as guidelines for the calibration of FST No. 4.

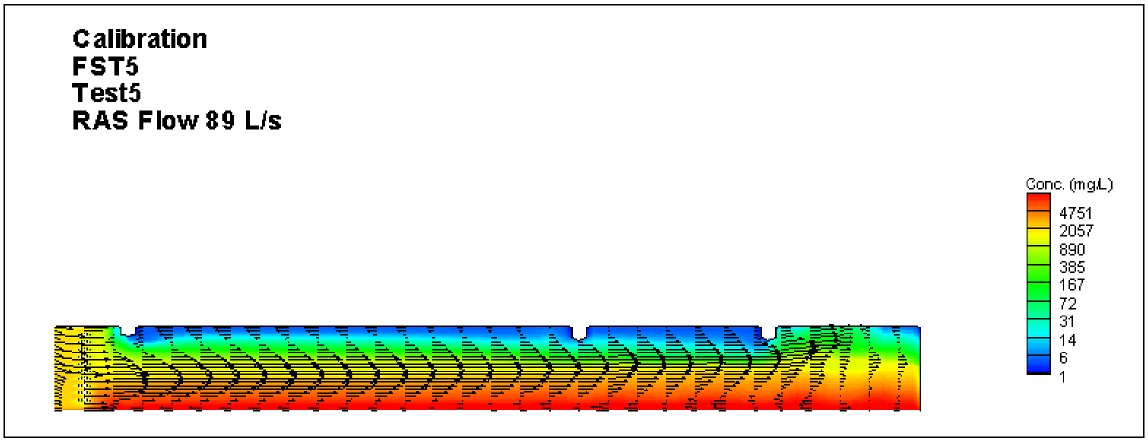


Figure 3. Solids Distribution for Calibration of FST No. 5.

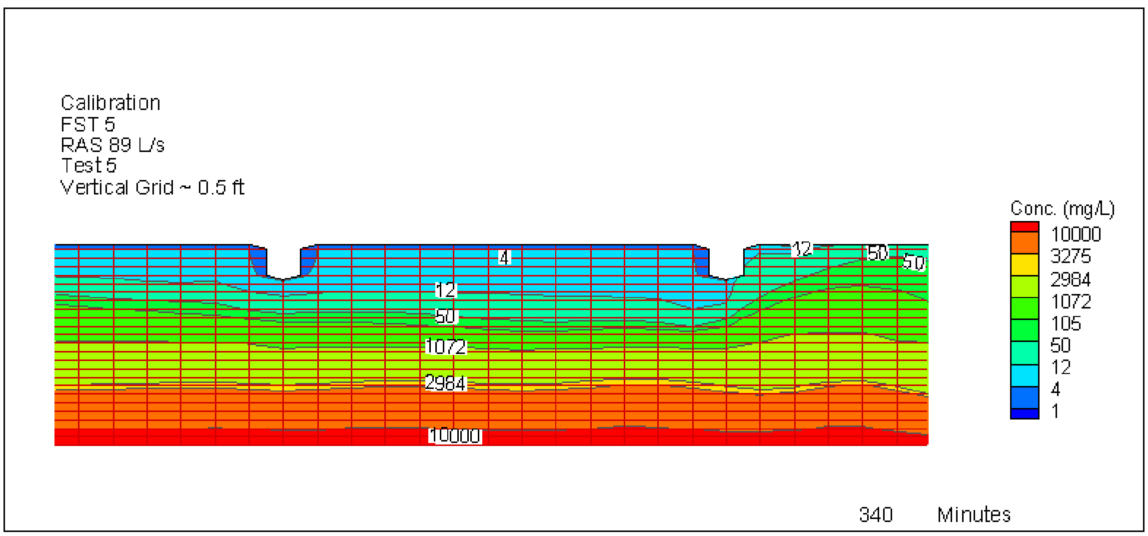


Figure 3b. Solids Distribution for Calibration of FST No. 5 showing the blanket.

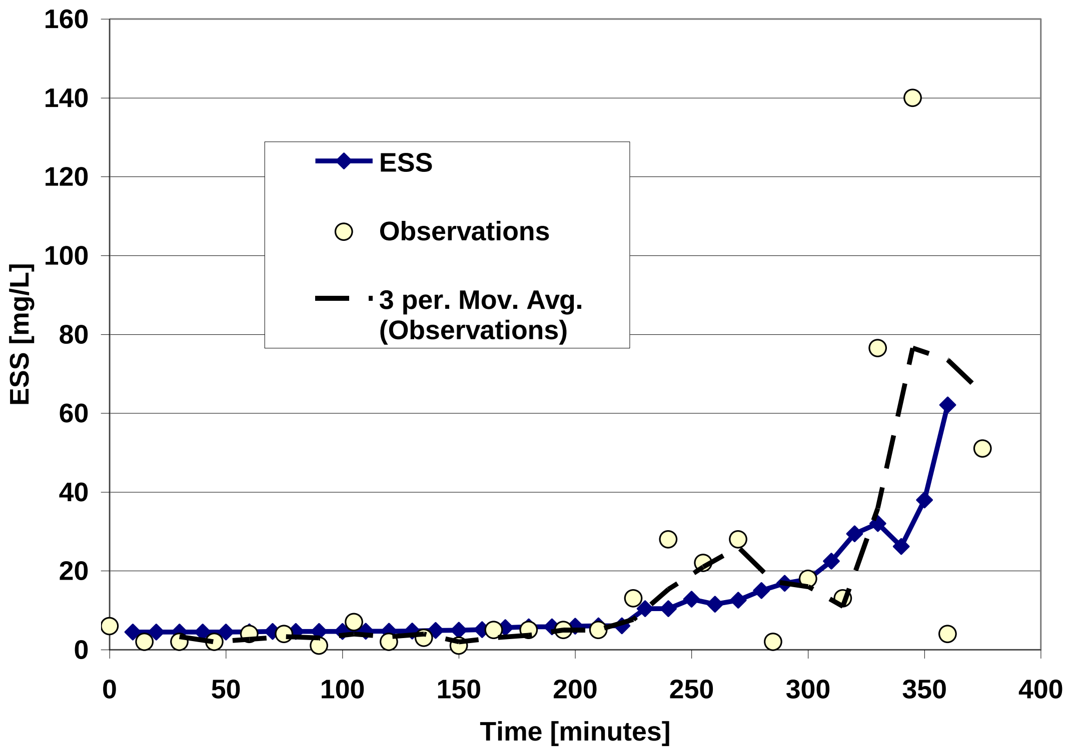


Figure 4. ESS Calibration of FST No. 5. Run 5

Calibrated Inputs

Settling Properties

13. !12.6 !13. !12.6 *!Maximum Settling Velocity (m/h) = Vo*

0.39 !!*K Floc Settling Parameter (m^3/Kg) = Fsp*

15.0 !Colloids Settling Parameter (m^3/Kg) = Csp NOT USED

0.00020 !*Concentration of nonsettling floc Kg/m3 = Cmin*

7.50975 ! *!Compression Settling velocity (m/h) = Vcom (Vo/2)*

0.175 ! !*Compression Settling Parameter (m^3/Kg) = Kcom*

1 !*Is flocculation submodel used? Yes =1; No = 2*

2.50 !*Flocculation Constant for Differential Settling (turbulence)*

4.E-8 !*Flocculation Constant for Aggregation, Ka (L/g)*

1.07E-8 !*Flocculation Constant for Breakup, Kb (sec)*

2.00 !*Floc Breakup exponent*

1200.0 *!Threshold for hindered Settling, mg/L (Threshold = 0*

when running Takacs Model for the complete settling curve)

800 ! *!Threshold for discrete particle settling, mg/L (Threshold = 0 when running Takacs Model for the*

complete settling curve)

4 !*Number of fractions for discrete particles, Limitation now = 5*

(Make it equal to 1 when running Takacs Model for the complete settling curve)

0.80 !80 !Fraction in class 1

21.50

0.1925

13.75 !5 !Settling Velocity for Fraction in class 2 (m/h)

0.005 !Fraction in class 3

3.7755 !2.5 !0.95 !Settling Velocity for Fraction in class 3 (m/h)

0.002500 !Fraction in class 4

0.899502 !Settling Velocity for Fraction in class 4 (m/h)

0.000 !Fraction in class 5

-100.0 !Settling Velocity for Fraction in class 5 (m/h)

Geometry

53.00 !Length of the tank(m)= xl

19. !Width of the tank (m) = wz

3.07 !Depth of the tank (m)= hy

0.0 !1.0 !1.0 !-4.0 !Bottom slope (%)= Slope

0 !Porous or Solid Inlet Wall (Porous=0, Solid=1)

0.5 !Porosity of the Inlet Wall

2. !2.25 !3.95 !.0 !Inlet Depth

1 !Modeling Skirt 1 (yes=1, no=0)'

5.0 !20. !4.0 !Skirt 1 Distance from Inlet (m)

0.4 !1.2 !Skirt 1 Depth (m)

1 !Modeling Skirt 2 (yes=1, no=0)'

32.5 !Skirt 2 Distance from Inlet (m)

0.6 !Skirt 2 Depth (m)

1 !Modeling Skirt 3 (yes=1, no=0)'

44. !Skirt 3 Distance from Inlet (m)

0.6 !Skirt 3 Depth (m)

Time Series

t min	SOR m/h	MLSS g/L	ToC
0	0.57353655	1.8	20.9
15	0.72315478	2.1	20.9
30	0.946917156	2.2	20.9
45	0.828552511	2.3	20.9
60	0.769370189	3.0	20.9
75	0.769370189	2.1	20.9
90	0.739779028	2.1	20.9
105	0.739779028	1.7	20.9
120	1.109668542	2.3	20.9
135	1.109668542	2.5	20.9
150	1.154055284	2.2	20.9
165	1.109668542	2.3	20.9
180	1.154055284	1.9	20.9
195	1.33160225	2.9	20.9
210	1.420375734	2.8	20.9
225	3.107071918	2.6	20.9
240	3.284618884	2.4	20.9
255	3.284618884	3.2	20.9
270	3.284618884	3.2	20.9
285	3.195845401	2.2	20.9

300	3.462165851	2.1	20.9
315	3.373392368	1.9	20.9
330	3.195845401	2.5	20.9
345	3.284618884	2.1	20.9
360	3.284618884	1.8	20.9

Typical Output

Time(min)	ESS	RSS SS	SOR m/s	SOR gpd/sf
10	4.5	5	0.57	336
20	4.5	27	0.72	424
30	4.5	5312	0.95	560
40	4.5	6769	0.95	560
50	4.5	8363	0.83	489
60	4.6	8392	0.77	454
70	4.6	8286	0.77	454
80	4.7	9425	0.77	454
90	4.6	8916	0.74	436
100	4.6	7772	0.74	436
110	4.7	7172	0.74	436
120	4.7	6805	1.11	654
130	4.8	6051	1.11	654
140	4.9	9246	1.11	654
150	5.0	10094	1.15	677
160	5.1	10466	1.15	677
170	5.6	10313	1.11	654
180	5.8	10210	1.15	677
190	5.8	10255	1.15	677
200	6.0	9564	1.33	783

210	6.1	10608	1.42	836
220	6.1	11317	1.42	836
230	10.4	10695	3.11	1832
240	10.4	11105	3.28	1932
250	12.8	13446	3.28	1932
260	11.5	15839	3.28	1932
270	12.5	16126	3.28	1932
280	15.0	16838	3.28	1932
290	16.8	17306	3.2	1885
300	17.9	17259	3.46	2038
310	22.5	17874	3.46	2038
320	29.4	17738	3.37	1985
330	32.0	17963	3.2	1885
340	26.2	17691	3.2	1885
350	38.0	17000	3.28	1932
360	62.1	17445	3.28	1932

APPENDIX C

WPCP HYDRAULIC MODEL CALIBRATION FILES

Oneida WPCP
PROFILE Model Calibration
DWF - 33 MGD

Location		TOC El.	Measured Depth from TOC to WSE	WSE (ft)	
Process	Description	(ft)	(ft)	Measured	Model
Grit Tank 1	Influent Channel	426	3.50	422.50	422.43
	Inside Tank	426	3.63	422.38	422.40
	Effluent Channel	426	7.13	418.88	418.73
Prim. Dist. Box	Inside	420.5	2.69	417.81	417.84
	Effluent	420.5	4.63	415.88	415.85
Prim. Tank 1	Inside Tank	418	2.42	415.58	415.46
	Effluent Channel	418	5.17	412.83	412.68
Bypass Chamber	In Box	415.5	4.29	411.21	411.20
Aeration Tank 1	Influent Channel	413.5	2.42	411.08	411.08
	Inside Tank	413.5	3.00	410.50	410.52
	Effluent Channel	413.5	4.46	409.04	408.71
Aeration Tank 3	Influent channel (within tank)	413.5	3.04	410.46	410.46
	Inside Tank	413.5	3.33	410.17	410.19
	Effluent Channel	413.5	4.71	408.79	408.65
Final Tank 8	Influent Channel	412.5	3.83	408.67	408.59
	Inside Tank (downstream end)	412.5	4.00	408.50	408.42
	Effluent Channel	412.5	5.08	407.42	407.53
Final Tank 1	Influent Cahnnel	412.5	3.75	408.75	408.59
	Inside Tank (downstream end)	412.5	4.04	408.46	408.45
	Effluent Channel	412.5	5.13	407.38	407.47
CCT	Infuent Channel	412.5	5.17	407.33	407.45
	Inside Tank (upstream end)	411.5	4.25	407.25	407.40
	Inside Tank (downstream end)	411.5	4.42	407.08	407.39
	Effluent Channel	411.5	5.42	406.08	406.10
	Flume Channel (upstream)	411.5	5.58	405.92	405.84
	Flume Channel (at flume)	411.5	5.71	405.79	405.79
	Flume Channel (after flume and drop)	411.5	12.08	399.42	399.52

Oneida WPCP
PROFILE Model Calibration
WWF - 54 MGD

Location		TOC El.	Measured Depth from TOC to WSE	WSE (ft)	
Process	Description	(ft)	(ft)	Measured	Model
Grit Tank 1	Influent Channel	426	3.33	422.67	422.64
	Inside Tank	426	3.50	422.50	422.55
	Effluent Channel	426	6.04	419.96	419.76
Prim. Dist. Box	Inside	420.5	2.33	418.17	418.03
	Effluent	420.5	3.92	416.58	416.20
Prim. Tank 1	Inside Tank	418	2.33	415.67	415.49
	Effluent Channel	418	4.92	413.08	412.95
Bypass Chamber	In Box	415.5	3.58	411.92	411.98
Aeration Tank 1	Influent Channel	413.5	1.92	411.58	411.59
	Inside Tank	413.5	2.83	410.67	410.66
	Effluent Channel	413.5	4.04	409.46	409.35
Aeration Tank 3	Influent channel (within tank)	413.5	2.71	410.79	410.70
	Inside Tank	413.5	3.25	410.25	410.25
	Effluent Channel	413.5	4.50	409.00	408.99
Final Tank 8	Influent Channel	412.5	3.75	408.75	408.87
	Inside Tank (downstream end)	412.5	3.96	408.54	408.45
	Effluent Channel	412.5	4.71	407.79	407.89
Final Tank 1	Influent Cahnnel	412.5	3.58	408.92	408.87
	Inside Tank (downstream end)	412.5	4.04	408.46	408.49
	Effluent Channel	412.5	4.83	407.67	407.74
CCT	Infuent Channel	412.5	4.83	407.67	407.68
	Inside Tank (upstream end)	411.5	4.00	407.50	407.55
	Inside Tank (downstream end)	411.5	4.25	407.25	407.55
	Effluent Channel	411.5	4.79	406.71	406.83
	Flume Channel (upstream)	411.5	5.17	406.33	406.53
	Flume Channel (at flume)	411.5	5.21	406.29	406.26
	Flume Channel (after flume and drop)	411.5	11.08	400.42	400.29

```
BBBBB      CCC      BROWN AND CALDWELL
BBBBBB    CCCCC     Consulting Engineers
BB  BBB  CCC CCC
BB   BB  CC   CC    PROFILE        SERIAL NO. 9709
BB  BBB  CC   CC    Version 2.00
BBBBBB   CC
BBBBB    CC         File name:
BBBBBB   CC         Data file:
BB  BBB  CC   CC    Utica WPCP - all tanks online, split flow paths for
BB   BB  CC   CC    outfall, FSTs, and aeration tanks.
BB  BBB  CC   CC    Oneida County
BBBBBB    CCCCC
BBBBB      CCC      11-2007             By:Dan Gilbert


PLANT FLOW =     51.06 CFS OR     33.00 MGD

DOWNSTREAM CONDITIONS:
    ENERGY GRADE =       413.70 FEET
    HYDRAULIC GRADE =    413.70 FEET

NUMBER OF UNITS IN SERVICE:

 1         2     Grit Tanks
 2         4     Primaries
 3         3     Aeration
 4         8     Finals
 5         2     Disinfection


**10
**Total Flow (100%, 54MGD)
 FLOW PERCENT
    FLOW =    51.06 CFS OR     33.00 MGD
          100.00 PERCENT OF TOTAL PLANT FLOW.

**1180
** subtract flow from primary tank 3&4 (50%)
 FLOW PERCENT
    FLOW =    25.53 CFS OR     16.50 MGD
           50.00 PERCENT OF FLOW THROUGH ADJACENT DOWNSTREAM ELEMENT.

**1220
**subtrac flow to tank 2 (60%)
 FLOW PERCENT
    FLOW =    15.32 CFS OR      9.90 MGD
           60.00 PERCENT OF FLOW THROUGH ADJACENT DOWNSTREAM ELEMENT.

BROWN AND CALDWELL         PROFILE        SERIAL NO. 9709
Consulting Engineers       Version 2.00

1290
primary tank 1 eff weir
V-NOTCH WEIR PLATE
    WEIR: DISCHARGE =     15.32 CFS
          LENGTH =    329.00 FEET
          TOP OF PLATE ELEV =   415.580
    V-NOTCH: SPACING =  8.00 INCHES
             ANGLE =   90.00 DEGREES
             DEPTH =    3.38 INCHES
             INVERT =   415.298
    WS ELEV DOWNSTREAM OF WEIR =   413.700
```

```
    FREEBOARD =  1.598
    ENERGY LOSS, FEET =   1.771
    ENERGY GRADE =      415.471
    HYDRAULIC GRADE =   415.471

1300
exit loss to primary tank 1
"K"LOSS IN FULL ROUND PIPE
    LOSS COEFFICIENT K =   1.000
    PIPE DIAMETER =        36.00 INCHES
    INVERT ELEVATION =   401.300

    VELOCITY =    2.17 FT/SEC
    ENERGY LOSS, FEET =    .073
    ENERGY GRADE =      415.544
    HYDRAULIC GRADE =   415.471

1310
90 deg turn
"K"LOSS IN FULL ROUND PIPE
    LOSS COEFFICIENT K =   1.500
    PIPE DIAMETER =        36.00 INCHES
    INVERT ELEVATION =   395.500

    VELOCITY =    2.17 FT/SEC
    ENERGY LOSS, FEET =    .109
    ENERGY GRADE =      415.653
    HYDRAULIC GRADE =   415.581

BROWN AND CALDWELL         PROFILE        SERIAL NO. 9709
Consulting Engineers       Version 2.00

1320
primary tanl 1 inf pipe
ROUND CONDUIT
    DIAMETER =  36.00 INCHES
    LENGTH=    130.00 FEET
    MANNING ROUGHNESS =   .0150
    SLOPE =   .00000 FEET/FOOT
    NUMBER OF ANALYSIS SECTIONS =   13.00
    INVERT ELEV AT OUTLET =   395.500


    SUBCRITICAL FLOW
    CONDUIT OUTLET SUBMERGED
    FULL CONDUIT FLOW THROUGHOUT LENGTH
    FRICTION FACTOR =  .00070 FT/FT
    VELOCITY =   2.2 FT/SEC
    CRITICAL SLOPE, FT/FT = .0059
    CRITICAL DEPTH, FEET =   1.21
    CHANGE IN HYDRAULIC GRADE WITHIN CONDUIT, FEET =  .091
    ENERGY LOSS, FEET =   .091

    INLET CONDITIONS:
       ENERGY GRADE =      415.745
       HYDRAULIC GRADE =   415.672

1330
entrance loss to primary tank inf pipe ( at distribution chamber)
"K"LOSS IN FULL ROUND PIPE
    LOSS COEFFICIENT K =   1.000
    PIPE DIAMETER =        36.00 INCHES
    INVERT ELEVATION =   400.500
```

```
    VELOCITY =    2.17 FT/SEC
    ENERGY LOSS, FEET =    .073
    ENERGY GRADE =      415.818
    HYDRAULIC GRADE =   415.745

1340
90 deg turn into pipe
"K"LOSS IN FULL ROUND PIPE
    LOSS COEFFICIENT K =   1.500
    PIPE DIAMETER =        36.00 INCHES
    INVERT ELEVATION =   400.500

    VELOCITY =    2.17 FT/SEC
    ENERGY LOSS, FEET =    .109
    ENERGY GRADE =      415.927
    HYDRAULIC GRADE =   415.854

BROWN AND CALDWELL          PROFILE        SERIAL NO. 9709
Consulting Engineers        Version 2.00

**1350
**split flow between two opening in dist box  (50%)
 FLOW PERCENT
    FLOW =     7.66 CFS OR      4.95 MGD
            50.00 PERCENT OF FLOW THROUGH ADJACENT DOWNSTREAM ELEMENT.

1360
weir in dist box
SHARP-CRESTED WEIR
    WEIR CREST ELEVATION =   417.000
    WEIR DISCHARGE =     7.66 CFS
    LENGTH =     3.00 FEET

    NO END CONTRACTIONS
    FREEBOARD =  1.146
    CALCULATED C VALUE =  3.332
    HEIGHT OF WATER OVER WEIR =   .837
    ENERGY LOSS, FEET =   1.910
    ENERGY GRADE =      417.837
    HYDRAULIC GRADE =   417.837

**1380
**add flow from other opeing to primary tank 1 (200%)
 FLOW PERCENT
    FLOW =    15.32 CFS OR      9.90 MGD
           200.00 PERCENT OF FLOW THROUGH ADJACENT DOWNSTREAM ELEMENT.

BROWN AND CALDWELL          PROFILE        SERIAL NO. 9709
Consulting Engineers        Version 2.00

**1390
**add flow to tanks 2 (166.67%)
 FLOW PERCENT
    FLOW =    25.53 CFS OR     16.50 MGD
           166.67 PERCENT OF FLOW THROUGH ADJACENT DOWNSTREAM ELEMENT.

**1395
**add flow to tanks 3-4 (200%)
 FLOW PERCENT
    FLOW =    51.06 CFS OR     33.00 MGD
           200.00 PERCENT OF FLOW THROUGH ADJACENT DOWNSTREAM ELEMENT.
```

```
1400
exit loss into dist chamber
"K"LOSS IN FULL ROUND PIPE
    LOSS COEFFICIENT K =   1.000
    PIPE DIAMETER =        48.00 INCHES
    INVERT ELEVATION =   400.000

    VELOCITY =    4.06 FT/SEC
    ENERGY LOSS, FEET =    .256
    ENERGY GRADE =      418.094
    HYDRAULIC GRADE =   417.837

BROWN AND CALDWELL         PROFILE        SERIAL NO. 9709
Consulting Engineers       Version 2.00

1410
contraction 60 to 48"
"K"LOSS IN FULL ROUND PIPE
    LOSS COEFFICIENT K =    .190
    PIPE DIAMETER =        60.00 INCHES
    INVERT ELEVATION =   400.500

    VELOCITY =    2.60 FT/SEC
    ENERGY LOSS, FEET =    .020
    ENERGY GRADE =      418.114
    HYDRAULIC GRADE =   418.009

1420
grit eff pipe to dist box
ROUND CONDUIT
    DIAMETER =  60.00 INCHES
    LENGTH=     82.00 FEET
    MANNING ROUGHNESS =   .0150
    SLOPE =   .02439 FEET/FOOT
    NUMBER OF ANALYSIS SECTIONS =   10.00
    INVERT ELEV AT OUTLET =   400.500


    SUPERCRITICAL CHANNEL
***WARNING:CONDUIT HAS NOT BEEN DEBUGGED AND VALIDATED FOR SUPERCRITICAL CASE!

    CONDUIT OUTLET SUBMERGED
    FULL CONDUIT FLOW THROUGHOUT LENGTH
    FRICTION FACTOR =  .00051 FT/FT
    VELOCITY =   2.6 FT/SEC
    CRITICAL SLOPE, FT/FT = .0050
    CRITICAL DEPTH, FEET =   1.95
    NORMAL DEPTH,   FEET =   1.29
    CHANGE IN HYDRAULIC GRADE WITHIN CONDUIT, FEET =  .042
    ENERGY LOSS, FEET =   .042

    INLET CONDITIONS:
       ENERGY GRADE =      418.156
       HYDRAULIC GRADE =   418.051

1430
entrance loss to grit eff pipe
"K"LOSS IN FULL ROUND PIPE
    LOSS COEFFICIENT K =   1.250
    PIPE DIAMETER =        60.00 INCHES
    INVERT ELEVATION =   402.500

    VELOCITY =    2.60 FT/SEC
```

```
    ENERGY LOSS, FEET =    .131
    ENERGY GRADE =      418.287
    HYDRAULIC GRADE =   418.182
```



```
1440
90 vert turn down grit eff manhole
"K" LOSS IN RECTANGULAR OPEN CHANNEL
    WIDTH =   6.00 FEET
    INVERT ELEV. =   416.500 FEET
    SIDEWALL =   9.50 FEET
    LOSS COEFFICIENT "K" =   1.50

    VELOCITY =    4.36 FT/SEC
    ENERGY LOSS, FEET =    .444
    ENERGY GRADE =      418.731
    HYDRAULIC GRADE =   418.435

**1450
** split flow between grit tanks (50%)
 FLOW PERCENT
    FLOW =    25.53 CFS OR     16.50 MGD
            50.00 PERCENT OF FLOW THROUGH ADJACENT DOWNSTREAM ELEMENT.

1460
90 deg turn from tank 1 eff channel to combined eff channel
"K" LOSS IN RECTANGULAR OPEN CHANNEL
    WIDTH =   6.00 FEET
    INVERT ELEV. =   416.500 FEET
    SIDEWALL =   6.50 FEET
    LOSS COEFFICIENT "K" =   1.00

    VELOCITY =    1.91 FT/SEC
    ENERGY LOSS, FEET =    .056
    ENERGY GRADE =      418.787
    HYDRAULIC GRADE =   418.731
```



```
1470
grit tank 1 eff weir
SHARP-CRESTED WEIR
    WEIR CREST ELEVATION =   422.000
    WEIR DISCHARGE =    25.53 CFS
    LENGTH =    30.00 FEET

    NO END CONTRACTIONS
    FREEBOARD =  3.269
    CALCULATED C VALUE =  3.350
    HEIGHT OF WATER OVER WEIR =   .401
    ENERGY LOSS, FEET =   3.614
    ENERGY GRADE =      422.401
    HYDRAULIC GRADE =   422.401

1480
grit tank 1
RECTANGULAR CONDUIT
    HEIGHT =   6.25 FEET
    WIDTH =  30.00 FEET
    LENGTH=     30.00 FEET
```

MANNING ROUGHNESS = .0150
SLOPE = .00000 FEET/FOOT
NUMBER OF ANALYSIS SECTIONS = 10.00
INVERT ELEV AT OUTLET = 419.750

SUBCRITICAL FLOW

STATION FEET	WATER DEPTH FEET	VELOCITY FT/SEC	FRICTION FACTOR FT/FOOT	AVERAGE FRICTION FACTOR FT/FOOT	FRICTION LOSS FEET	HYDRAULIC GRADE	ENERGY GRADE
.000	2.650	.321	.00000			422.400	422.401
3.000	2.650	.321	.00000	.00000	.000	422.400	422.401
6.000	2.650	.321	.00000	.00000	.000	422.400	422.401
9.000	2.650	.321	.00000	.00000	.000	422.400	422.401
12.000	2.650	.321	.00000	.00000	.000	422.400	422.401
15.000	2.650	.321	.00000	.00000	.000	422.400	422.401
18.000	2.650	.321	.00000	.00000	.000	422.400	422.401
21.000	2.650	.321	.00000	.00000	.000	422.400	422.401
24.000	2.650	.321	.00000	.00000	.000	422.400	422.401
27.000	2.650	.321	.00000	.00000	.000	422.400	422.401
30.000	2.650	.321	.00000	.00000	.000	422.400	422.401

CRITICAL SLOPE, FT/FT = .0051
CRITICAL DEPTH, FEET = .28
CHANGE IN HYDRAULIC GRADE WITHIN CONDUIT, FEET = .000
ENERGY LOSS, FEET = .000

INLET CONDITIONS:
ENERGY GRADE = 422.401
HYDRAULIC GRADE = 422.400

1500
Grit Tank 1 Inf gates
SUBMERGED RECTANGULAR ORIFICE
NO OF ORIFICES = 5
ORIFICE HEIGHT = 1.70 FEET
ORIFICE WIDTH = 3.50 FEET
DISCHARGE COEFFICIENT = .600

FLOW PER ORIFICE = 5.11 CFS
VELOCITY THROUGH ORIFICE, FPS = .86
ENERGY LOSS, FEET = .032
ENERGY GRADE = 422.433
HYDRAULIC GRADE = 422.433

BROWN AND CALDWELL PROFILE SERIAL NO. 9709
Consulting Engineers Version 2.00

1510
90 deg turn in tank 1 inf channel to last gate opening
"K" LOSS IN RECTANGULAR OPEN CHANNEL
WIDTH = 6.00 FEET
INVERT ELEV. = 420.500 FEET
SIDEWALL = 5.50 FEET
LOSS COEFFICIENT "K" = 1.00

VELOCITY = 2.20 FT/SEC
ENERGY LOSS, FEET = .075
ENERGY GRADE = 422.508

HYDRAULIC GRADE =   422.433

```
BBBBB      CCC      BROWN AND CALDWELL
BBBBBB    CCCCC     Consulting Engineers
BB  BBB  CCC CCC
BB   BB  CC   CC    PROFILE        SERIAL NO. 9709
BB  BBB  CC   CC    Version 2.00
BBBBBB   CC
BBBBB    CC         File name:
BBBBBB   CC         Data file:
BB  BBB  CC   CC    Utica WPCP - all tanks online, split flow paths for
BB   BB  CC   CC    outfall, FSTs, and aeration tanks.
BB  BBB  CC   CC    Oneida County
BBBBBB    CCCCC
BBBBB      CCC      11-2007              By:Dan Gilbert


PLANT FLOW =     83.55 CFS OR     54.00 MGD

DOWNSTREAM CONDITIONS:
    ENERGY GRADE =       413.70 FEET
    HYDRAULIC GRADE =    413.70 FEET

NUMBER OF UNITS IN SERVICE:

 1         2     Grit Tanks
 2         4     Primaries
 3         3     Aeration
 4         8     Finals
 5         2     Disinfection


**10
**Total Flow (100%, 54MGD)
 FLOW PERCENT
    FLOW =    83.55 CFS OR     54.00 MGD
           100.00 PERCENT OF TOTAL PLANT FLOW.

**1180
** subtract flow from primary tank 3&4 (50%)
 FLOW PERCENT
    FLOW =    41.77 CFS OR     27.00 MGD
            50.00 PERCENT OF FLOW THROUGH ADJACENT DOWNSTREAM ELEMENT.

**1220
**subtrac flow to tank 2 (50%)
 FLOW PERCENT
    FLOW =    20.89 CFS OR     13.50 MGD
            50.00 PERCENT OF FLOW THROUGH ADJACENT DOWNSTREAM ELEMENT.

BROWN AND CALDWELL         PROFILE        SERIAL NO. 9709
Consulting Engineers       Version 2.00

1290
primary tank 1 eff weir
V-NOTCH WEIR PLATE
    WEIR: DISCHARGE =     20.89 CFS
          LENGTH =    329.00 FEET
          TOP OF PLATE ELEV =   415.580
    V-NOTCH: SPACING =  8.00 INCHES
             ANGLE =   90.00 DEGREES
             DEPTH =    3.38 INCHES
             INVERT =   415.298
    WS ELEV DOWNSTREAM OF WEIR =   413.700
```

```
    FREEBOARD =  1.598
    ENERGY LOSS, FEET =   1.794
    ENERGY GRADE =      415.494
    HYDRAULIC GRADE =   415.494

1300
exit loss to primary tank 1
"K"LOSS IN FULL ROUND PIPE
    LOSS COEFFICIENT K =   1.000
    PIPE DIAMETER =        36.00 INCHES
    INVERT ELEVATION =   401.300

    VELOCITY =    2.95 FT/SEC
    ENERGY LOSS, FEET =    .136
    ENERGY GRADE =      415.630
    HYDRAULIC GRADE =   415.494

1310
90 deg turn
"K"LOSS IN FULL ROUND PIPE
    LOSS COEFFICIENT K =   1.500
    PIPE DIAMETER =        36.00 INCHES
    INVERT ELEVATION =   395.500

    VELOCITY =    2.95 FT/SEC
    ENERGY LOSS, FEET =    .203
    ENERGY GRADE =      415.833
    HYDRAULIC GRADE =   415.697
```



```
1320
primary tanl 1 inf pipe
ROUND CONDUIT
    DIAMETER =  36.00 INCHES
    LENGTH=    130.00 FEET
    MANNING ROUGHNESS =   .0150
    SLOPE =   .00000 FEET/FOOT
    NUMBER OF ANALYSIS SECTIONS =   13.00
    INVERT ELEV AT OUTLET =   395.500


    SUBCRITICAL FLOW
    CONDUIT OUTLET SUBMERGED
    FULL CONDUIT FLOW THROUGHOUT LENGTH
    FRICTION FACTOR =  .00131 FT/FT
    VELOCITY =   3.0 FT/SEC
    CRITICAL SLOPE, FT/FT = .0061
    CRITICAL DEPTH, FEET =   1.43
    CHANGE IN HYDRAULIC GRADE WITHIN CONDUIT, FEET =  .170
    ENERGY LOSS, FEET =   .170

    INLET CONDITIONS:
       ENERGY GRADE =      416.003
       HYDRAULIC GRADE =   415.867

1330
entrance loss to primary tank inf pipe ( at distribution chamber)
"K"LOSS IN FULL ROUND PIPE
    LOSS COEFFICIENT K =   1.000
    PIPE DIAMETER =        36.00 INCHES
    INVERT ELEVATION =   400.500
```

```
    VELOCITY =    2.95 FT/SEC
    ENERGY LOSS, FEET =    .136
    ENERGY GRADE =      416.138
    HYDRAULIC GRADE =   416.003

1340
90 deg turn into pipe
"K"LOSS IN FULL ROUND PIPE
    LOSS COEFFICIENT K =   1.500
    PIPE DIAMETER =        36.00 INCHES
    INVERT ELEVATION =   400.500

    VELOCITY =    2.95 FT/SEC
    ENERGY LOSS, FEET =    .203
    ENERGY GRADE =      416.342
    HYDRAULIC GRADE =   416.206

BROWN AND CALDWELL          PROFILE        SERIAL NO. 9709
Consulting Engineers        Version 2.00

**1350
**split flow between two opening in dist box  (50%)
 FLOW PERCENT
    FLOW =    10.44 CFS OR      6.75 MGD
            50.00 PERCENT OF FLOW THROUGH ADJACENT DOWNSTREAM ELEMENT.

1360
weir in dist box
SHARP-CRESTED WEIR
    WEIR CREST ELEVATION =   417.000
    WEIR DISCHARGE =    10.44 CFS
    LENGTH =     3.00 FEET

    NO END CONTRACTIONS
    FREEBOARD =   .794
    CALCULATED C VALUE =  3.333
    HEIGHT OF WATER OVER WEIR =  1.029
    ENERGY LOSS, FEET =   1.688
    ENERGY GRADE =      418.029
    HYDRAULIC GRADE =   418.029

**1380
**add flow from other opeing to primary tank 1 (200%)
 FLOW PERCENT
    FLOW =    20.89 CFS OR     13.50 MGD
           200.00 PERCENT OF FLOW THROUGH ADJACENT DOWNSTREAM ELEMENT.

BROWN AND CALDWELL          PROFILE        SERIAL NO. 9709
Consulting Engineers        Version 2.00

**1390
**add flow to tanks 2 (200%)
 FLOW PERCENT
    FLOW =    41.77 CFS OR     27.00 MGD
           200.00 PERCENT OF FLOW THROUGH ADJACENT DOWNSTREAM ELEMENT.

**1395
**add flow to tanks 3-4 (200%)
 FLOW PERCENT
    FLOW =    83.55 CFS OR     54.00 MGD
           200.00 PERCENT OF FLOW THROUGH ADJACENT DOWNSTREAM ELEMENT.
```

```
1400
exit loss into dist chamber
"K"LOSS IN FULL ROUND PIPE
    LOSS COEFFICIENT K =   1.000
    PIPE DIAMETER =        48.00 INCHES
    INVERT ELEVATION =   400.000

    VELOCITY =    6.65 FT/SEC
    ENERGY LOSS, FEET =    .686
    ENERGY GRADE =      418.716
    HYDRAULIC GRADE =   418.029

BROWN AND CALDWELL         PROFILE        SERIAL NO. 9709
Consulting Engineers       Version 2.00

1410
contraction 60 to 48"
"K"LOSS IN FULL ROUND PIPE
    LOSS COEFFICIENT K =    .190
    PIPE DIAMETER =        60.00 INCHES
    INVERT ELEVATION =   400.500

    VELOCITY =    4.26 FT/SEC
    ENERGY LOSS, FEET =    .053
    ENERGY GRADE =      418.769
    HYDRAULIC GRADE =   418.488

1420
grit eff pipe to dist box
ROUND CONDUIT
    DIAMETER =  60.00 INCHES
    LENGTH=     82.00 FEET
    MANNING ROUGHNESS =   .0150
    SLOPE =   .02439 FEET/FOOT
    NUMBER OF ANALYSIS SECTIONS =   10.00
    INVERT ELEV AT OUTLET =   400.500


    SUPERCRITICAL CHANNEL
***WARNING:CONDUIT HAS NOT BEEN DEBUGGED AND VALIDATED FOR SUPERCRITICAL CASE!

    CONDUIT OUTLET SUBMERGED
    FULL CONDUIT FLOW THROUGHOUT LENGTH
    FRICTION FACTOR =  .00137 FT/FT
    VELOCITY =   4.3 FT/SEC
    CRITICAL SLOPE, FT/FT = .0052
    CRITICAL DEPTH, FEET =   2.54
    NORMAL DEPTH,   FEET =   1.66
    CHANGE IN HYDRAULIC GRADE WITHIN CONDUIT, FEET =  .112
    ENERGY LOSS, FEET =   .112

    INLET CONDITIONS:
       ENERGY GRADE =      418.882
       HYDRAULIC GRADE =   418.600

1430
entrance loss to grit eff pipe
"K"LOSS IN FULL ROUND PIPE
    LOSS COEFFICIENT K =   1.250
    PIPE DIAMETER =        60.00 INCHES
    INVERT ELEVATION =   402.500

    VELOCITY =    4.26 FT/SEC
```

```
    ENERGY LOSS, FEET =    .351
    ENERGY GRADE =      419.233
    HYDRAULIC GRADE =   418.952

BROWN AND CALDWELL          PROFILE        SERIAL NO. 9709
Consulting Engineers        Version 2.00

1440
90 vert turn down grit eff manhole
"K" LOSS IN RECTANGULAR OPEN CHANNEL
    WIDTH =   6.00 FEET
    INVERT ELEV. =   416.500 FEET
    SIDEWALL =   9.50 FEET
    LOSS COEFFICIENT "K" =   1.50

    VELOCITY =    4.77 FT/SEC
    ENERGY LOSS, FEET =    .529
    ENERGY GRADE =      419.762
    HYDRAULIC GRADE =   419.409

**1450
** split flow between grit tanks (50%)
 FLOW PERCENT
    FLOW =    41.77 CFS OR     27.00 MGD
            50.00 PERCENT OF FLOW THROUGH ADJACENT DOWNSTREAM ELEMENT.

1460
90 deg turn from tank 1 eff channel to combined eff channel
"K" LOSS IN RECTANGULAR OPEN CHANNEL
    WIDTH =   6.00 FEET
    INVERT ELEV. =   416.500 FEET
    SIDEWALL =   6.50 FEET
    LOSS COEFFICIENT "K" =   1.00

    VELOCITY =    2.13 FT/SEC
    ENERGY LOSS, FEET =    .071
    ENERGY GRADE =      419.833
    HYDRAULIC GRADE =   419.762

BROWN AND CALDWELL          PROFILE        SERIAL NO. 9709
Consulting Engineers        Version 2.00

1470
grit tank 1 eff weir
SHARP-CRESTED WEIR
    WEIR CREST ELEVATION =   422.000
    WEIR DISCHARGE =    41.77 CFS
    LENGTH =    30.00 FEET

    NO END CONTRACTIONS
    FREEBOARD =  2.238
    CALCULATED C VALUE =  3.364
    HEIGHT OF WATER OVER WEIR =   .556
    ENERGY LOSS, FEET =   2.723
    ENERGY GRADE =      422.556
    HYDRAULIC GRADE =   422.556

1480
grit tank 1
RECTANGULAR CONDUIT
    HEIGHT =   6.25 FEET
    WIDTH =  30.00 FEET
    LENGTH=     30.00 FEET
```

MANNING ROUGHNESS = .0150
SLOPE = .00000 FEET/FOOT
NUMBER OF ANALYSIS SECTIONS = 10.00
INVERT ELEV AT OUTLET = 419.750

SUBCRITICAL FLOW

STATION FEET	WATER DEPTH FEET	VELOCITY FT/SEC	FRICTION FACTOR FT/FOOT	AVERAGE FRICTION FACTOR FT/FOOT	FRICTION LOSS FEET	HYDRAULIC GRADE	ENERGY GRADE
.000	2.802	.497	.00001			422.552	422.556
3.000	2.802	.497	.00001	.00001	.000	422.552	422.556
6.000	2.802	.497	.00001	.00001	.000	422.552	422.556
9.000	2.802	.497	.00001	.00001	.000	422.552	422.556
12.000	2.802	.497	.00001	.00001	.000	422.552	422.556
15.000	2.802	.497	.00001	.00001	.000	422.552	422.556
18.000	2.802	.497	.00001	.00001	.000	422.552	422.556
21.000	2.802	.497	.00001	.00001	.000	422.552	422.556
24.000	2.802	.497	.00001	.00001	.000	422.552	422.556
27.000	2.802	.497	.00001	.00001	.000	422.552	422.556
30.000	2.802	.497	.00001	.00001	.000	422.552	422.556

CRITICAL SLOPE, FT/FT = .0046
CRITICAL DEPTH, FEET = .39
CHANGE IN HYDRAULIC GRADE WITHIN CONDUIT, FEET = .000
ENERGY LOSS, FEET = .000

INLET CONDITIONS:
ENERGY GRADE = 422.556
HYDRAULIC GRADE = 422.552

1500
Grit Tank 1 Inf gates
SUBMERGED RECTANGULAR ORIFICE
NO OF ORIFICES = 5
ORIFICE HEIGHT = 1.70 FEET
ORIFICE WIDTH = 3.50 FEET
DISCHARGE COEFFICIENT = .600

FLOW PER ORIFICE = 8.35 CFS
VELOCITY THROUGH ORIFICE, FPS = 1.40
ENERGY LOSS, FEET = .085
ENERGY GRADE = 422.641
HYDRAULIC GRADE = 422.641

BROWN AND CALDWELL PROFILE SERIAL NO. 9709
Consulting Engineers Version 2.00

1510
90 deg turn in tank 1 inf channel to last gate opening
"K" LOSS IN RECTANGULAR OPEN CHANNEL
WIDTH = 6.00 FEET
INVERT ELEV. = 420.500 FEET
SIDEWALL = 5.50 FEET
LOSS COEFFICIENT "K" = 1.00

VELOCITY = 3.25 FT/SEC
ENERGY LOSS, FEET = .164
ENERGY GRADE = 422.805

HYDRAULIC GRADE =   422.641

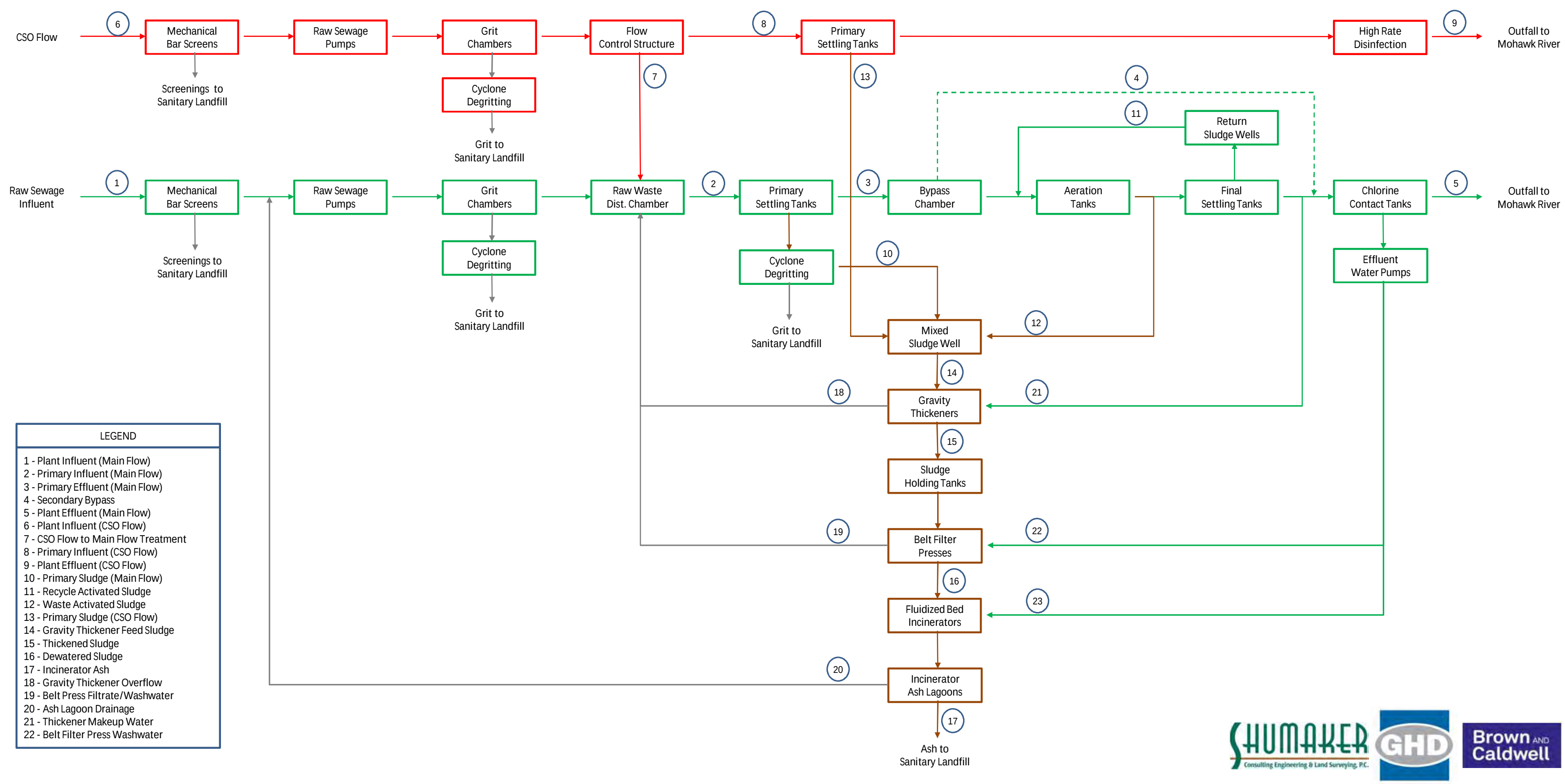
APPENDIX D

SOLIDS HANDLING EVALUATION

APPENDIX D-1

SOLIDS MASS BALANCES

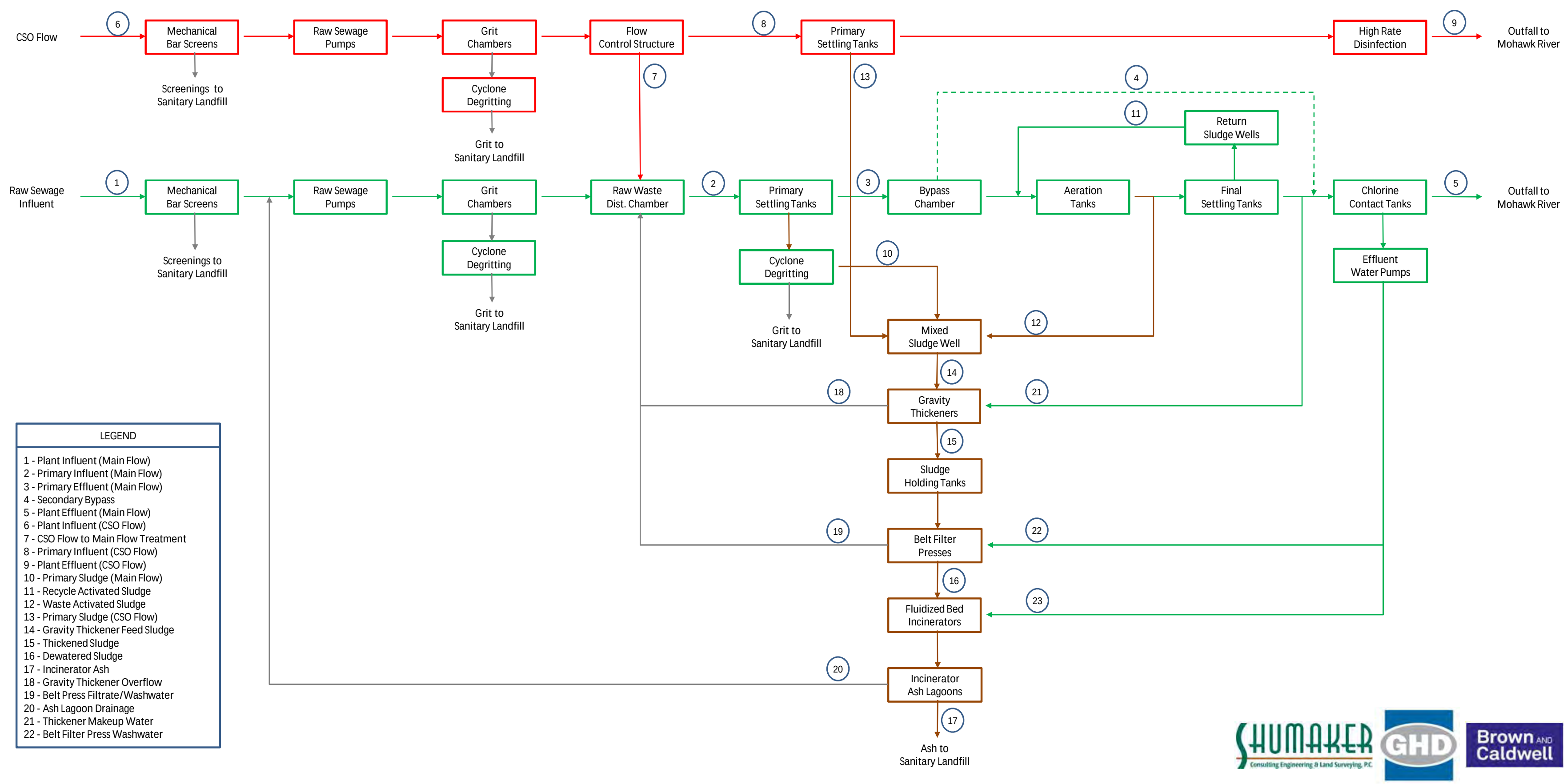
SLUDGE PROCESSING ALTERNATIVES



Flow Stream	1	2	3	4	5	6	7	8	9	10	11	12
Flow, MGD	51.00	67.53	67.03	-	66.27	60	14	46	45.641	0.505	15	0.63
BOD5, mg/L	78.6	-	-	-	12.4	60	60	60				
BOD5, lb/day	33432	-	-	-	6853	30024	7006	23018				
TSS, mg/L	60.3	71.6	56.0	56.0	21.94	50	50	50	27	2142.6	14,467	2721.3
TSS, lb/day	25648	40,304	31280	-	12126	25020	5838	19182	10277	9024	1,809,822	14298
VSS, mg/L	48.8	48.7	36.2	36.2	14.76	40	40	40	21	1713.6	9,732	1830.5
VSS, lb/day	20757	27,443	20226	-	8158	20016	4670	15346	7994	7217	1,217,473	9618
TKN, mg/L	12.4	11.0	-	-	4.23	8	8	8				
TKN, lb/day	5274	6,208	-	-	2338	4003	934	3069				
TS, %												
TS, lb/day												
TVS, % TS												
TVS, lb/day												

Flow Stream	13	14	15	16	17	18	19	20	21	22	23
Flow, MGD	0.359	1.494	0.1	0.02		1.394	0.2	0.94	-	0.12	0.94
BOD5, mg/L									12.4		
BOD5, lb/day									-		
TSS, mg/L	6324.5	3391.5	48,135	235,863		181.7	494.6	750	21.94	21.94	21.94
TSS, lb/day	18936	42258	40,145	39,342	663	2113	825	5880	-	22.0	172.0
VSS, mg/L	4067.7	2328.6	33,050	161,948		124.7	339.2		14.76	14.76	14.76
VSS, lb/day	12179	29014	27,564	27,013		1450	566			14.8	115.7
TKN, mg/L									4.23	4.23	4.23
TKN, lb/day										4.2	33.2
TS, %			4.81	23.59							
TS, lb/day			40145	39342							
TVS, % TS			68.66	68.66							
TVS, lb/day			27564	27013							

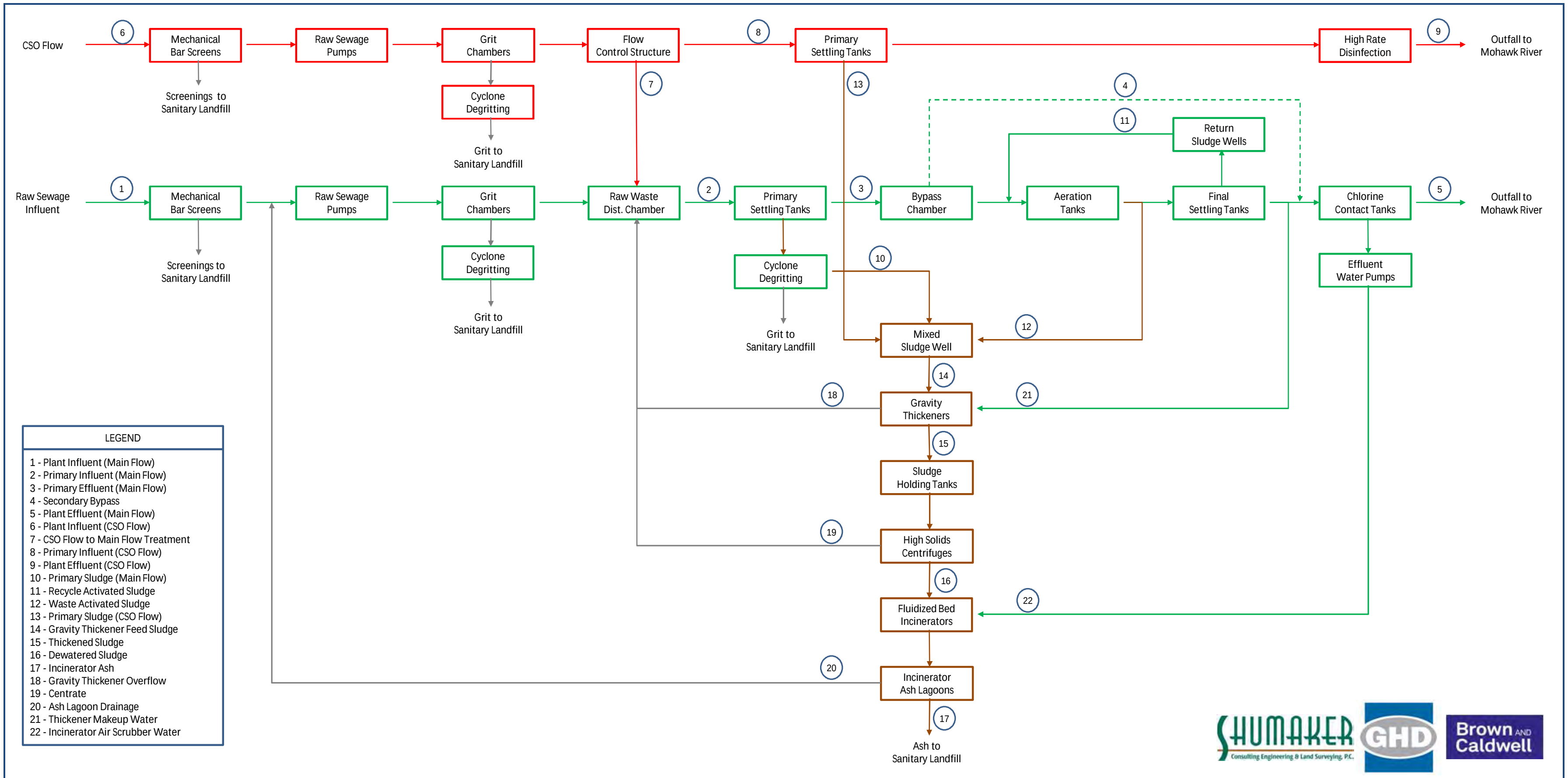
SOLIDS BALANCE
SLUDGE PROCESSING ALTERNATIVE NO. 1
DESIGN DAILY AVERAGE CONDITIONS



Flow Stream	1	2	3	4	5	6	7	8	9	10	11	12
Flow, MGD	60.00	67.73	67.23	-	66.27	51	5	46	45.641	0.505	18	0.84
BOD5, mg/L	86.8	-	-	-	9.59	60	60	60				
BOD5, lb/day	43435	-	-	-	5300	25520	2502	23018				
TSS, mg/L	65.1	78.0	41.6	41.6	12.95	50	50	50	27	4914.9	13,895	2980.2
TSS, lb/day	32576	44,046	23346	-	7157	21267	2085	19182	10277	20700	2,085,917	20878
VSS, mg/L	48.8	50.4	26.9	26.9	8.6	40	40	40	21	3178.5	9,230	1979.6
VSS, lb/day	24420	28,474	15087	-	4753	17014	1668	15346	7994	13387	1,385,608	13868
TKN, mg/L	12.4	11.6	-	-	3.5	8	8	8				
TKN, lb/day	6205	6,539	-	-	1934	3403	334	3069				
TS, %												
TS, lb/day												
TVS, % TS												
TVS, lb/day												

Flow Stream	13	14	15	16	17	18	19	20	21	22	23
Flow, MGD	0.359	1.704	0.115	0.03		1.589	0.205	0.94	-	0.12	0.94
BOD5, mg/L									9.59		
BOD5, lb/day									-		
TSS, mg/L	3014.0	3560.7	50,121	188,289		191.0	569.7	750	12.95	12.95	12.95
TSS, lb/day	9024	50602	48,071	47,110	808	2531	974	5880	-	13.0	101.5
VSS, mg/L	2410.4	2425.7	34,146	128,273		130.0	388.1		8.6	8.6	8.6
VSS, lb/day	7217	34472	32,749	32,094		1723	664			8.6	67.4
TKN, mg/L									3.5	3.5	3.5
TKN, lb/day										3.5	27.4
TS, %			5.01	18.83							
TS, lb/day			48071	47110							
TVS, % TS			68.13	68.13							
TVS, lb/day			32749	32094							

SOLIDS BALANCE
SLUDGE PROCESSING ALTERNATIVE NO. 1
DESIGN MAXIMUM MONTH CONDITION



Flow Stream	1	2	3	4	5	6	7	8	9	10	11
Flow, MGD	51.00	67.53	67.03	-	66.28	60	14	46	45.641	0.505	15
BOD5, mg/L	78.6	-	-	-	12.4	60	60	60			
BOD5, lb/day	33432	-	-	-	6854	30024	7006	23018			
TSS, mg/L	60.3	71.6	38.2	38.2	21.95	50	50	50	27	4495.8	14,467
TSS, lb/day	25648	40,303	21368	-	12133	25020	5838	19182	10277	18935	1,809,822
VSS, mg/L	48.8	48.7	27.3	27.3	14.76	40	40	40	21	2891.7	9,723
VSS, lb/day	20757	27,444	15265	-	8159	20016	4670	15346	7994	12179	1,216,347
TKN, mg/L	12.4	11.0	-	-	4.23	8	8	8			
TKN, lb/day	5274	6,208	-	-	2338	4003	934	3069			
TS, %											
TS, lb/day											
TVS, % TS											
TVS, lb/day											

Flow Stream	12	13	14	15	16	17	18	19	20	21	22	
Flow, MGD	0.63	0.359	1.494	0.1	0.02		1.394	0.2	0.94	-	0.12	0.94
BOD5, mg/L										12.4		
BOD5, lb/day										-		
TSS, mg/L	2721.1	3014.0	3391.3	48,134	235,857		181.7	494.6	750	21.95	21.95	21.95
TSS, lb/day	14297	9024	42256	40,144	39,341	661	2112	825	5880	-	22.0	172.1
VSS, mg/L	1830.5	2410.4	2328.6	33,049	161,942		124.8	339.2		14.76	14.76	14.76
VSS, lb/day	9618	7217	29014	27,563	27,012		1451	566			14.8	115.7
TKN, mg/L										4.23	4.23	4.23
TKN, lb/day											4.2	33.2
TS, %				4.81	23.59							
TS, lb/day				40144	39341							
TVS, % TS				68.66	68.66							
TVS, lb/day				27563	27012							

SOLIDS BALANCE
SLUDGE PROCESSING ALTERNATIVE NO. 2
DESIGN DAILY AVERAGE CONDITION

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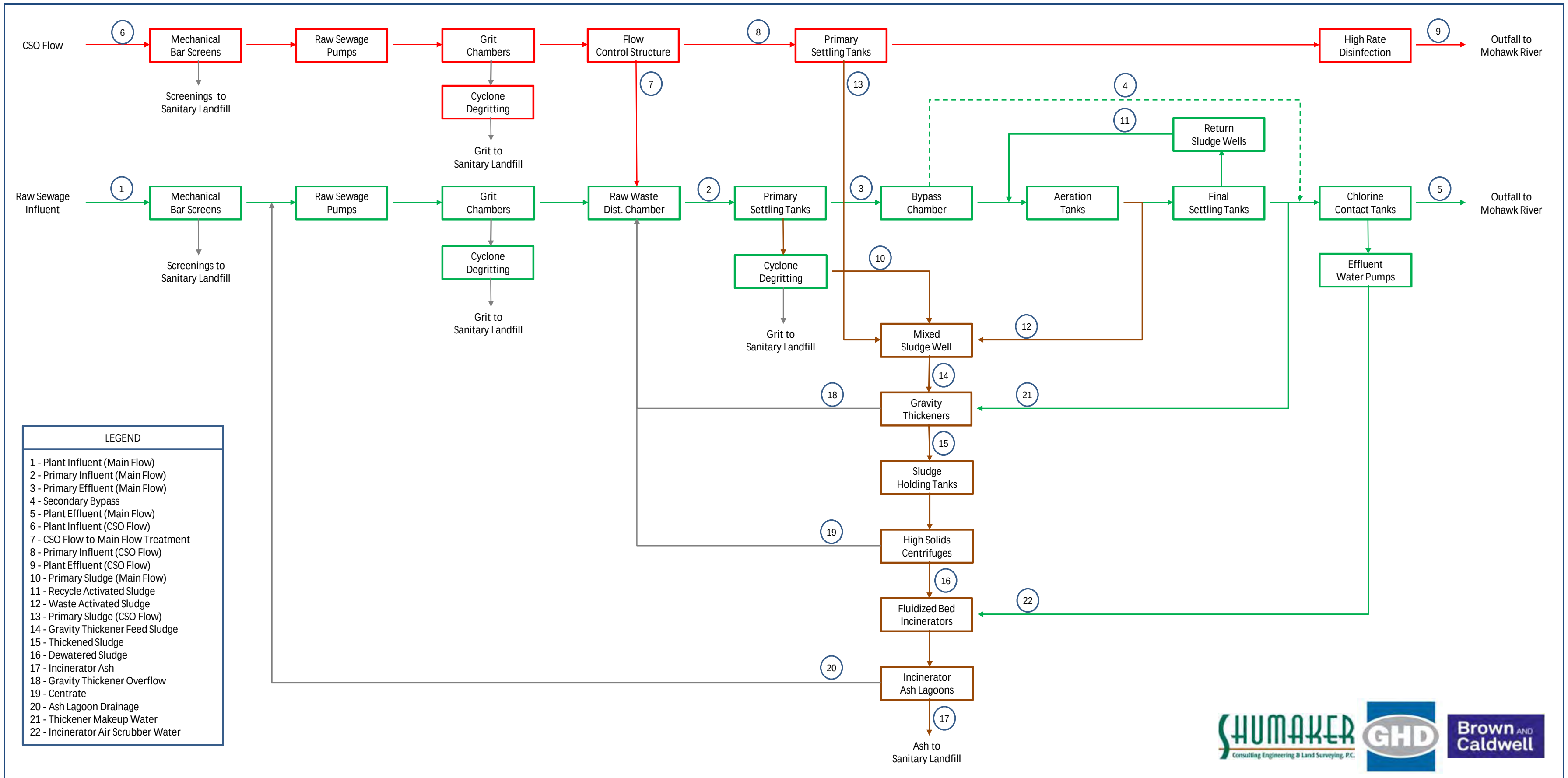
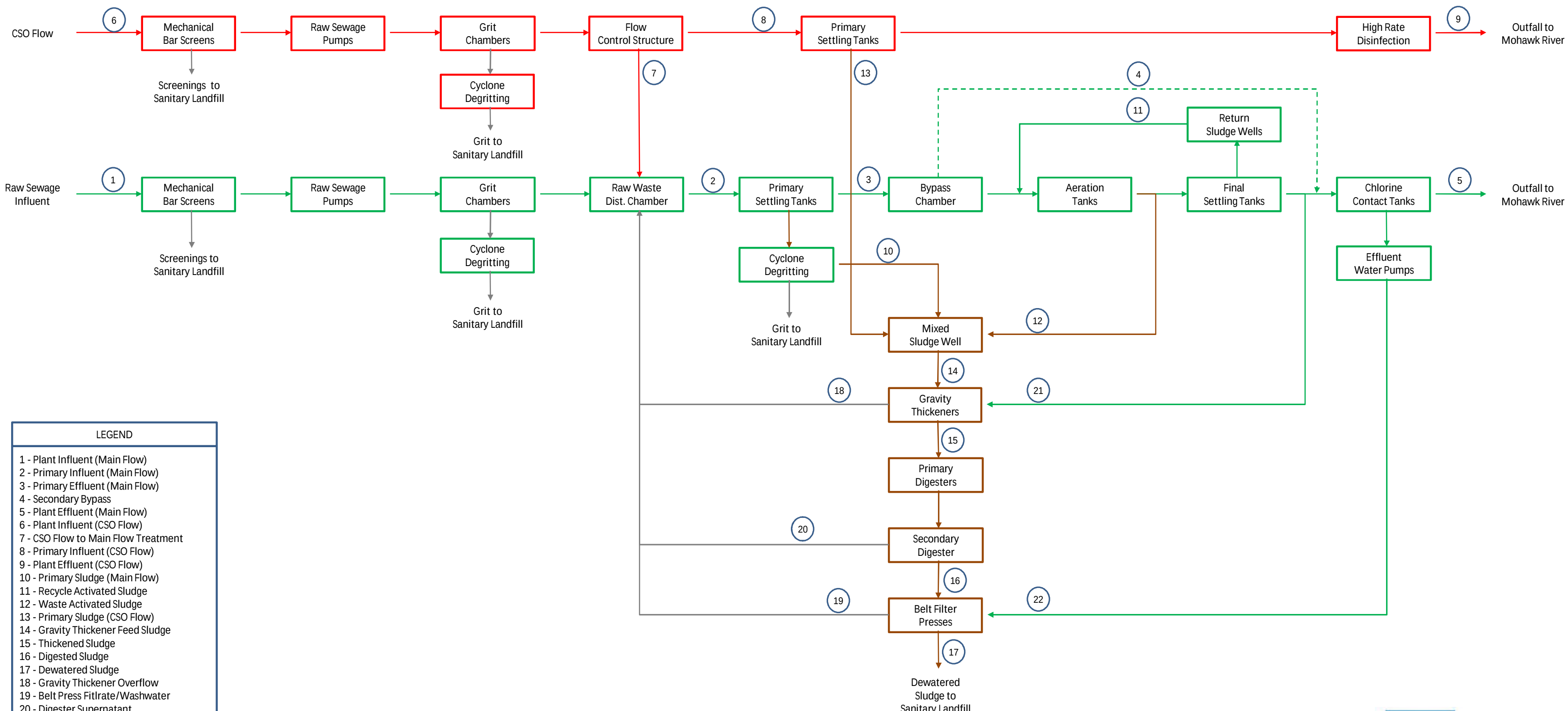
Incin-Cent (Design ADF)



Flow Stream	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	
Flow, MGD	60.00	67.74	67.24	-	66.27	51	5	46	45.641	0.505	18	0.84	0.359	1.704	0.12	0.02		1.584	0.22	0.94	-	0.12	0.94
BOD5, mg/L	86.8	-	-	-	9.59	60	60	60													9.59		
BOD5, lb/day	43435	-	-	-	5300	25520	2502	23018													-		
TSS, mg/L	65.1	78.0	41.6	41.6	12.95	50	50	50	27	4914.9	13,895	2979.9	3014.0	3560.5	48,032	282,422		191.5	531.4	750	12.95	12.95	12.95
TSS, lb/day	32576	44,046	23346	-	7157	21267	2085	19182	10277	20700	2,085,917	20876	9024	50600	48,070	47,108	807	2530	975	5880	-	13.0	101.5
VSS, mg/L	48.8	50.4	26.9	26.9	8.6	40	40	40	21	3178.5	9,230	1979.4	2410.4	2425.6	32,722	192,404		130.4	361.7		8.6	8.6	8.6
VSS, lb/day	24420	28,474	15087	-	4753	17014	1668	15346	7994	13387	1,385,608	13867	7217	34471	32,748	32,093		1723	664			8.6	67.4
TKN, mg/L	12.4	11.6	-	-	3.54	8	8	8													3.54	3.54	3.54
TKN, lb/day	6205	6,539	-	-	1957	3403	334	3069														3.5	27.8
TS, %															4.80	28.24							
TS, lb/day															48070	47108							
TVS, % TS															68.13	68.13							
TVS, lb/day															32748	32093							

SOLIDS BALANCE
SLUDGE PROCESSING ALTERNATIVE NO. 2
DESIGN MAXIMUM MONTH CONDITION



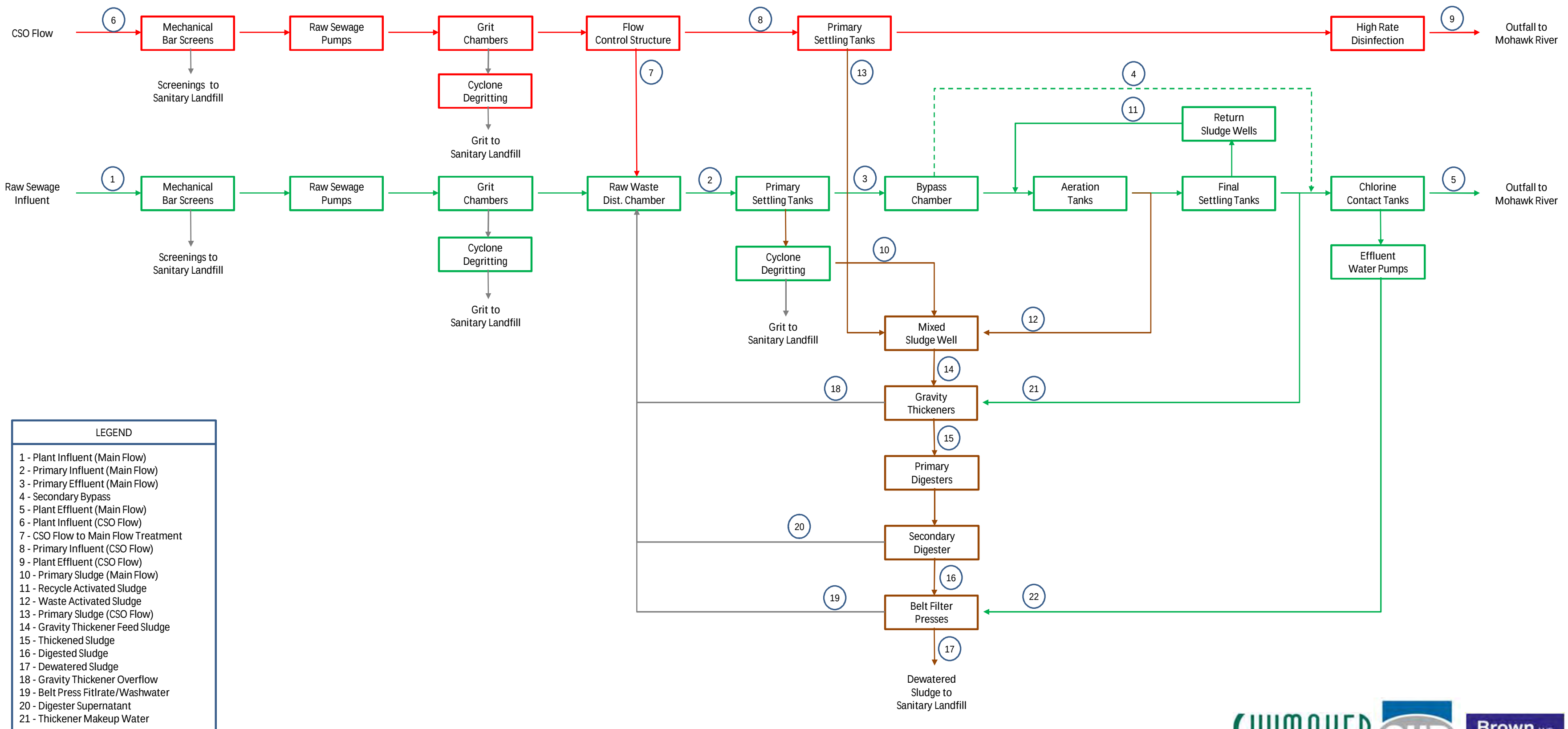
LEGEND

1 - Plant Influent (Main Flow)
2 - Primary Influent (Main Flow)
3 - Primary Effluent (Main Flow)
4 - Secondary Bypass
5 - Plant Effluent (Main Flow)
6 - Plant Influent (CSO Flow)
7 - CSO Flow to Main Flow Treatment
8 - Primary Influent (CSO Flow)
9 - Plant Effluent (CSO Flow)
10 - Primary Sludge (Main Flow)
11 - Recycle Activated Sludge
12 - Waste Activated Sludge
13 - Primary Sludge (CSO Flow)
14 - Gravity Thickener Feed Sludge
15 - Thickened Sludge
16 - Digested Sludge
17 - Dewatered Sludge
18 - Gravity Thickener Overflow
19 - Belt Press Fitlrate/Washwater
20 - Digester Supernatant
21 - Thickener Makeup Water

Flow Stream	1	2	3	4	5	6	7	8	9	10	11
Flow, MGD	51.00	66.01	65.50	-	65.35	60	14	46	45.641	0.505	15
BOD5, mg/L	78.6	-	-	-	12.14	60	60	60			
BOD5, lb/day	33432	-	-	-	6617	30024	7006	23018			
TSS, mg/L	60.3	80.7	52.4	52.4	21.78	50	50	50	27	3756.2	14,466
TSS, lb/day	25648	44,450	28630	-	11871	25020	5838	19182	10277	15820	1,809,697
VSS, mg/L	48.8	67.9	46.4	46.4	16.62	40	40	40	21	2847.1	11,043
VSS, lb/day	20757	37,360	25369	-	9058	20016	4670	15346	7994	11991	1,381,479
TKN, mg/L	12.4	11.3	-	-	3.65	8	8	8			
TKN, lb/day	5274	6,208	-	-	1989	4003	934	3069			
TS, %											
TS, lb/day											
TVS, % TS											
TVS, lb/day											

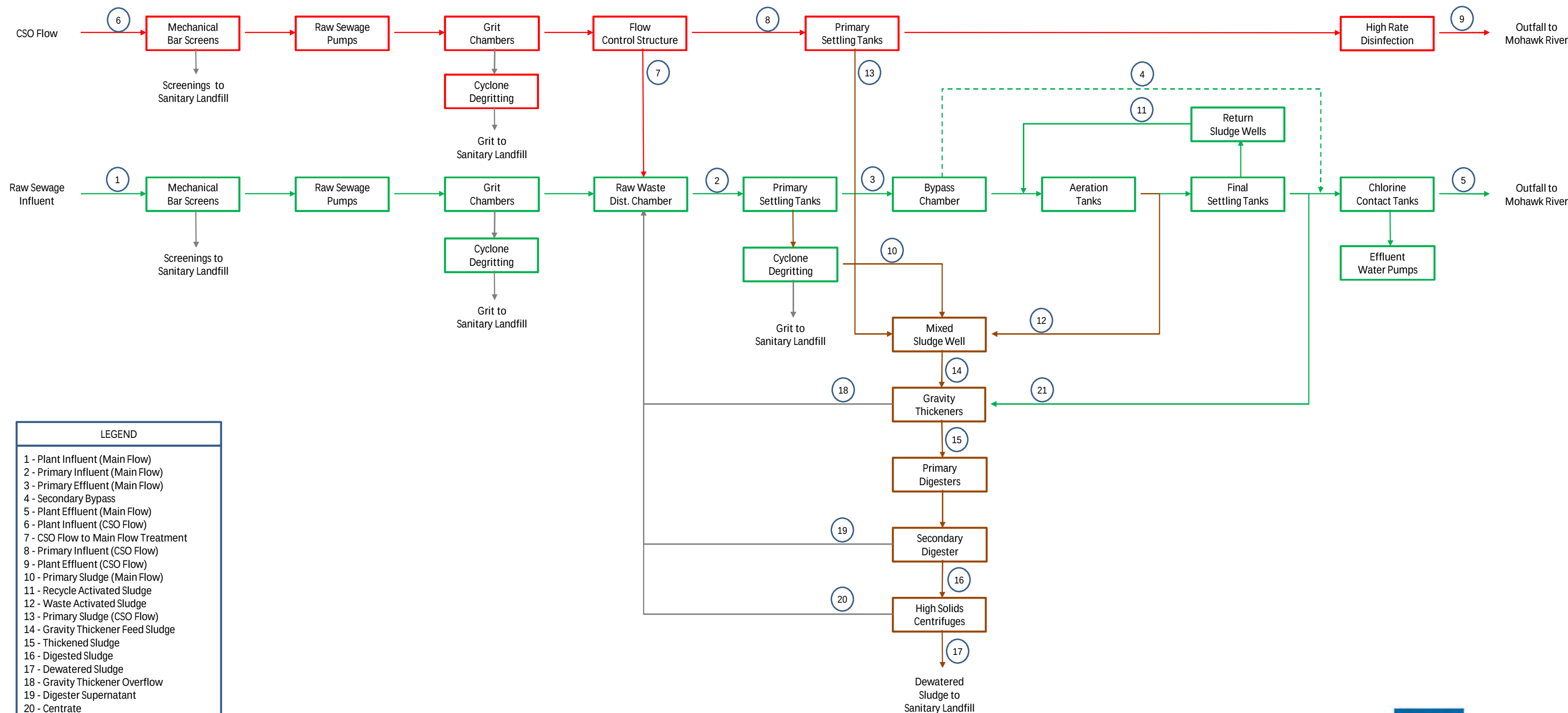
Flow Stream	12	13	14	15	16	17	18	19	20	21	22	
Flow, MGD	0.46	0.359	0.965	0.08	0.08	0.0132	0.885	0.12	0	-	0.12	
BOD5, mg/L										12.14		
BOD5, lb/day										-		
TSS, mg/L	2,758	3014.0	4401.7	47,926	33,698	200,140	467.3	9507.2	-	21.78	21.78	
TSS, lb/day	10,581	9024	35,425	31,976	22,483	22,033	3449	9515	-	-	21.8	
VSS, mg/L	2,105	2410.4	3390.2	36,841	23,035	136,818	366.5	9220.3	-	16.62	16.62	
VSS, lb/day	8,077	7217	27285	24,580	15,369	15,062	2705	9228	-	-	16.6	
TKN, mg/L										3.65	3.65	
TKN, lb/day										-	3.7	
TS, %				4.79	3.37	20.01						
TS, lb/day				31976	22483	22033						
TVS, % TS				76.87	68.36	68.36						
TVS, lb/day				24580	15369	15062						

SOLIDS BALANCE
SLUDGE PROCESSING ALTERNATIVE NO. 3
DESIGN DAILY AVERAGE CONDITION



Flow Stream	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	
Flow, MGD	60.00	66.20	65.69	-	65.3	51	5	46	45.641	0.505	18	0.67	0.359	1.175	0.1	0.1	0.02	1.075	0.12	0	-	0.12	
BOD5, mg/L	86.8	-	-	-	9.2	60	60	60													9.2		
BOD5, lb/day	43435	-	-	-	5010	25520	2502	23018													-		
TSS, mg/L	65.1	89.8	58.4	58.4	12.75	50	50	50	27	4,179	13,997	3,044	3014.0	4452.7	49,703	34,442	168,765	243.4	12729.6	-	12.75	12.75	
TSS, lb/day	32576	49,583	31982	-	6944	21267	2085	19182	10,277	17,601	2,101,230	17,009	9024	43634	41,452	28,725	28,150	2182	12740	-	-	12.8	
VSS, mg/L	48.8	72.5	48.9	48.9	9.53	40	40	40	21	3,141	10,464	2,276	2410.4	3384.1	37,776	23,080	113,094	184.8	12255.7	-	9.53	9.53	
VSS, lb/day	24420	40,010	26781	-	5190	17014	1668	15346	7,994	13,229	1,570,856	12,716	7217	33162	31,505	19,249	18,864	1657	12266	-	-	9.5	
TKN, mg/L	12.4	11.8	-	-	2.98	8	8	8													2.98	2.98	
TKN, lb/day	6205	6,539	-	-	1623	3403	334	3069													-	3.0	
TS, %															4.97	3.44	16.88						
TS, lb/day															41452	28725	28150						
TVS, % TS															76.00	67.01	67.01						
TVS, lb/day															31505	19249	18864						

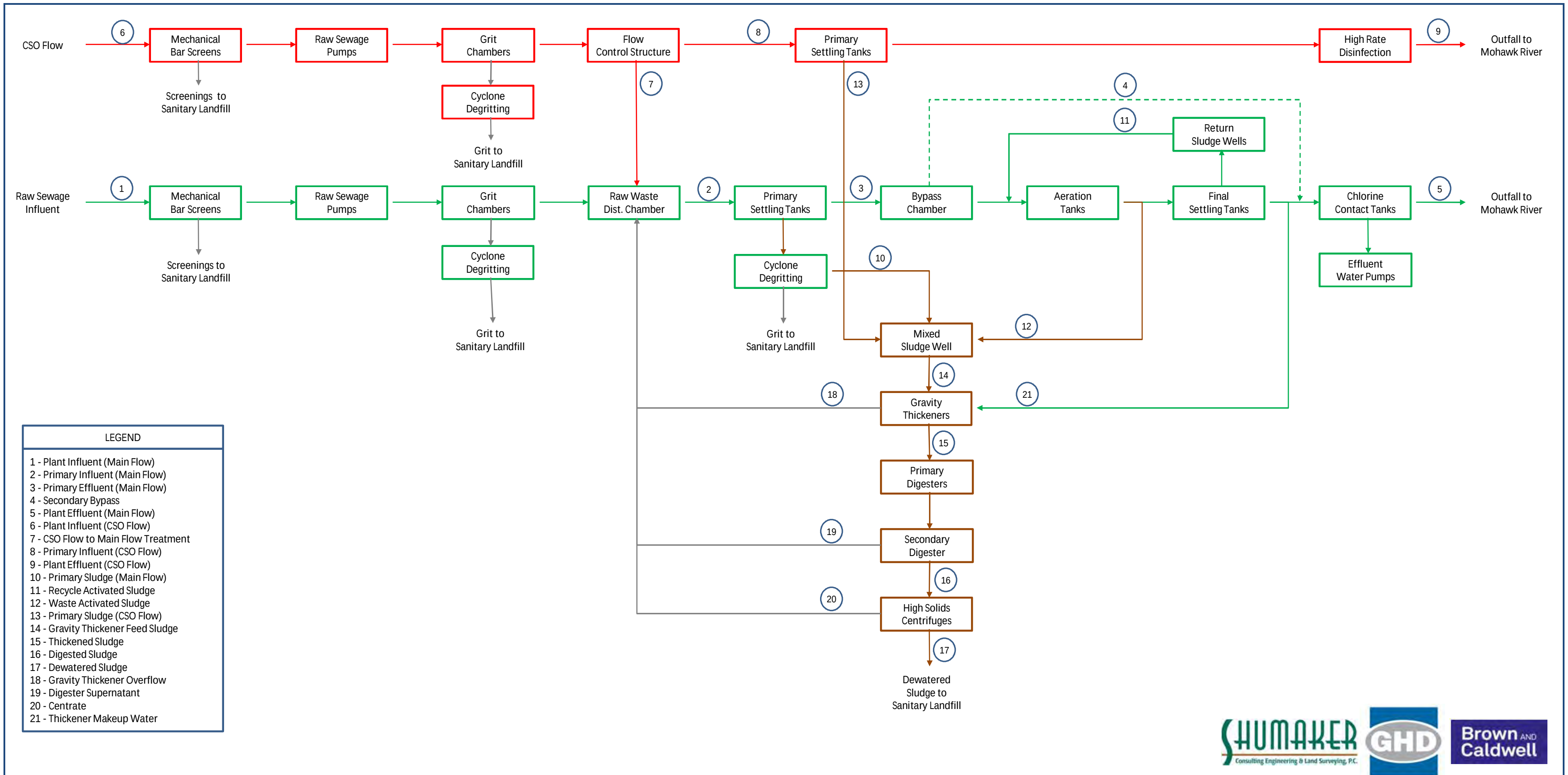
SOLIDS BALANCE
SLUDGE PROCESSING ALTERNATIVE NO. 3
DESIGN MAXIMUM MONTH CONDITION



Flow Stream	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	
Flow, MGD	51.00	66.01	65.50	-	65.35	60	14	46	45.641	0.505	15	0.46	0.359	0.965	0.08	0.08	0.0112	0.885	0.12	0	-	0.12	
BOD5, mg/L	78.6	-	-	-	12.14	60	60	60													12.14		
BOD5, lb/day	33432	-	-	-	6617	30024	7006	23018													-		
TSS, mg/L	60.3	80.7	52.4	52.4	21.78	50	50	50	27	3756.2	14,466	2,758	3014.0	4401.5	47,926	33,696	235,868	467.2	9508.2	-	21.78	21.78	
TSS, lb/day	25648	44,450	28630	-	11871	25020	5838	19182	10277	15820	1,809,697	10,580	9024	35424	31,976	22,482	22,032	3448	9516	-	-	21.8	
VSS, mg/L	48.8	67.9	46.4	46.4	16.63	40	40	40	21	2847.1	11,044	2,105	2410.4	3390.2	36,841	23,035	161,250	366.5	9220.3	-	16.63	16.63	
VSS, lb/day	20757	37,360	25369	-	9064	20016	4670	15346	7994	11991	1,381,604	8,077	7217	27285	24,580	15,369	15,062	2705	9228	-	-	16.6	
TKN, mg/L	12.4	11.3	-	-	3.65	8	8	8													3.65	3.65	
TKN, lb/day	5274	6,208	-	-	1989	4003	934	3069													-	3.7	
TS, %															4.79	3.37	23.59						
TS, lb/day															31976	22482	22032						
TVS, % TS															76.87	68.36	68.36						
TVS, lb/day															24580	15369	15062						

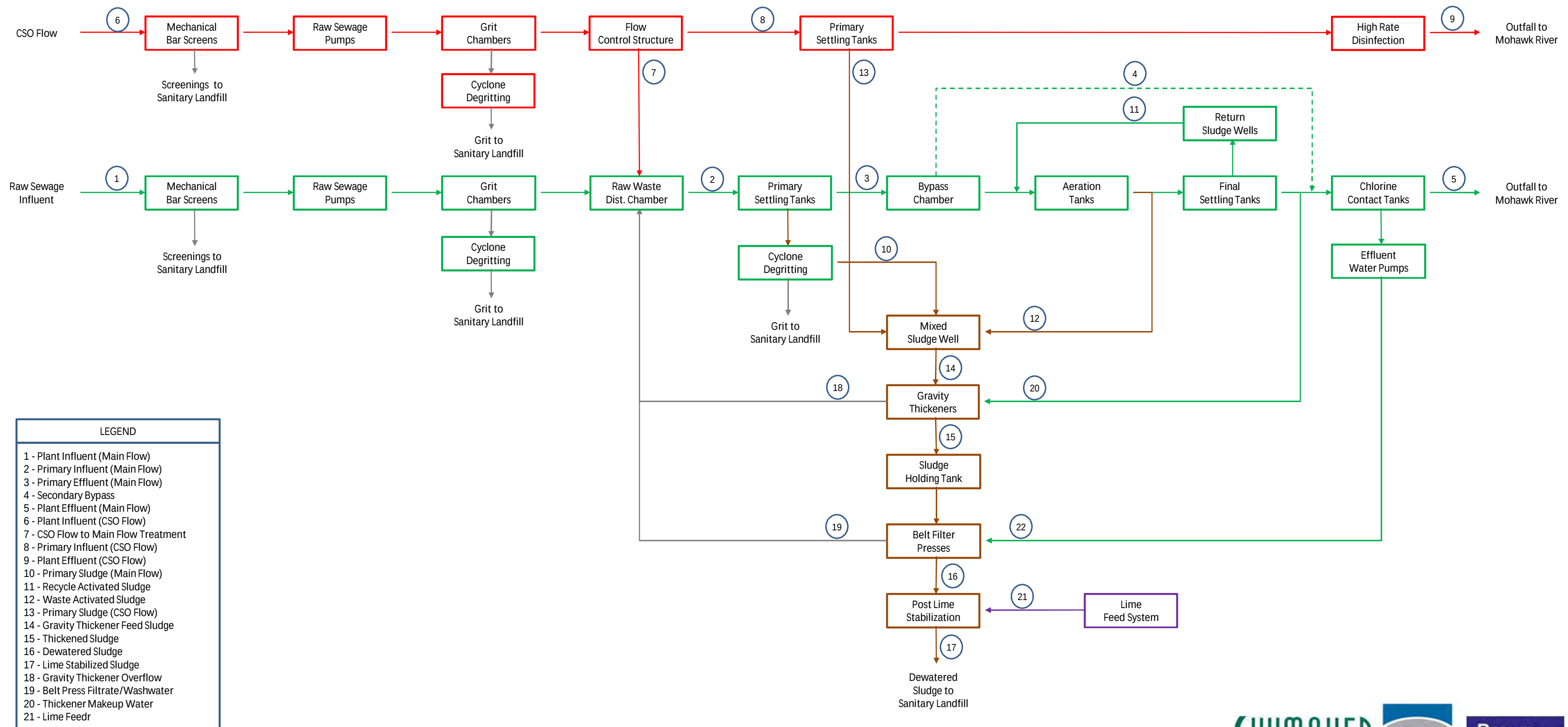
SOLIDS BALANCE
SLUDGE PROCESSING ALTERNATIVE NO. 4
DESIGN DAILY AVERAGE CONDITION

C:\Documents and Settings\nghatala\Desktop\Oneida County\Solids_Balance.xlsx
Dig-Cent (Design ADF)



Flow Stream	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	
Flow, MGD	60.00	66.20	65.69	-	65.35	51	5	46	45.641	0.505	18	0.67	0.359	1.175	0.1	0.1	0.0139	1.075	0.12	0	-	0.12	
BOD5, mg/L	86.8	-	-	-	9.18	60	60	60													9.18		
BOD5, lb/day	43435	-	-	-	5003	25520	2502	23018													-		
TSS, mg/L	65.1	89.8	58.4	58.4	12.75	50	50	50	27	4179.1	14,000	3,044	3014.0	4453.0	49,706	34,444	242,836	243.4	12731.6	-	12.75	12.75	
TSS, lb/day	32576	49,585	31984	-	6949	21267	2085	19182	10277	17601	2,101,680	17,012	9024	43637	41,455	28,726	28,151	2182	12742	-	-	12.8	
VSS, mg/L	48.8	72.5	48.9	48.9	9.53	40	40	40	21	3141.0	10,467	2,276	2410.4	3384.4	37,778	23,082	162,733	184.9	12256.7	-	9.53	9.53	
VSS, lb/day	24420	40,012	26783	-	5194	17014	1668	15346	7994	13229	1,571,306	12,719	7217	33165	31,507	19,250	18,865	1658	12267	-	-	9.5	
TKN, mg/L	12.4	11.8	-	-	2.98	8	8	8													2.98	2.98	
TKN, lb/day	6205	6,539	-	-	1624	3403	334	3069													-	3.0	
TS, %															4.97	3.44	24.28						
TS, lb/day															41455	28726	28151						
TVS, % TS															76.00	67.01	67.01						
TVS, lb/day															31507	19250	18865						

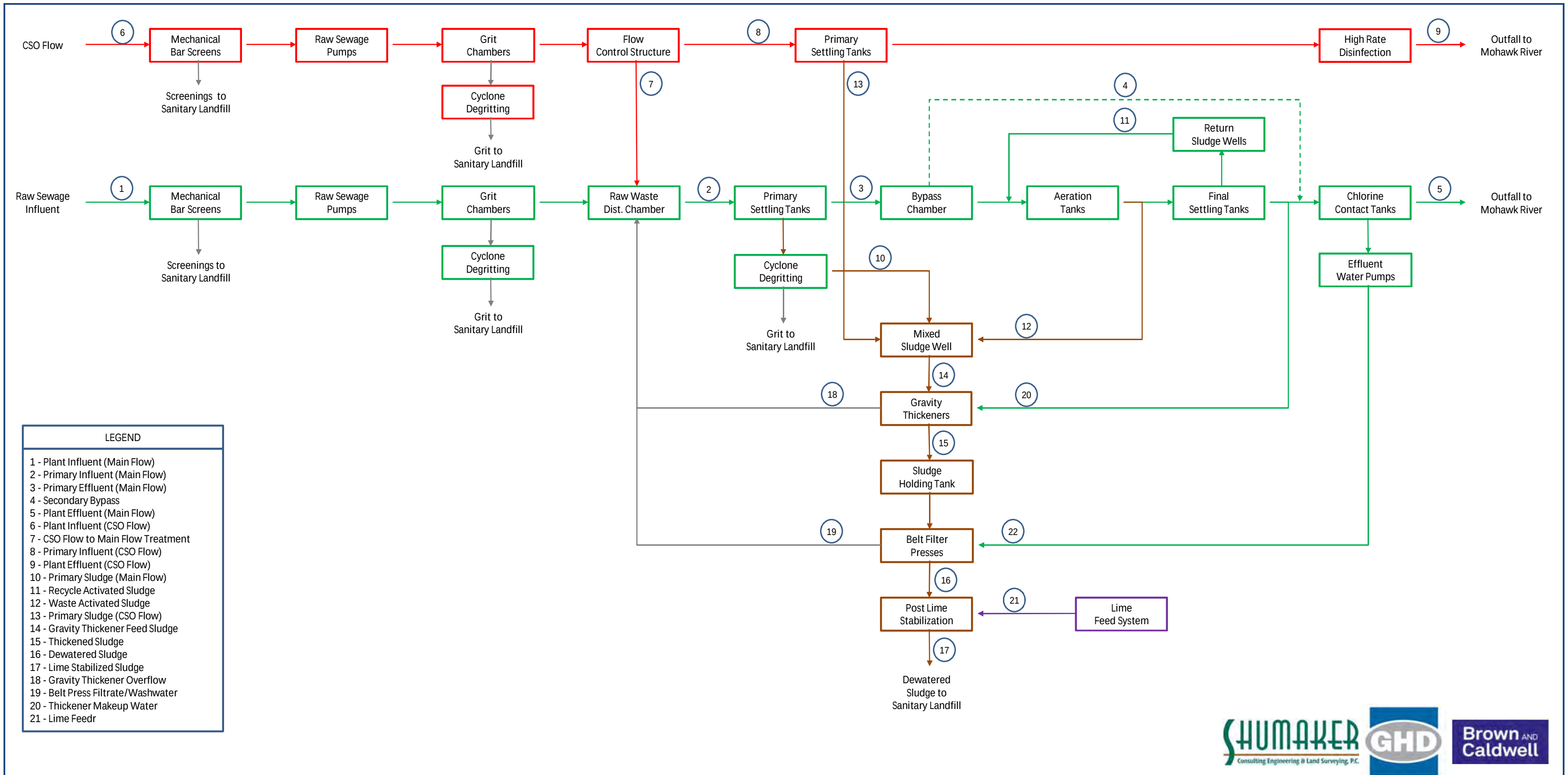
SOLIDS BALANCE
SLUDGE PROCESSING ALTERNATIVE NO. 4
DESIGN MAXIMUM MONTH CONDITION



Flow Stream	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	
Flow, MGD	51.00	66.06	65.56	-	65.34	60	14	46	45.641	0.505	15	0.46	0.359	0.965	0.08	0.0204	0.0204	0.885	0.1796	-		0.12	
BOD5, mg/L	78.6	-	-	-	12.1	60	60	60															
BOD5, lb/day	33432	-	-	-	6594	30024	7006	23018															
TSS, mg/L	60.3	61.6	32.9	32.9	21.84	50	50	50	27	3790.6	14,466	2,758	3014.0	4419.8	50,647	194,644	194,644	241.0	465.9	21.84		21.84	
TSS, lb/day	25648	33962.86	17998	-	11901	25020	5838	19182	10277	15965	1,809,697	10,582	9024	35571	33,792	33,116	33,116	1779	698	-		21.9	
VSS, mg/L	48.8	49.6	27.8	27.8	16.68	40	40	40	21	2879.6	11,046	2,106	2410.4	3407.6	39,050	150,068		185.7	359.6	16.68		16.68	
VSS, lb/day	20757	27336.69	15209	-	9090	20016	4670	15346	7994	12128	1,381,855	8,080	7217	27425	26,054	25,532		1371	539	-		16.7	
TKN, mg/L	12.4	11.3	-	-	3.75	8	8	8												3.75		3.75	
TKN, lb/day	5274	6208.30	-	-	2044	4003	934	3069												-		3.8	
TS, %															5.06	19.46							
TS, lb/day															33792	33116							
TVS, % TS															77.10	77.10							
TVS, lb/day															26054	25532							

SOLIDS BALANCE
SLUDGE PROCESSING ALTERNATIVE NO. 5
DESIGN DAILY AVERAGE CONDITION

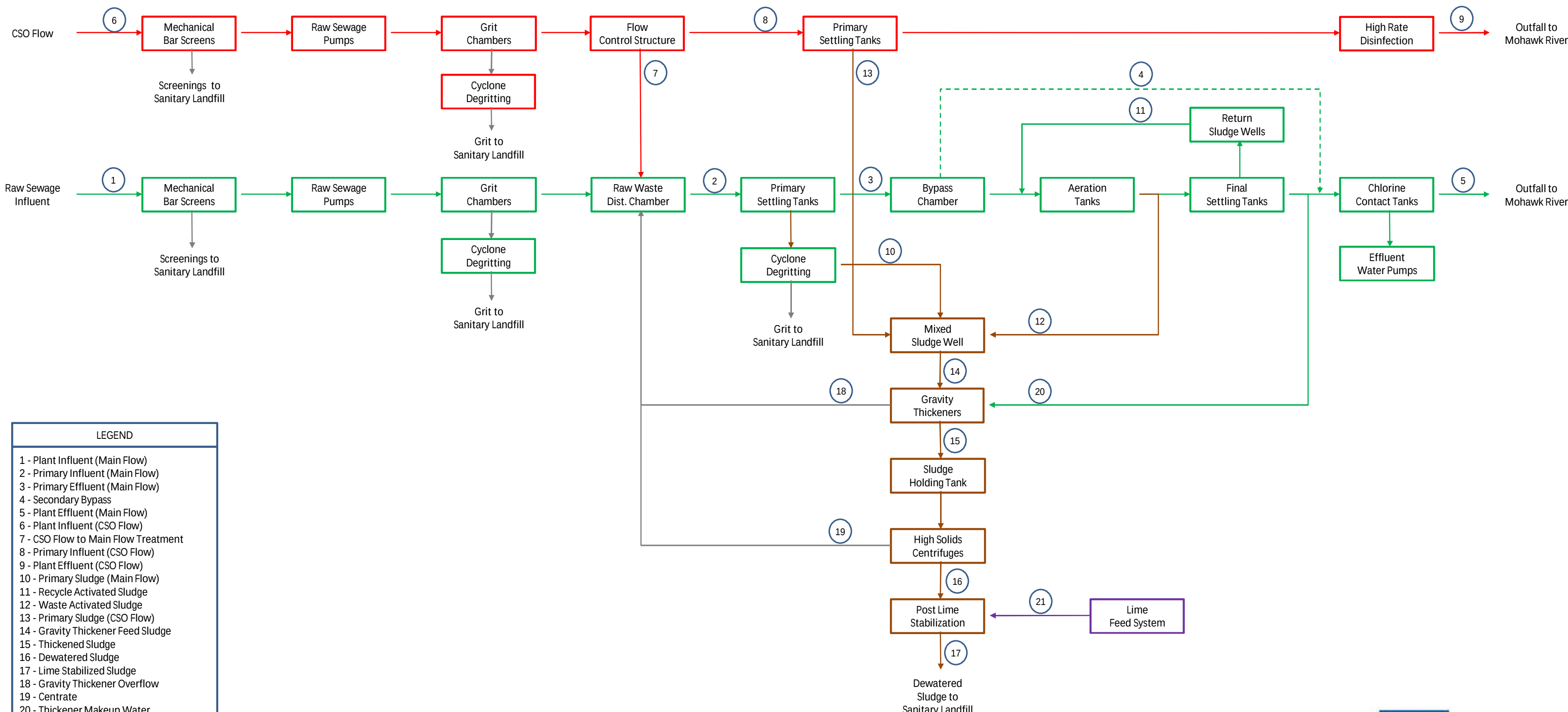
C:\Documents and Settings\nghatala\Desktop\Oneida County\Solids_Balance.xlsx
Lime-BFP (Design ADF)



Flow Stream	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
Flow, MGD	60.00	66.26	65.75	-	65.33	51	5	46	45.641	0.505	18	0.66	0.359	1.165	0.1	0.03	0.03	1.065	0.19	-		0.12	
BOD5, mg/L	86.8	-	-	-	9.1	60	60	60															
BOD5, lb/day	43435	-	-	-	4958	25520	2502	23018															
TSS, mg/L	65.1	68.2	36.4	36.4	12.8	50	50	50	27	4,207	14,143	3,067	3014.0	4490.1	49,693	162,334	162,334	245.7	530.6	12.8		12.8	
TSS, lb/day	32576	37683.85	19964	-	6974	21267	2085	19182	10,277	17,720	2,123,147	16,882	9024	43626	41,444	40,616	40,616	2182	841	-		12.8	
VSS, mg/L	48.8	51.4	27.4	27.4	9.55	40	40	40	21	3,169	10,563	2,291	2410.4	3413.9	37,783	123,425		186.8	403.6	9.55		9.55	
VSS, lb/day	24420	28386.08	15041	-	5203	17014	1668	15346	7,994	13,345	1,585,718	12,608	7217	33170	31,511	30,881		1659	640	-		9.6	
TKN, mg/L	12.4	11.8	-	-	3.1	8	8	8												3.1		3.1	
TKN, lb/day	6205	6538.56	-	-	1689	3403	334	3069												-		3.1	
TS, %															4.97	16.23							
TS, lb/day															41444	40616							
TVS, % TS															76.03	76.03							
TVS, lb/day															31511	30881							

SOLIDS BALANCE
SLUDGE PROCESSING ALTERNATIVE NO. 5
DESIGN MAXIMUM MONTH CONDITION

C:\Documents and Settings\nghatala\Desktop\Oneida County\Solids_Balance.xlsx
Lime-BFP (Design MMF)



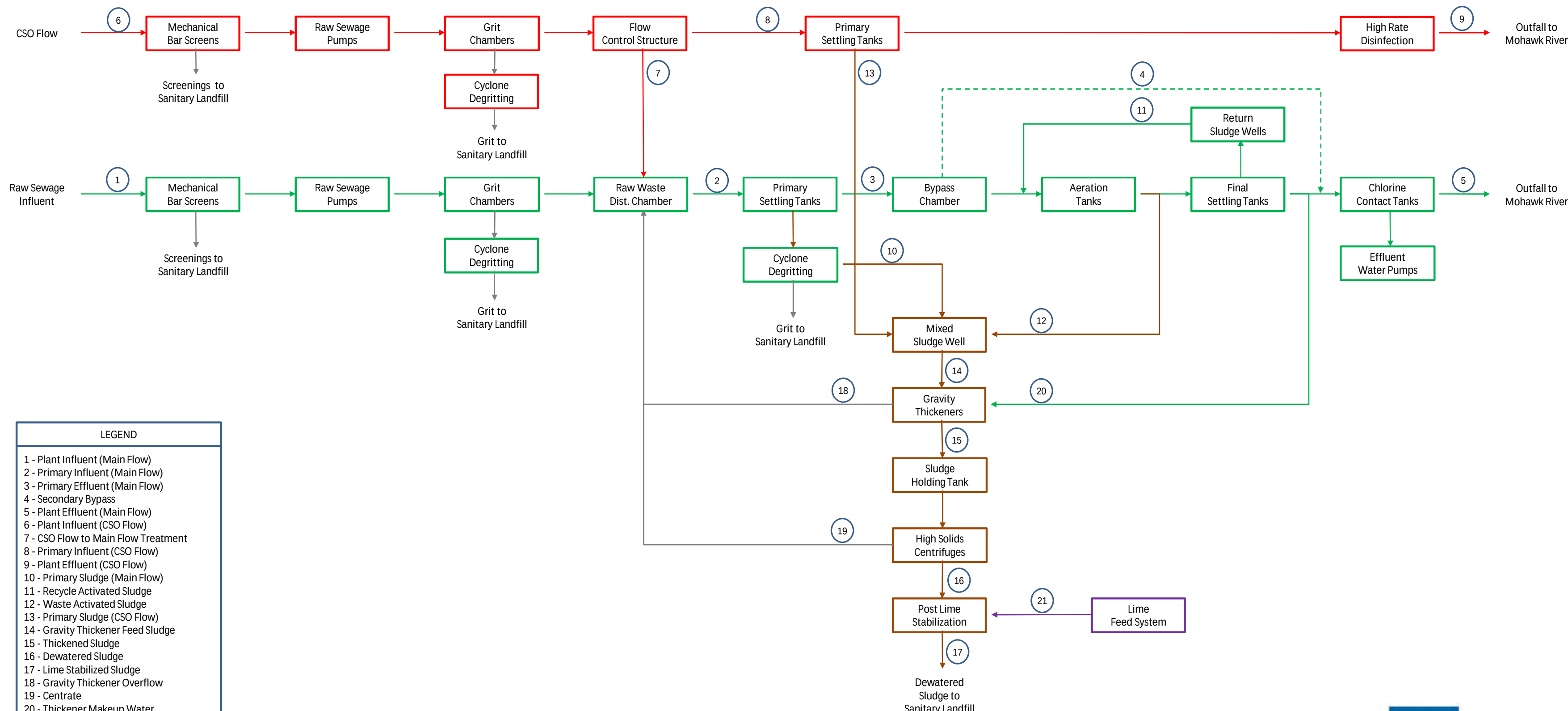
LEGEND

1 - Plant Influent (Main Flow)
2 - Primary Influent (Main Flow)
3 - Primary Effluent (Main Flow)
4 - Secondary Bypass
5 - Plant Effluent (Main Flow)
6 - Plant Influent (CSO Flow)
7 - CSO Flow to Main Flow Treatment
8 - Primary Influent (CSO Flow)
9 - Plant Effluent (CSO Flow)
10 - Primary Sludge (Main Flow)
11 - Recycle Activated Sludge
12 - Waste Activated Sludge
13 - Primary Sludge (CSO Flow)
14 - Gravity Thickener Feed Sludge
15 - Thickened Sludge
16 - Dewatered Sludge
17 - Lime Stabilized Sludge
18 - Gravity Thickener Overflow
19 - Centrate
20 - Thickener Makeup Water
21 - Lime Feed

Flow Stream	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	
Flow, MGD	51.00	66.07	65.56	-	65.34	60	14	46	45.641	0.505	15	0.46	0.359	0.965	0.08	0.016	0.016	0.885	0.184	-		0.12	
BOD5, mg/L	78.6	-	-	-	12.1	60	60	60															
BOD5, lb/day	33432	-	-	-	6594	30024	7006	23018															
TSS, mg/L	60.3	61.6	32.9	32.9	21.84	50	50	50	27	3790.6	14,466	2,758	3014.0	4419.7	50,647	248,171	248,171	240.9	454.8	21.84		21.84	
TSS, lb/day	25648	33961.86	17997	-	11901	25020	5838	19182	10277	15965	1,809,697	10,581	9024	35570	33,792	33,116	33,116	1778	698	-		21.9	
VSS, mg/L	48.8	49.6	27.8	27.8	16.68	40	40	40	21	2879.6	11,046	2,106	2410.4	3407.6	39,048	191,337		185.9	350.4	16.68		16.68	
VSS, lb/day	20757	27336.69	15209	-	9090	20016	4670	15346	7994	12128	1,381,855	8,080	7217	27425	26,053	25,532		1372	538	-		16.7	
TKN, mg/L	12.4	11.3	-	-	3.75	8	8	8												3.75		3.75	
TKN, lb/day	5274	6208.30	-	-	2044	4003	934	3069												-		3.8	
TS, %															5.06	24.82							
TS, lb/day															33792	33116							
TVS, % TS															77.10	77.10							
TVS, lb/day															26053	25532							

SOLIDS BALANCE
SLUDGE PROCESSING ALTERNATIVE NO. 6
DESIGN DAILY AVERAGE CONDITION

C:\Documents and Settings\nghatala\Desktop\Oneida County\Solids_Balance.xlsx
Lime-Cent (Design ADF)



LEGEND

1 - Plant Influent (Main Flow)
2 - Primary Influent (Main Flow)
3 - Primary Effluent (Main Flow)
4 - Secondary Bypass
5 - Plant Effluent (Main Flow)
6 - Plant Influent (CSO Flow)
7 - CSO Flow to Main Flow Treatment
8 - Primary Influent (CSO Flow)
9 - Plant Effluent (CSO Flow)
10 - Primary Sludge (Main Flow)
11 - Recycle Activated Sludge
12 - Waste Activated Sludge
13 - Primary Sludge (CSO Flow)
14 - Gravity Thickener Feed Sludge
15 - Thickened Sludge
16 - Dewatered Sludge
17 - Lime Stabilized Sludge
18 - Gravity Thickener Overflow
19 - Centrate
20 - Thickener Makeup Water
21 - Lime Feed

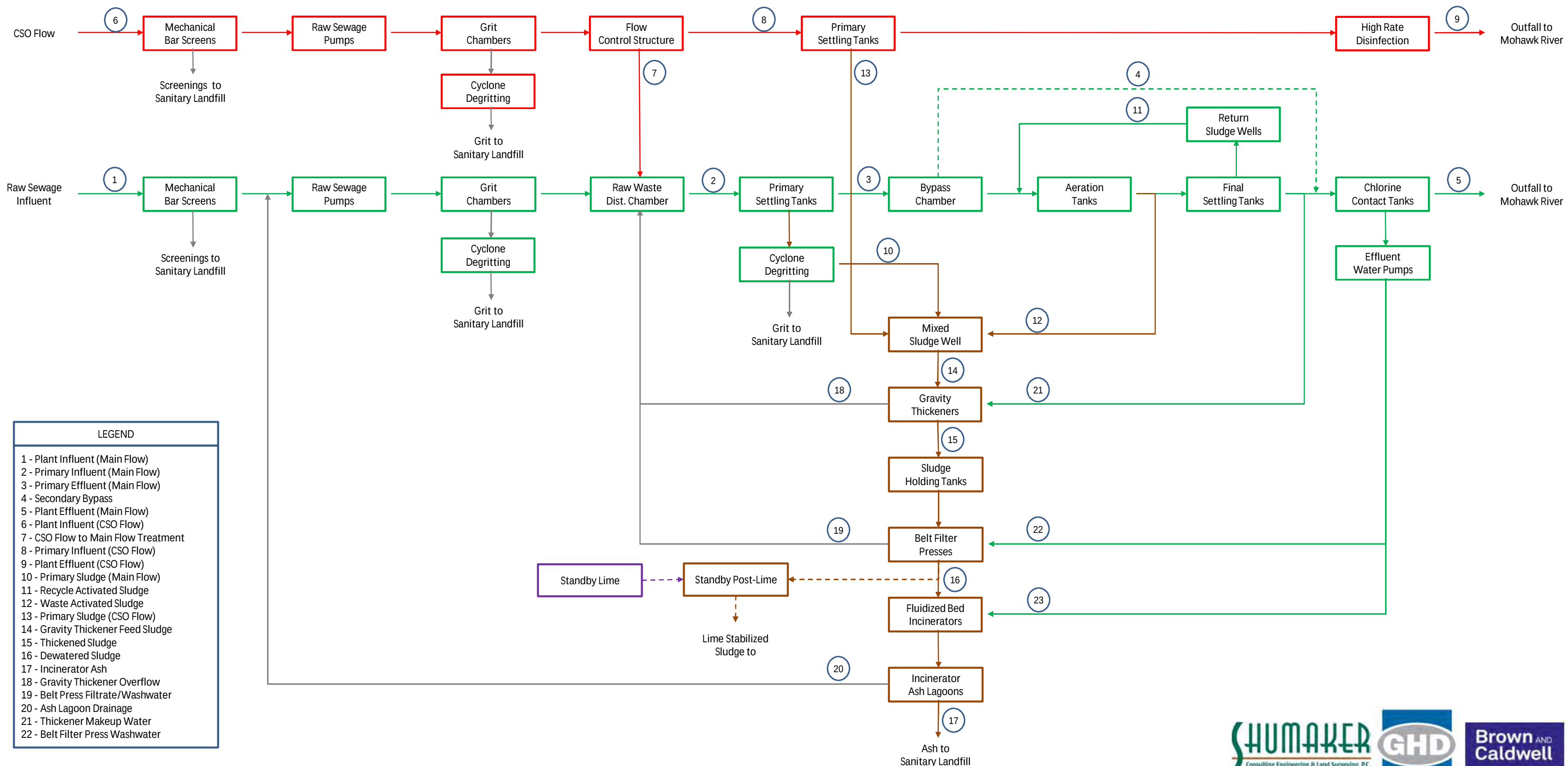
SHUMAKER Consulting Engineering & Land Surveying, P.C.
GHD
Brown AND Caldwell

Flow Stream	1	2	3	4	5	6	7	8	9	10	11	12
Flow, MGD	60.00	66.27	65.76	-	65.33	51	5	46	45.641	0.505	18	0.66
BOD5, mg/L	86.8	-	-	-	9.1	60	60	60				
BOD5, lb/day	43435	-	-	-	4958	25520	2502	23018				
TSS, mg/L	65.1	68.2	36.4	36.4	12.8	50	50	50	27	4207.3	14,144	3,067
TSS, lb/day	32576	37683.85	19964	-	6974	21267	2085	19182	10277	17720	2,123,297	16,881
VSS, mg/L	48.8	51.4	27.4	27.4	9.55	40	40	40	21	3168.6	10,563	2,290
VSS, lb/day	24420	28385.08	15040	-	5203	17014	1668	15346	7994	13345	1,585,718	12,607
TKN, mg/L	12.4	11.8	-	-	3.1	8	8	8				
TKN, lb/day	6205	6538.56	-	-	1689	3403	334	3069				
TS, %												
TS, lb/day												
TVS, % TS												
TVS, lb/day												

Flow Stream	13	14	15	16	17	18	19	20	21	22	23
Flow, MGD	0.359	1.165	0.1	0.019	0.019	1.065	0.201	-		0.12	
BOD5, mg/L											
BOD5, lb/day											
TSS, mg/L	3014.0	4490.0	49,693	256,311	256,311	245.6	502.2	12.8		12.8	
TSS, lb/day	9024	43625	41,444	40,615	40,615	2181	842	-		12.8	
VSS, mg/L	2410.4	3413.8	37,783	194,882		186.7	381.5	9.55		9.55	
VSS, lb/day	7217	33169	31,511	30,881		1658	640	-		9.6	
TKN, mg/L								3.1		3.1	
TKN, lb/day								-		3.1	
TS, %			4.97	25.63							
TS, lb/day			41444	40615							
TVS, % TS			76.03	76.03							
TVS, lb/day			31511	30881							

SOLIDS BALANCE
SLUDGE PROCESSING ALTERNATIVE NO. 6
DESIGN MAXIMUM MONTH CONDITION

C:\Documents and Settings\nghatala\Desktop\Oneida County\Solids_Balance.xlsx
Lime-Cent (Design MMF)



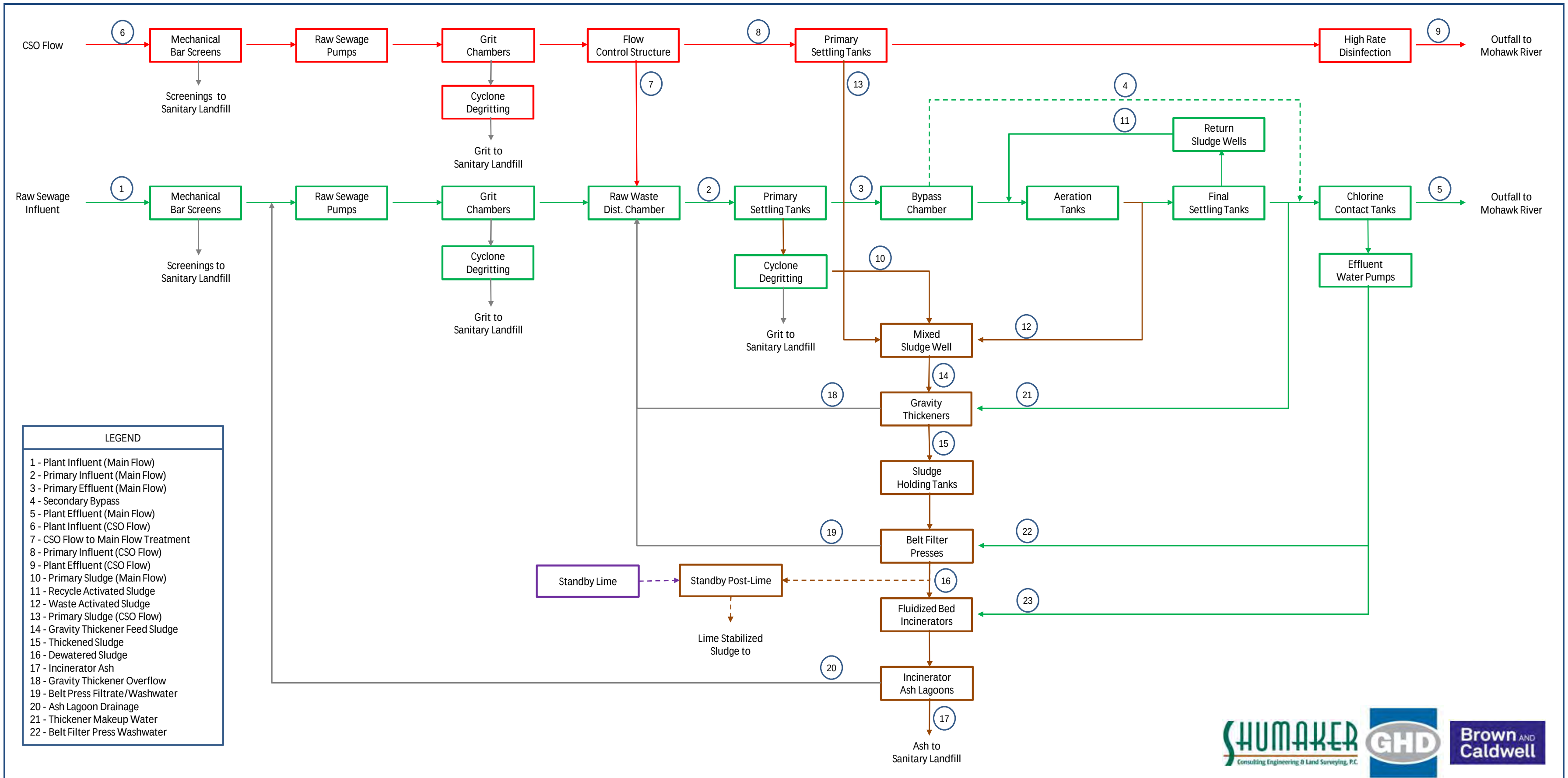
LEGEND

1 - Plant Influent (Main Flow)
2 - Primary Influent (Main Flow)
3 - Primary Effluent (Main Flow)
4 - Secondary Bypass
5 - Plant Effluent (Main Flow)
6 - Plant Influent (CSO Flow)
7 - CSO Flow to Main Flow Treatment
8 - Primary Influent (CSO Flow)
9 - Plant Effluent (CSO Flow)
10 - Primary Sludge (Main Flow)
11 - Recycle Activated Sludge
12 - Waste Activated Sludge
13 - Primary Sludge (CSO Flow)
14 - Gravity Thickener Feed Sludge
15 - Thickened Sludge
16 - Dewatered Sludge
17 - Incinerator Ash
18 - Gravity Thickener Overflow
19 - Belt Press Filtrate/Washwater
20 - Ash Lagoon Drainage
21 - Thickener Makeup Water
22 - Belt Filter Press Washwater

Flow Stream	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
Flow, MGD	51.00	67.53	67.03	-	66.27	60	14	46	45.641	0.505	15	0.63	0.359	1.494	0.1	0.02		1.394	0.2	0.94	-	0.12	0.94
BOD5, mg/L	78.6	-	-	-	12.4	60	60	60													12.4		
BOD5, lb/day	33432	-	-	-	6853	30024	7006	23018													-		
TSS, mg/L	60.3	71.6	56.0	56.0	21.94	50	50	50	27	2142.6	14,467	2721.3	6324.5	3391.5	48,135	235,863		181.7	494.6	750	21.94	21.94	21.94
TSS, lb/day	25648	40,304	31280	-	12126	25020	5838	19182	10277	9024	1,809,822	14298	18936	42258	40,145	39,342	663	2113	825	5880	-	22.0	172.0
VSS, mg/L	48.8	48.7	36.2	36.2	14.76	40	40	40	21	1713.6	9,732	1830.5	4067.7	2328.6	33,050	161,948		124.7	339.2		14.76	14.76	14.76
VSS, lb/day	20757	27,443	20226	-	8158	20016	4670	15346	7994	7217	1,217,473	9618	12179	29014	27,564	27,013		1450	566			14.8	115.7
TKN, mg/L	12.4	11.0	-	-	4.23	8	8	8													4.23	4.23	4.23
TKN, lb/day	5274	6,208	-	-	2338	4003	934	3069														4.2	33.2
TS, %															4.81	23.59							
TS, lb/day															40145	39342							
TVS, % TS															68.66	68.66							
TVS, lb/day															27564	27013							

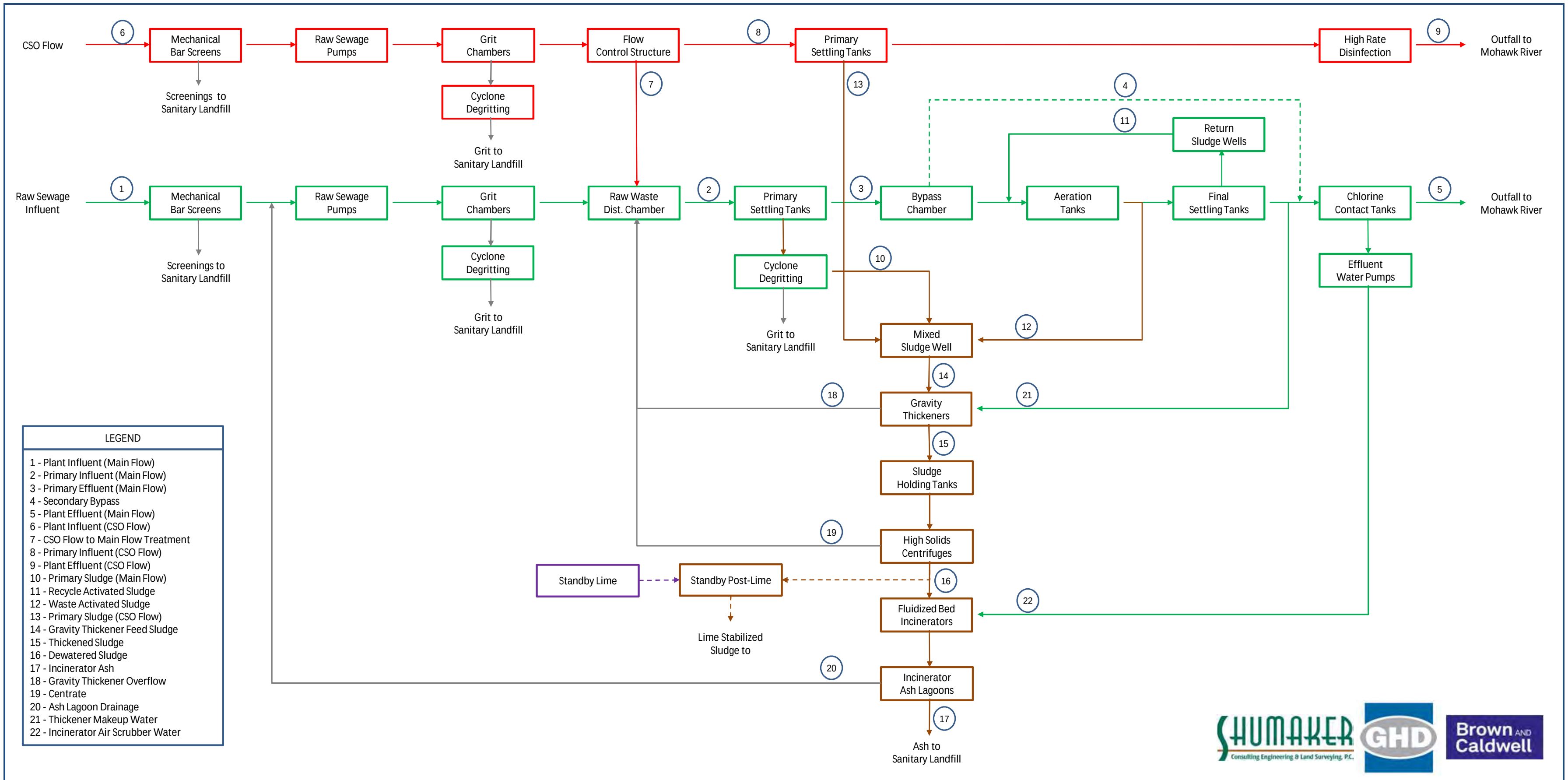
SOLIDS BALANCE
SLUDGE PROCESSING ALTERNATIVE NO. 7
DESIGN DAILY AVERAGE CONDITIONS

C:\Documents and Settings\nghatala\Desktop\Oneida County\Solids_Balance.xlsx
Incin-BFP (Design ADF) (2)



Flow Stream	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
Flow, MGD	60.00	67.73	67.23	-	66.27	51	5	46	45.641	0.505	18	0.84	0.359	1.704	0.115	0.03		1.589	0.205	0.94	-	0.12	0.94
BOD5, mg/L	86.8	-	-	-	9.59	60	60	60													9.59		
BOD5, lb/day	43435	-	-	-	5300	25520	2502	23018													-		
TSS, mg/L	65.1	78.0	41.6	41.6	12.95	50	50	50	27	4914.9	13,895	2980.2	3014.0	3560.7	50,121	188,289		191.0	569.7	750	12.95	12.95	12.95
TSS, lb/day	32576	44,046	23346	-	7157	21267	2085	19182	10277	20700	2,085,917	20878	9024	50602	48,071	47,110	808	2531	974	5880	-	13.0	101.5
VSS, mg/L	48.8	50.4	26.9	26.9	8.6	40	40	40	21	3178.5	9,230	1979.6	2410.4	2425.7	34,146	128,273		130.0	388.1		8.6	8.6	8.6
VSS, lb/day	24420	28,474	15087	-	4753	17014	1668	15346	7994	13387	1,385,608	13868	7217	34472	32,749	32,094		1723	664			8.6	67.4
TKN, mg/L	12.4	11.6	-	-	3.5	8	8	8													3.5	3.5	3.5
TKN, lb/day	6205	6,539	-	-	1934	3403	334	3069														3.5	27.4
TS, %															5.01	18.83							
TS, lb/day															48071	47110							
TVS, % TS															68.13	68.13							
TVS, lb/day															32749	32094							

SOLIDS BALANCE
SLUDGE PROCESSING ALTERNATIVE NO. 7
DESIGN MAXIMUM MONTH CONDITION



Flow Stream	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	
Flow, MGD	51.00	67.53	67.03	-	66.28	60	14	46	45.641	0.505	15	0.63	0.359	1.494	0.1	0.02		1.394	0.2	0.94	-	0.12	0.94
BOD5, mg/L	78.6	-	-	-	12.4	60	60	60													12.4		
BOD5, lb/day	33432	-	-	-	6854	30024	7006	23018													-		
TSS, mg/L	60.3	71.6	38.2	38.2	21.95	50	50	50	27	4495.8	14,467	2721.1	3014.0	3391.3	48,134	235,857		181.7	494.6	750	21.95	21.95	21.95
TSS, lb/day	25648	40,303	21368	-	12133	25020	5838	19182	10277	18935	1,809,822	14297	9024	42256	40,144	39,341	661	2112	825	5880	-	22.0	172.1
VSS, mg/L	48.8	48.7	27.3	27.3	14.76	40	40	40	21	2891.7	9,723	1830.5	2410.4	2328.6	33,049	161,942		124.8	339.2		14.76	14.76	14.76
VSS, lb/day	20757	27,444	15265	-	8159	20016	4670	15346	7994	12179	1,216,347	9618	7217	29014	27,563	27,012		1451	566			14.8	115.7
TKN, mg/L	12.4	11.0	-	-	4.23	8	8	8													4.23	4.23	4.23
TKN, lb/day	5274	6,208	-	-	2338	4003	934	3069														4.2	33.2
TS, %															4.81	23.59							
TS, lb/day															40144	39341							
TVS, % TS															68.66	68.66							
TVS, lb/day															27563	27012							

SOLIDS BALANCE
SLUDGE PROCESSING ALTERNATIVE NO. 8
DESIGN DAILY AVERAGE CONDITION

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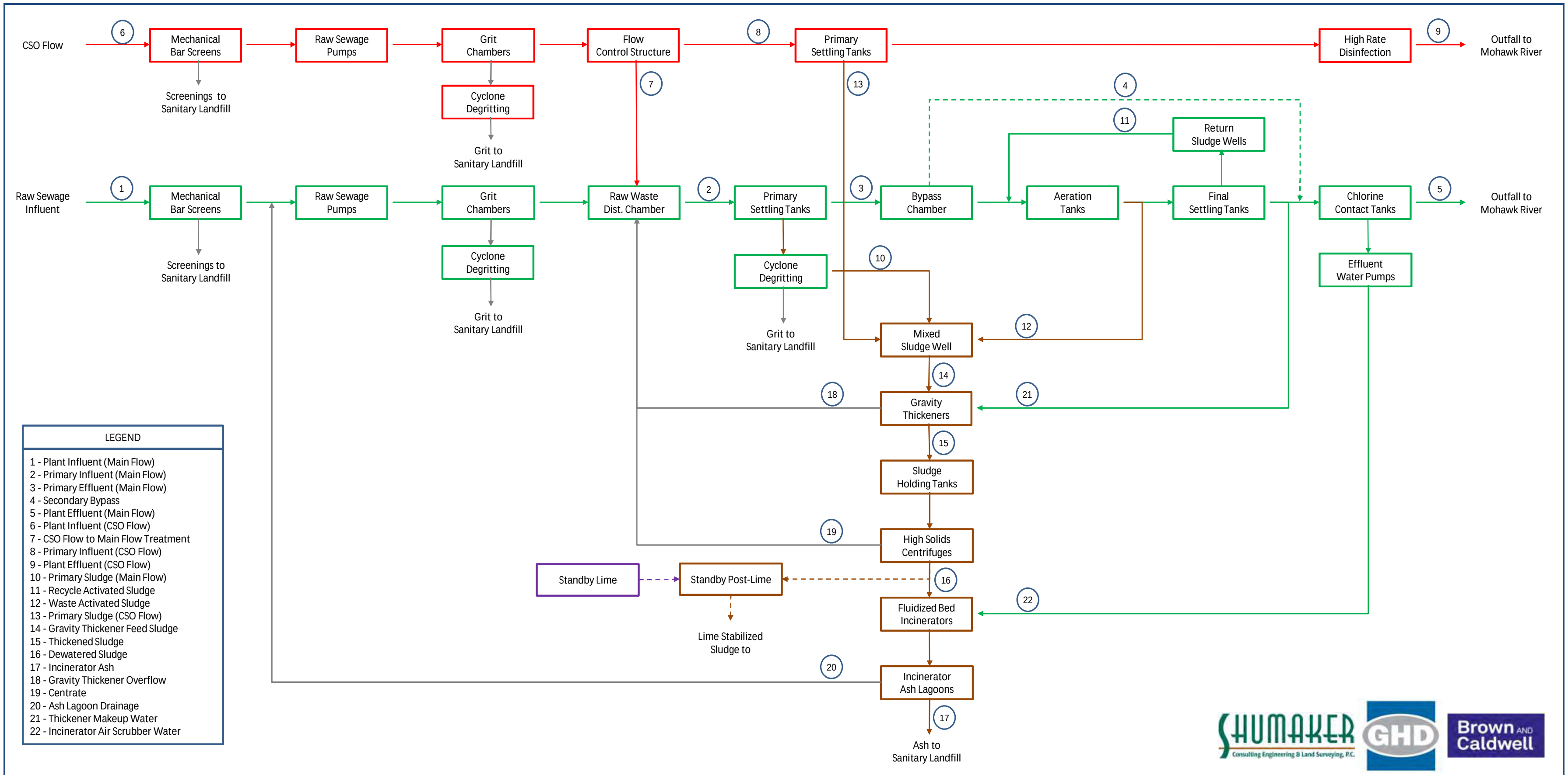
Incin-Cent (Design ADF) (2)



Flow Stream	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	
Flow, MGD	60.00	67.74	67.24	-	66.27	51	5	46	45.641	0.505	18	0.84	0.359	1.704	0.12	0.02		1.584	0.22	0.94	-	0.12	0.94
BOD5, mg/L	86.8	-	-	-	9.59	60	60	60													9.59		
BOD5, lb/day	43435	-	-	-	5300	25520	2502	23018													-		
TSS, mg/L	65.1	78.0	41.6	41.6	12.95	50	50	50	27	4914.9	13,895	2979.9	3014.0	3560.5	48,032	282,422		191.5	531.4	750	12.95	12.95	12.95
TSS, lb/day	32576	44,046	23346	-	7157	21267	2085	19182	10277	20700	2,085,917	20876	9024	50600	48,070	47,108	807	2530	975	5880	-	13.0	101.5
VSS, mg/L	48.8	50.4	26.9	26.9	8.6	40	40	40	21	3178.5	9,230	1979.4	2410.4	2425.6	32,722	192,404		130.4	361.7		8.6	8.6	8.6
VSS, lb/day	24420	28,474	15087	-	4753	17014	1668	15346	7994	13387	1,385,608	13867	7217	34471	32,748	32,093		1723	664			8.6	67.4
TKN, mg/L	12.4	11.6	-	-	3.54	8	8	8													3.54	3.54	3.54
TKN, lb/day	6205	6,539	-	-	1957	3403	334	3069														3.5	27.8
TS, %															4.80	28.24							
TS, lb/day															48070	47108							
TVS, % TS															68.13	68.13							
TVS, lb/day															32748	32093							

SOLIDS BALANCE
SLUDGE PROCESSING ALTERNATIVE NO. 8
DESIGN MAXIMUM MONTH CONDITION

APPENDIX D-2

PRELIMINARY BASIS OF DESIGN

SLUDGE PROCESSING ALTERNATIVES

PRELIMINARY BASIS OF DESIGN
SLUDGE PROCESSING ALTERNATIVES
WPCP and Saquoit Creek Pump Station Evaluation
Oneida County, New York

	Sludge Processing Alternative							
	1	2	3	4	5	6	7	8
Sludge Production								
Primary Sludge (Main Flow)								
Daily Average Conditions								
Flow, MGD	0.45	0.45	0.38	0.38	0.38	0.38	0.45	0.45
TS, %	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
TS, lb/day	18,900	18,900	15,800	15,800	16,000	16,000	18,900	18,900
TVS, % TS	65	65	76	76	76	76	65	65
TVS, lb/day	12,200	12,200	12,000	12,000	12,100	12,100	12,200	12,200
Maximum Month Conditions								
Flow, MGD	0.50	0.50	0.42	0.42	0.42	0.42	0.50	0.50
TS, %	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
TS, lb/day	20,700	20,700	17,600	17,600	17,700	17,700	20,700	20,700
TVS, % TS	65	65	75	75	75	75	65	65
TVS, lb/day	13,400	13,400	13,200	13,200	13,300	13,300	13,400	13,400
Primary Sludge (CSO Flow)								
Daily Average Conditions								
Flow, MGD	0.22	0.22	0.22	0.22	0.22	0.22	0.22	0.22
TS, %	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
TS, lb/day	9,000	9,000	9,000	9,000	9,000	9,000	9,000	9,000
TVS, % TS	80	80	80	80	80	80	80	80
TVS, lb/day	7,200	7,200	7,200	7,200	7,200	7,200	7,200	7,200
Maximum Month Conditions								
Flow, MGD	0.22	0.22	0.22	0.22	0.22	0.22	0.22	0.22
TS, %	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
TS, lb/day	9,000	9,000	9,000	9,000	9,000	9,000	9,000	9,000
TVS, % TS	80	80	80	80	80	80	80	80
TVS, lb/day	7,200	7,200	7,200	7,200	7,200	7,200	7,200	7,200
Waste Activated Sludge								
Daily Average Conditions								
Flow, MGD	0.57	0.57	0.42	0.42	0.42	0.42	0.57	0.57
TS, %	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
TS, lb/day	14,300	14,300	10,600	10,600	10,600	10,600	14,300	14,297
TVS, % TS	67	67	76	76	76	76	67	67
TVS, lb/day	9,600	9,600	8,100	8,100	8,100	8,100	9,600	9,600
Maximum Month Conditions								
Flow, MGD	0.84	0.84	0.68	0.68	0.68	0.68	0.84	0.84
TS, %	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
TS, lb/day	20,900	20,900	17,000	17,000	16,900	16,900	20,900	20,900
TVS, % TS	67	67	75	75	75	75	67	67
TVS, lb/day	13,900	13,900	12,700	12,700	12,600	12,600	13,900	13,900
Mixed Sludge								
Daily Average Conditions								
Flow, MGD	1.24	1.24	1.02	1.02	1.02	1.02	1.24	1.24
TS, %	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
TS, lb/day	42,200	42,200	35,400	35,400	35,600	35,600	42,200	42,197
TVS, % TS	69	69	77	77	77	77	69	69
TVS, lb/day	29,000	29,000	27,300	27,300	27,400	27,400	29,000	29,000
Maximum Month Conditions								
Flow, MGD	1.56	1.56	1.32	1.32	1.32	1.32	1.56	1.56
TS, %	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
TS, lb/day	50,600	50,600	43,600	43,600	43,600	43,600	50,600	50,600
TVS, % TS	68	68	76	76	76	76	68	68
TVS, lb/day	34,500	34,500	33,100	33,100	33,100	33,100	34,500	34,500

PRELIMINARY BASIS OF DESIGN
SLUDGE PROCESSING ALTERNATIVES
WPCP and Saquoit Creek Pump Station Evaluation
Oneida County, New York

	Sludge Processing Alternative							
	1	2	3	4	5	6	7	8
Gravity Sludge Thickening								
Number of gravity thickeners								
Service	3	3	3	3	3	3	3	3
Standby	1	1	1	1	1	1	1	1
Total	4	4	4	4	4	4	4	4
Thickener diameter, feet	55	55	55	55	55	55	55	55
Thickener side water depth, feet	10	10	10	10	10	10	10	10
Thickener surface area, ft2	2,376	2,376	2,376	2,376	2,376	2,376	2,376	2,376
Total surface area (in service), ft2	7,127	7,127	7,127	7,127	7,127	7,127	7,127	7,127
Hydraulic loading, gal/day/ft2								
Daily average	174	174	143	143	143	143	174	174
Maximum month	219	219	185	185	185	185	219	219
Solids loading, lb/day/ft2								
Daily average	5.9	5.9	5.0	5.0	5.0	5.0	5.9	5.9
Maximum month	7.1	7.1	6.1	6.1	6.1	6.1	7.1	7.1
Solids capture efficiency, %	95	95	95	95	95	95	95	95
Thickened sludge production								
Daily Average Conditions								
Flow, gal/day	96,000	96,000	81,000	81,000	81,000	81,000	96,000	96,000
TS, %	5	5	5	5	5	5	5	5
TS, lb/day	40,100	40,100	33,600	33,600	33,800	33,800	40,100	40,100
TVS, % TS	69	69	77	77	77	77	69	69
TVS, lb/day	27,600	27,600	25,900	25,900	26,000	26,000	27,600	27,600
Maximum Month Conditions								
Flow, gal/day	115,000	115,000	99,000	99,000	99,000	99,000	115,000	115,000
TS, %	5	5	5	5	5	5	5	5
TS, lb/day	48,100	48,100	41,400	41,400	41,400	41,400	48,100	48,100
TVS, % TS	68	68	76	76	76	76	68	68
TVS, lb/day	32,800	32,800	31,400	31,400	31,400	31,400	32,800	32,800
Anaerobic Sludge Digestion								
Number of primary digesters			2	2				
Number of secondary digesters			1	1				
Digester diameter, feet			75	75				
Digester side water depth, feet			30	30				
Primary digester volume, ft3			265,072	265,072				
Primary digester volume, MG			2.0	2.0				
Digester detention time, days								
Daily average			24	24				
Maximum month			20	20				
Volatile solids loading, lb/day/ft3								
Daily average			0.10	0.10				
Maximum month			0.12	0.12				
Volatile solids reduction, %								
Daily average			52	52				
Maximum month			50	50				
Digested sludge production								
Daily Average Conditions								
Flow, gal/day			81,000	81,000				
TS, %			3	3				
TS, lb/day			20,100	20,100				
TVS, % TS			62	62				
TVS, lb/day			12,400	12,400				
Maximum Month Conditions								
Flow, gal/day			99,000	99,000				
TS, %			3	3				
TS, lb/day			25,700	25,700				
TVS, % TS			61	61				
TVS, lb/day			15,700	15,700				

PRELIMINARY BASIS OF DESIGN
SLUDGE PROCESSING ALTERNATIVES
WPCP and Saquoit Creek Pump Station Evaluation
Oneida County, New York

	Sludge Processing Alternative							
	1	2	3	4	5	6	7	8
Belt Filter Press Sludge Dewatering								
Number of belt filter presses								
Service	4		5		5		4	
Standby	2		1		1		2	
Effective belt width, meters	2		2		2		2	
Weekly dewatering time, hours	168		35		60		168	
Solids throughput, lb/hr/m								
Maximum	700		700		700		700	
Daily average	209		402		394		209	
Maximum month	251		514		483		251	
Solids Capture Efficiency, %	98		98		98		98	
Dewatered sludge production								
Daily Average Conditions								
Wet tons/hour	4.1		9.8		9.7		4.1	
TS, %	20		20		20		20	
TS, lb/hour	1,637		3,940		3,864		1,637	
TVS, % TS	69		62		77		69	
TVS, lb/hour	1,125		2,430		2,974		1,125	
Maximum Month Conditions								
Wet tons/hour	4.9		12.6		11.8		4.9	
TS, %	20		20		20		20	
TS, lb/hour	1,964		5,037		4,733		1,964	
TVS, % TS	68		61		76		68	
TVS, lb/hour	1,339		3,077		3,593		1,339	
Centrifuge Sludge Dewatering								
Number of centrifuges								
Service		2		2		2		2
Standby		1		1		1		1
Total		3		3		3		3
Weekly dewatering time, hours		168		35		60		168
Centrifuge sludge feed rate, gpm								
Daily average		33		135		79		33
Maximum month		40		165		96		40
Solids throughput, lb/hr								
Daily average		835		2,010		1,972		835
Maximum month		1,002		2,570		2,415		1,002
Solids Capture Efficiency, %		98		98		98		98
Dewatered sludge production								
Daily Average Conditions								
Wet tons/hour		3.4		8.2		8.1		3.4
TS, %		24		24		24		24
TS, lb/hour		1,637		3,940		3,864		1,637
TVS, % TS		69		62		77		69
TVS, lb/hour		1,125		2,430		2,974		1,125
Maximum Month Conditions								
Wet tons/hour		4.1		10.5		9.9		4.1
TS, %		24		24		24		24
TS, lb/hour		1,964		5,037		4,733		1,964
TVS, % TS		68		61		76		68
TVS, lb/day		1,339		3,077		3,593		1,339

PRELIMINARY BASIS OF DESIGN
SLUDGE PROCESSING ALTERNATIVES
WPCP and Saquoit Creek Pump Station Evaluation
Oneida County, New York

	Sludge Processing Alternative							
	1	2	3	4	5	6	7	8
Sludge Incineration								
Number of incinerators								
Service	2	2					2	2
Standby	1	1					-	-
Total	3	3					2	2
Weekly operating time, hours	168	168					168	168
Annual operating time, weeks/year	52	52					48	48
Incinerator charging rate, lb DS/hr								
Capacity, lb/hour								
Daily average, lb/hour	819	819					819	1,637
Maximum month, lb/hour	982	982					982	1,964
Post-Lime Sludge Stabilization								
Weekly operating time, hours					60	60		
Annual operating time, weeks/year					52	52	4	4
Lime dosage, lb/wet ton					100	100	100	100
Lime feed, lb/hour								
Capacity, lb/hour								
Daily average, lb/hour					966	805		
Maximum month, lb/hour					1,183	986		
Landfill Disposal of Sludge/Ash								
Wet tons per year	4,474	4,474	17,925	14,938	31,650	26,375	6,565	6,159
TS, %	50	50	20	24	24	28	40	50
TS, dry tons per year	2,237	2,237	3,585	3,585	7,536	7,285	2,645	2,625
TVS, % TS	-	-	62	62	62	64	13	14
TVS, dry tons per year	-	-	2,212	2,212	4,640	4,640	357	357

APPENDIX D-3

CAPITAL AND ANNUAL O&M COST ESTIMATES

SLUDGE PROCESSING ALTERNATIVES

ESTIMATED CAPITAL COSTS FOR SLUDGE PROCESSING ALTERNATIVES

	Estimated Capital Costs for Sludge Processing Alternatives (August 2012 dollars)							
	Alternative 1	Alternative 2	Alternative 3	Alternative 4	Alternative 5	Alternative 6	Alternative 7	Alternative 8
Gravity Sludge Thickening								
Replace sludge scraper mechanism and drive in Gravity Thickener No. 4	$ 200,000	$ 200,000	$ 200,000	$ 200,000	$ 200,000	$ 200,000	$ 200,000	$ 200,000
Replace overflow weirs, grating and handrails in Gravity Thickener No. 4	$ 25,000	$ 25,000	$ 25,000	$ 25,000	$ 25,000	$ 25,000	$ 25,000	$ 25,000
Anaerobic Sludge Digestion								
Site work			$ 350,000	$ 350,000				
Concrete construction for digesters and digester piping galleries			$ 1,850,000	$ 1,850,000				
Furnish and install digester covers			$ 1,300,000	$ 1,300,000				
Furnish and install digester heating and mixing equipment			$ 2,000,000	$ 2,000,000				
Furnish and install digester gas collection and safety equipment			$ 700,000	$ 700,000				
Furnish and install digester gas cleaning and cogeneration equipment			$ 4,600,000	$ 4,600,000				
Sludge pumping & piping modifications			$ 500,000	$ 500,000				
Digester building, including heating, ventilation and plumbing			$ 3,100,000	$ 3,100,000				
Belt Filter Press/Centrifuge Sludge Dewatering								
Demolish existing belt filter press sludge dewatering equipment	$ 300,000	$ 450,000	$ 300,000	$ 450,000	$ 300,000	$ 450,000	$ 300,000	$ 450,000
Furnish and install new belt filter press sludge dewatering equipment	$ 1,200,000		$ 1,200,000		$ 1,200,000		$ 1,200,000	
Furnish and install new centrifuge sludge dewatering equipment		$ 1,800,000		$ 1,800,000		$ 1,800,000		$ 1,800,000
Furnish and install sludge feed pumps and grinders		$ 150,000		$ 150,000		$ 150,000		$ 150,000
Furnish and install bridge crane and grated operating platforms		$ 80,000		$ 80,000		$ 80,000		$ 80,000
Furnish and install dewatered sludge conveyors	$ 380,000	$ 380,000	$ 380,000	$ 380,000	$ 380,000	$ 380,000	$ 380,000	$ 380,000
Furnish and install dewatered sludge pumps	$ 1,300,000	$ 1,100,000	$ 1,300,000	$ 1,100,000	$ 1,300,000	$ 1,100,000	$ 1,300,000	$ 1,100,000
Modifications to inside process piping and valves	$ 150,000	$ 350,000	$ 150,000	$ 350,000	$ 150,000	$ 350,000	$ 150,000	$ 350,000
Fluidized Bed Sludge Incineration								
Demolish Fluidized Bed Incinerator No. 2	$ 400,000	$ 400,000	$ 400,000	$ 400,000	$ 400,000	$ 400,000	$ 400,000	$ 400,000
Furnish and install new fluidized bed incinerator (incl mercury reduction)	$ 20,000,000	$ 20,000,000						
Modifications to Fluidized Bed Incinerators No. 1 and 3	$ 1,500,000	$ 1,500,000					$ 1,500,000	$ 1,500,000
Building Modification/ Structural Steel/Ductwork/Joints	$ 675,000	$ 675,000					$ 725,000	$ 725,000
New water pumps/ash slurry system/CEMS and performance testing	$ 425,000	$ 425,000					$ 350,000	$ 350,000
Mercury Reduction System on Units 1 and 3 (1)	$ 3,750,000	$ 3,750,000					$ 3,750,000	$ 3,750,000
Post-Lime Sludge Stabilization								
Site work					$ 250,000	$ 250,000	$ 250,000	$ 250,000
Furnish and install lime stabilization equipment (silo, feeder, mixer)					$ 2,000,000	$ 2,000,000	$ 750,000	$ 750,000
Inside process piping					$ 250,000	$ 250,000	$ 125,000	$ 125,000
Building modifications					$ 250,000	$ 250,000	$ 125,000	$ 125,000
Subtotal	$ 30,305,000	$ 31,285,000	$ 18,355,000	$ 19,335,000	$ 6,705,000	$ 7,685,000	$ 11,530,000	$ 12,510,000
Electrical, instrumentation and controls	$ 4,600,000	$ 4,700,000	$ 2,800,000	$ 3,000,000	$ 1,100,000	$ 1,200,000	$ 1,800,000	$ 1,900,000
Temporary sludge dewatering/stabilization/disposal	$ 2,200,000	$ 2,200,000	$ 2,200,000	$ 2,200,000	$ 2,200,000	$ 2,200,000	$ 2,200,000	$ 2,200,000
Contractor mobilization, bonds, insurance and general conditions	$ 3,195,000	$ 3,315,000	$ 2,045,000	$ 2,165,000	$ 895,000	$ 915,000	$ 1,370,000	$ 1,490,000
Subtotal	$ 40,300,000	$ 41,500,000	$ 25,400,000	$ 26,700,000	$ 10,900,000	$ 12,000,000	$ 16,900,000	$ 18,100,000
Contingency allowance	$ 8,100,000	$ 8,300,000	$ 5,100,000	$ 5,400,000	$ 2,200,000	$ 2,400,000	$ 3,400,000	$ 3,700,000
Estimated Construction Cost	$ 48,400,000	$ 49,800,000	$ 30,500,000	$ 32,100,000	$ 13,100,000	$ 14,400,000	$ 20,300,000	$ 21,800,000
Allowance for engineering, legal and administrative costs (20%)	$ 9,700,000	$ 10,000,000	$ 6,100,000	$ 6,500,000	$ 2,700,000	$ 2,900,000	$ 4,100,000	$ 4,400,000
Estimated Project Cost	**$ 58,100,000**	**$ 59,800,000**	**$ 36,600,000**	**$ 38,600,000**	**$ 15,800,000**	**$ 17,300,000**	**$ 24,400,000**	**$ 26,200,000**

(1) May not be required depending on stack testing results

ESTIMATED ANNUAL OPERATING AND MAINTENANCE COSTS FOR SLUDGE PROCESSING ALTERNATIVES

	Estimated Annual Cost, $/year											
	Alternate 1		Alternate 2						Alternate 7		Alternate 8	
	1A	1B	2A	2B	Alternate 3	Alternate 4	Alternate 5	Alternate 6	7A	7B	8A	8B
Dewatered Sludge/Ash Disposal	$ 330,000	$ 330,000	$ 330,000	$ 330,000	$ 1,290,000	$ 1,080,000	$ 2,300,000	$ 1,900,000	$ 480,000	$ 480,000	$ 450,000	$ 450,000
Utilities												
Fuel Oil	$ 1,100,000		$ 350,000						$ 1,020,000		$ 330,000	
Natural Gas		$ 500,000		$ 150,000						$ 460,000		$ 140,000
Electric (Energy Recovery)					$ (195,000)	$ (195,000)						
Electric (Sludge Dewatering)	$ 50,000	$ 50,000	$ 90,000	$ 90,000	$ 15,000	$ 40,000	$ 25,000	$ 70,000	$ 50,000	$ 50,000	$ 90,000	$ 90,000
Chemicals												
Lime							$ 420,000	$ 350,000			$ 30,000	$ 25,000
Miscellaneous												
Stack Testing	$ 75,000	$ 75,000	$ 75,000	$ 75,000					$ 75,000	$ 75,000	$ 75,000	$ 75,000
Wet Scrubber Replacement	$ 100,000	$ 100,000	$ 100,000	$ 100,000					$ 100,000	$ 100,000	$ 100,000	$ 100,000
Maintenance and Training	$ 110,000	$ 110,000	$ 110,000	$ 110,000					$ 110,000	$ 110,000	$ 110,000	$ 110,000
Totals	**$ 1,765,000**	**$ 1,165,000**	**$ 1,055,000**	**$ 855,000**	**$ 1,110,000**	**$ 925,000**	**$ 2,745,000**	**$ 2,320,000**	**$ 1,835,000**	**$ 1,275,000**	**$ 1,185,000**	**$ 990,000**

APPENDIX E

WPCP ELECTRICAL EVALUATION

ELECTRICAL EVALUATION STUDY

1.0 STATEMENT OF PURPOSE OF STUDY

The purpose of the study is to evaluate portions of the existing electrical distribution system at the facility to determine condition, expected remaining life, future load capacity and capability for future expansion. The evaluation is limited to the outdoor main unit substation, interior unit substations, motor control centers, power transformers, main distribution switchgears and the medium voltage generator. All branch circuit panelboards, instrumentation and control equipment are not included in the study. Recommendations are provided for existing equipment based on the evaluation within the study. The plant will be expanded in the future based on the split flow concept design. Recommendations and options for electrical expansion are provided. Opinion of probable construction costs are included for replacement electrical equipment and electrical expansion based on recommendations provided. One-Line diagrams are provided for the existing electrical system and the proposed expansion options.

SECTION 2.0

EXISTING ELECTRICAL EQUIPMENT ASSESSMENT

2.0 EXISTING ELECTRICAL EQUIPMENT ASSESSMENT

2.0.1 **Existing Outdoor Main Unit Substation** **-** Power Transformers, 46kV Group Operated Air Break Switches, Vertical Mount Disconnectable Power Fuses, Lightning Arrestor and 4160Y/2400V-Metal-Clad Switchgear Lineup

Existing Outdoor Main Unit Substation
46kV to 4160V

2.0.1.1 Existing Power Transformers

46kV to 4160V Power Transformer

Existing Power Transformer (#1):

Age: 1969
Make: General Electric
Serial No: F- 961747
Size: 46kV DELTA Primary, 4160Y/2400V WYE Secondary, Class OA/FA
3750 kVA @ 55°C Rise OA
4687 kVA @ 55°C Rise FA
5250 kVA @ 65°C Rise FA
Impedance (Z): 6.80%
Condition: Fair, some parts appear to be rusted.
Existing Load: 3297 kVA
Available Load Capacity for Future Load: Approximately 1950kVA
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

Existing Power Transformer (#2):

Age: 1970
Make: General Electric
Serial No: H-879075
Size: 46kV DELTA Primary, 4160Y/2400V WYE Secondary, Class OA/FA
3750 kVA @ 55°C Rise OA
4687 kVA @ 55°C Rise FA
5250 kVA @ 65°C Rise FA
Impedance (Z): 6.71%
Condition: Fair
Existing Load: 3297 kVA
Available Load Capacity for Future Load: Approximately 1950kVA
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

2.0.1.2 **<u>Existing 46 kV Receiving Structures</u>** – 46kV Group Operated Air Break Switches (GOAB), Vertical Mount Disconnectable Power Fuses and Lightning Arrestor

46kVGroup Operated Air Break Switches, Vertical Mount Disconnectable Power Fuses and Lightning Arrestor

<u>Existing 46kV GOAB:</u>

Age: Approximately over 40 years old
Make and Model: Unknown
Condition: Fair
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

<u>Existing Power Fuses:</u>

Age: Approximately over 40 years old
Make: S&C
Model: SMD-2C, 100E (Std. speed)
Condition: Fair
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

<u>Existing Lightning Arrestor:</u>

Age: Approximately over 40 years old
Make and Model: Unknown
Condition: Fair
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

2.0.1.3 Existing 4160Y/2400V Metal-Clad Switchgear

4160Y/2400V Metal-Clad Switchgear Lineup

Age: Approximately over 40 years old
Make: GE (Air Circuit Breakers) Distribution Sections / ABB (Vacuum Breakers) Main Sections
Model: GE AMH-4.76-250-1D / ABB 5 VHK-R (AMH-4.76-250)
Date Manufactured: GE Switchgear and Breakers Mar-70 / ABB Breakers Sep-97
Size: 4160Y/2400V, 1200A Bus, 3P/3W
Condition: Fair
Future Available Space Capacity: None
Existing Load: 3297 kVA
Available Load Capacity for Future Load: Approximately 1950kVA
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

2.0.1.4 Existing Main Unit Substation 46kV to 4160V – Summary

The existing outdoor main unit substation appears to be in fair and working condition. It is estimated to be over 40 years old and is expected to be near the end of its useful life. The arrangement of the existing outdoor main unit substation includes two utility receiving structures (both fed from the same structure) with 46kVgroup operated air break switches, vertical mounted disconnectable power fuses, lightning arrestors, 46kV to 4160Y/2400V power transformers and a metal-clad switchgear lineup. The metal-clad switchgear lineup consists of two main drawout/vacuum circuit breakers cubicle sections with protective devices, two utility metering compartments, six distribution drawout/air circuit breaker cubicle sections with protective devices, one generator air circuit breaker section with automatic start functions and protective devices, two auxiliary CT and PT sections. The two group operated air break switches and vertical mounted disconnectable power fuses that feed and protect the power transformers are fed from the existing 46kV overhead utility (National Grid) service. The power transformers high side bushings are connected to the incoming utility receiving structures via circular overhead buses. Transition sections are provided on the 4160V transformer sides and are close-coupled to the metal-clad switchgear. The secondary sides of the power transformers are equipped with on-load tap changers with 32 steps for voltage regulation. Forced air fans are provided for a transformer continuous max rating at 65 degrees C rise. Under normal operation, Transformer #1 and Transformer #2 are energized, Main #1 breaker is closed, Main #2 breaker is open and all feeder breakers are closed. Under emergency condition and loss of the normal power source via Transformer #1 (Source #1); the SEL-351 protective relays will switchover to Transformer #2 (Standby Source #2). The control room remote control panel with indication lights will show the main breakers positions; green lights indicate breakers are open, red lights indicate breakers are closed and blue lights indicate the availability of Source #1 and Source #2. The existing 2250kW, 4160V standby generator provides emergency power to the entire facility in the event of losing commercial power. Both relay sources (Source #1 and Source #2) will sense loss of potential and send a signal to the standby generator to start and trip the main breakers (Main#1 and Main#2). The emergency generator starts and verifies the frequency and voltage and that the main source breakers are open before closing the generator breaker. A peak demand of 3297 kVA was recorded by the utility company over the past 2 years. Based on this peak kVA demand each of the two power transformers have approximately an additional 1950kVA @0.8PF or 37% of its max continuous transformer rating (5250kVA) load capacity remaining. However only one power transformer is operational at any given time. The existing 4160V metal clad switchgear lineup currently has no additional breaker or cubicle spaces available for future loads. However, the lineup can handle the additional load to be realized in the split flow project. The projected additional load for the split flow project is estimated to be 750 kVA. The main breakers (Main #1 and Main #2) CT's are rated for the maximum transformers continuous rating (5250kVA).

2.0.2 **<u>Existing Administration Building</u>** – Double Ended Unit Substation (4160V Voltage Primary Switches, 4160V to 480Y/277V Unit Substation Transformers and 480Y/277V Switchgear Lineup) and 480V Motor Control Centers (MCCs)

Double-Ended Unit Substation

4160V Primary Switch and 4160V to 480Y/277V Power Transformer

2.0.2.1 <u>Existing Double-Ended Unit Substation</u>

<u>Existing 4160V Primary Switch (Qty. 2)</u>

Age: Approximately over 40 years old
Make: Unknown
Model: Unknown
Size: 4160V, 1200A
Condition: Fair
Available Load Capacity for Future Load: Unknown without individual metering
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

<u>Existing 4160V to 480Y/277V Power Transformer</u>

Age: Approximately over 40 years old
Make: General Electric
Model: G-856712A
Size: 4160V DELTA Primary, 480Y/277V WYE Secondary
750kVA @ 150°C Rise SC
1000kVA @ 150°C Rise FA
Condition: Fair
Available Load Capacity for Future Load: Unknown without individual metering
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

2.0.2.2 **<u>Existing Double-Ended Unit Substation</u>** – 480Y/277V Switchgear Lineup

480Y/277V Switchgear Lineup

<u>Existing 480Y/277V Switchgear Lineup</u>

Age: Approximately over 40 years old
Make: General Electric
Model: AKD-5
Trip Unit: GE Microversa Plus trip unit
Size: 480Y/277V, 1600A, 3P/3W, 30kAIC
Condition: Fair Condition
Future Available Space Capacity: Yes
Available Load Capacity for Future Load: Unknown without individual metering
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

2.0.2.3 **<u>Existing Double-Ended Unit Substation</u>** – Summary

The existing double-ended unit substation appears to be in fair and working condition. It is estimated to be over 40 years old and is expected to be near the end of its useful life. The arrangement of the existing double-ended unit substation includes two 4160V primary switches that are fed from the main outdoor substation’s 4160V switchgear cubicles #6 and #7 via 5kV underground distribution feeders. Two 4160V to 480Y/277V unit substation transformers are closed-coupled to the 480Y/277V switchgear lineup. The switchgear lineup consists of a main-tie-main breaker and feeder drawout circuit breakers that provide 480/277V power to downstream distribution panels and MCC loads. The switchgear lineup currently has no breaker spares or spaces available for futureload.

2.0.2.4 **Existing 480V Motor Control Centers** – 480V MCC #1, #2, #3, #4, #6, #7

MCC #1

MCC #2

MCC #3

MCC #4

MCC #6

MCC #7

Existing 480V MCC #1, 480V MCC #3, 480V MCC #4, 480V MCC #6, 480V MCC #7

Age: Less than 5 years old
Make: General Electric
Model: Evolution Series 9000
Size: 480V, (MCC #1, #3, # 4, # 7 – 600A MCB), (MCC #6 - 800A MCB), 800A Bus, 3P/3W
Condition: Excellent
Future Available Space Capacity: Yes
Available Load Capacity for Future Load: Unknown without individual metering
Expected Remaining Life: 20 – 25 Years
Recommendations: Refer to Section 3.0

Existing 480V MCC #2 – Currently used for temporary power for field press

Make: Cutler Hammer
Model: Unitrol
Size: 480V, 600A MCB, 800A Bus, 3P/3W
Condition: Poor
Recommendations: Refer to Section 3.0

2.0.2.5 **Existing 480V Motor Control Centers** – Summary

The existing 480V Motor Control Centers (MCCs) are fed from the existing switchgear lineup in the existing double ended unit substation. With the exception of MCC #2 all the MCCs are in excellent working condition and less than 5 years old. MCC #2 is currently used to provide temporary power for a field press. MCC #2 was used for incinerator #2 that is currently not operational.

2.0.3 **Existing Maintenance Garage** – 2250kW, 4160V, Standby Diesel Engine Generator and 480Y/277V main circuit breaker panelboard

2250kW, 4160V, Standby Diesel Engine Generator

Existing 2250kW, 4160V, Standby Diesel Engine Generator
Age: Less than 5 years old
Make: Milton CAT
Model: 3516B
Size: 2250kW, 4160V, 3P/3W, 60HZ, 0.8pf
Condition: Excellent
Available Load Capacity for Future Load: None
Expected Remaining Life: 25 – 30 years
Recommendations: Refer to Section 3.0

2.0.3.1 **Existing Maintenance Garage** – Summary

The existing 480Y/277V main circuit breaker panelboard and the existing 2250kW, 4160V, standby diesel generator are less than 5 years old and located inside the existing maintenance garage. The main circuit breaker panelboard is fed from existing MCC #4 located in the administration building via 480V underground feed. The existing generator is connected to one of the existing outdoor main unit substation's 1200A drawout air circuit breakers via 4160V underground feed. During commercial power loss the generator provides power to the entire facility via the outdoor main unit substation metal-clad switchgear. The peak kW demand load recorded by the utility company over the last 2 years was 2637kW. Based on the peak kW demand load the generator is currently beyond its max capacity. During commercial power loss the generator log reports that the generator is running at 63% of its total load capacity. This is mostly in part because of load shedding and not operating all of the facilities large motors during commercial power loss. Specifically, under normal operation two 700HP blower motors operate. During loss of commercial power (Source #1 and Source #2) only one 700HP pump is operated while the entire facilities power is provided by the standby generator.

2.0.4 **Existing Blower Building –** 4160V Primary Selector Switches, Motor Control Center, Unit Substation (4160V Primary Switch, 4160V to 480Y/277V Power Transformer), and 480V Motor Control Center

Medium Voltage Motor Control Switchgear

2.0.4.1 Existing 4160V Primary Selector Switches (Qty. 2)

4160V Primary Selector Switches

Age: Approximately over 40 years old
Make: I-T-E
Model: Unknown
Size: 4160V, 1200A
Condition: Fair
Available Load Capacity for Future Load: Unknown without individual metering
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

2.0.4.2 Existing 4160V Motor Control Switchgear

4160V Drawout Fused Switches

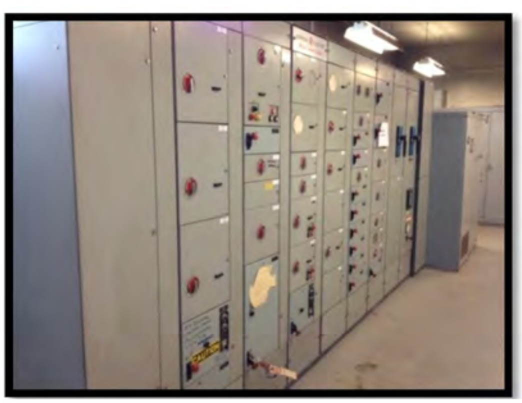
Existing 4160V Drawout Fused Switches

Age: Approximately over 40 years old
Make: General Electric
Model: Limitamp Control
Rating: 4160Y/2400V, 1200A Bus, 3P/3W
Fuse Sizes: 9R (200A) – Blower Motors
150E – MCC Unit Substation
Condition: Fair
Future Available Space Capacity: None
Available Load Capacity for Future Load: Unknown without individual metering
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

2.0.4.3 **Existing 4160V Motor Control Switchgear**– Summary

The existing medium voltage motor control switchgear appears to be in fair and working condition. It is estimated to be over 40 years old and is expected to be near the end of its useful life. The arrangement of the existing medium voltage motor control switchgear includes two 4160V primary selector switches, four 4160V drawout fused switches with motor control relays and one 4160V drawout fused switch. The four drawout fused switches with motor controls protect and control four 4160V, 700 – 800 HP blower motors. The blower motors have across the line start. The one drawout fused switch feeds an existing motor control center unit substation located in the same building. The two primary selector switches are fed from the 4160V main substation metal-clad switchgear cubicles #4 & #5 via 5KV underground distribution feeders. The 4160V motor control center currently has no additional spaces available to add future loads.

2.0.4.4 **Existing 480V Motor Control Center Unit Substation** – 4160V Primary Switch, 4160V to 480Y/277V Power Transformer and 480V Motor Control Center (MCC #10)



480V Motor Control Center Unit Substation

2.0.4.5 **Existing 4160V Primary Switch and 4160Y to 480Y/277V Unit Substation Transformer**

4160V Primary Switch and 4160 to 480Y/277V Unit Substation Transformer

Existing 4160V Primary Switch

Age: Approximately over 40 years old
Make: Unknown
Model: Unknown
Size: 4160V, 1200A
Condition: Fair
Available Load Capacity for Future Load: Unknown without individual metering
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

Existing Unit SubstationTransformer

Age: Approximately over 40 years old
Make: General Electric
Model: G-856711
Impedance (Z%): 6.01
Size: 4160V DELTA Primary, 480Y/277V WYE Secondary, class AA
750kVA @ 150°C Rise SC
1000kVA @ 150°C Rise FA
Condition: Fair
Available Load Capacity for Future Load: Unknown without individual metering
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

2.0.4.6 **<u>Existing 480Y/277V Motor Control Center</u>** - MCC #10

MCC #10

Age: Approximately over 40 years old
Make: General Electric
Model: 7700 Line
Size: 480Y/277V, 800A MCB, 1200A, 3P/3W
Condition: Fair
Future Available Space Capacity: Yes
Available Load Capacity for Future Load: Unknown without individual metering
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

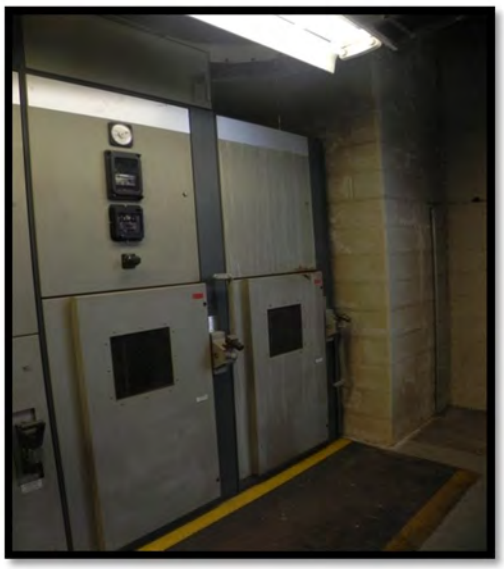
2.0.4.7 **<u>Existing 480V Motor Control Center Unit Substation</u>** - Summary

The existing 480V motor control center unit substation appears to be in fair and working condition. It is estimated to be over 40 years old and is expected to be near the end of its useful life. The existing motor control center unit substation is fed from the existing 4160V motor control center located in the same building.

2.0.5 **<u>Existing Raw Waste Pumping Station</u>** – 4160V Primary Selector Switches, 4160V Motor Control Center with Fused Switches, Exterior 4160V to 480V Pad Mounted Transformers, Interior 4160V to 480Y/277V Dry Type Transformer and 480V Motor Control Center (MCC #9)

Medium Voltage Motor Control Switchgear

2.0.5.1 **<u>Existing 4160V Primary Selector Switches (Qty. 2)</u>**



4160V Primary Selector Switches

Age: Approximately over 40 years
Make: S & C
Model: Unknown
Size: 4160V, 1200A
Condition: Fair, rust appears on equipment access door covers due to water leak from the floor above
Available Load Capacity for Future Load: Unknown without individual metering
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

2.0.5.2 Existing 4160V Drawout Fused Switches (Qty.4)

4160V Drawout Fused Switches

Age: Approximately over 40 years
Make: General Electric
Model: Limitamp Control
Rating : 4160Y/2400V, 1200A Bus, 3P/3W
Switch Sizes: Unknown – 4160V to 480V Power Transformers
100E – 4160V to 480Y/277V Interior Transformer
Condition: Fair
Available Load Capacity for Future Load: Unknown without individual metering
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

2.0.5.3 Existing 4160V Motor Control Center Switchgear – Summary

The existing medium voltage motor control center appears to be in fair and working condition. Some rust appears on equipment covers. It is estimated to be over 40 years old and is expected to be near the end of its useful life. The arrangement of the existing medium voltage motor control center includes two 4160V primary selector switches, four 4160V drawout fused switches with motor controls that have been modified to serve the existing outdoor pad mounted transformers and one 4160V drawout fused switch to serve an existing interior dry type transformer. The four existing outdoor pad mount transformers feed four existing large 460V, 300 HP raw waste pumps. The large raw waste pumps are controlled via variable speed drives. The one drawout fused switch feeds an existing interior-transformer. This transformer feeds MCC #9. Both the interior transformer and MCC #9 are located in the raw waste pumping station. The two primary selector switches are fed from the main substation 4160V metal-clad switchgear cubicles #8 and #9 via 5KV with two 4160V underground distribution underground feeders. The 4160V motor control center currently has no additional spaces available to add-future loads.

2.0.5.4 Existing Exterior Pad Mount 4160V to 480V Transformers

Exterior Pad Mount 4160V to 480V Power Transformers

Existing Exterior Pad Mount 4160V to 480V Power Transformers

Age: Approximately less than 5 years old
Make: Unknown
Model: Unknown
Size: 4160V DELTA Primary, 480V DELTA Secondary, 500Kva, class OA
Condition: Good
Expected Remaining Life: 20-30 years
Recommendations: Refer to Section 3.0

2.0.5.5 Existing Exterior Pad Mounted 4160V to 480V Transformers – Summary

The existing exterior pad mounted power transformers appear to be in good and working condition and are estimated to be less than 5 years old. They are fed via 4160V underground feeds from the existing 4160v motor control center switchgear located inside the raw waste pumping station. The four existing pad mount exterior transformers feed four existing large 480V, 300 HP raw waste pumps that are located inside the raw waste pumping station via the variable speed drives.

2.0.5.6 Existing Interior Dry Type 4160V to 480Y/277V Transformer

Interior 4160V to 480Y/277V Transformer

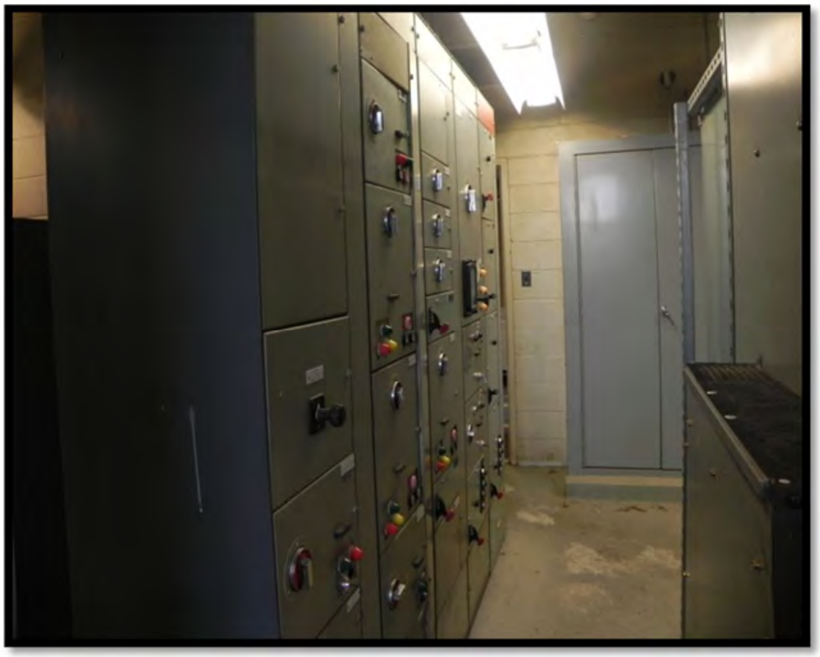
Existing Interior 4160V to 480Y/277V Transformer

Age: Approximately over 30 years old
Make: General Electric
Model: 9T25B5658 66
Size: 4160V DELTA Primary, 480Y/277V WYE Secondary, 500kVA
Condition: Fair
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

2.0.5.7 Existing Interior Dry Type 4160V to 480Y/277V Transformer – Summary

The existing interior power transformer appears to be in fair and working condition. It is estimated to be over 30 years old and is expected to be near the end of its useful life. It is fed from the existing 4160V motor control center switchgear located inside the raw waste pumping station.

2.0.5.8 **<u>Existing Motor Control Center</u>** – MCC #9



MCC #9

Age: Approximately over 40 years old
Make: General Electric
Model: 8000 Line
Size: 480V, 600A, 3P/3W
Condition: Fair
Future Available Space Capacity: Yes
Available Load Capacity for Future Load: Unknown without individual metering
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

2.0.5.9 **<u>Existing 480V Motor Control Center (MCC #9)</u>** – Summary

The existing motor control center (MCC #9) appears to be in fair and working condition. It is estimated to be over 40 years old and is expected to be near the end of its useful life. It is fed from the existing 500kVA dry type transformer located in the existing raw waste pumping station. Currently there are spare spaces in the MCC to add future loads.

2.0.6 **Existing Thickener Complex** – Motor Control Center (MCC #5)

MCC-5

Age: Approximately over 40 years old
Make: General Electric
Model: 7700 Line
MCC Rating: 480V, 500A MCB, 1200A Bus, 3P/3W
Transformer section Size: 480V DELTA Primary, 208Y/120V WYE Secondary, 30kVA, dry type
Condition: Fair
Future Available Space Capacity: Yes
Available Load Capacity for Future Load: Unknown without individual metering
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

2.0.6.1 **Existing Motor Control Center (MCC #5)** – Summary

The existing motor control center (MCC #5) appears to be in fair and working condition. It is estimated to be over 40 years old and is expected to be near the end of its useful life. It is fed via underground distribution feeders from the existing 480V main-tie-main double ended unit substation in the administration building. A transformer section rated for 30kVA and a circuit breaker distribution section are provided within the same MCC lineup to serve the building's 208Y/120V loads. Currently there are spare spaces in the MCC for future loads.

2.0.7 **<u>Existing Grit Building</u> –** Motor Control Center (MCC#8)

480V Motor Control Center #8

Age: Approximately over 40 years old
Make: General Electric
Model: 8000 Line
MCC Rating: 480V, 400A MCB, 600A Bus, 3P/3W
Transformer section Size: 480V DELTA Primary, 208Y/120V WYE Secondary, 45kVA, dry type
Condition: Fair
Future Available Space Capacity: Yes
Available Load Capacity for Future Load: Unknown without individual metering
Expected Remaining Life: 5 years
Recommendations: Refer to Section 3.0

2.0.7.1 **<u>Existing Motor Control Center (MCC #8)</u>** – Summary

The existing 480V motor control center (MCC #8) appears to be in fair and working condition. It is estimated to be over 40 years old and is expected to be near the end of its useful life. It is fed with a 480V underground feed from an existing interior transformer in the raw waste pumping station. MCC #8 feeds a 45kVa, 480V to 208Y/120V step down transformer located in the existing grit building to feed 208Y/120V loads. Currently there are spare spaces in the 480V MCC #8 for future loads.

2.0.8 **<u>Existing Chemical Storage Building</u> –** 480V Main Fused Disconnect Switch, 480V to 208Y/120V Transformer and 208Y/120V Main Circuit Breaker Panelboard.

2.0.8.1 **Existing Chemical Storage Building** – Summary

The existing 480V main fused disconnect appears to be in excellent and working condition. It is estimated to be approximately 5 years old. It is fed with a 480V underground feed from existing MCC #10 in the existing blower building. The main disconnect switch feeds a 45kVa, 480V to 208Y/120V step down transformer located in the chemical storage building to feed 208Y/120V loads.

SECTION 3.0

EXISTING ELECTRICAL EQUIPMENT RECOMMENDATIONS

3.0 EXISTING ELECTRICAL EQUIPMENT RECOMMENDATIONS

NOTE: Refer to Opinion of Probable Construction Cost in Section 5.0 for equipment recommendations.

3.0.1 **Existing Outdoor Main Unit Substation** **-** Power Transformers, 46kV Group Operated Air Break Switches, Vertical Mount Disconnectable Power Fuses, Lightning Arrestor and 4160Y/2400V-Metal-Clad Switchgear Lineup

1. Existing outdoor main unit substation recommendations are listed in the four proposed options in section 4.0.

3.0.2 **Existing Administration Building** **–** Existing Double Ended Unit Substation (4160V Voltage Primary Switches, 4160V to 480Y/277V Unit Substation Transformers and 480Y/277V Switchgear Lineup) and 480V Motor Control Centers (MCCs)

1. Remove and replace the existing 4160V primary switches with metal enclosed switchgear that includes fused switches suitable for indoor installation.
2. Remove existing unit substation's 750KVA transformers and replace with dry type transformers that are close-coupled to the replacement primary switches and replacement main-tie-main switchgear.
3. Provide 480/277V, 1600Amp, 3 phase, 4 wire switchgear with main-tie-main, and feeder power circuit breakers, drawout type. Replacement equipment shall be provided with solid state overcurrent protection devices and power metering.
4. Remove and replace existing 4160V underground feeders from existing outdoor main unit substation to replacement main-tie-main double ended unit substation in existing administration building.
5. The existing 480V Motor Control Centers #1, #3, #4, #6 and #7 are in excellent condition and shall remain.
6. Disconnect and remove existing 480V Motor Control Center #2.
7. Remove and replace existing 480V underground feed from replacement main-tie-main double ended unit substation to replacement MCC #5 in existing thickener complex.

3.0.3 **Existing Maintenance Garage** **–** Existing 2250kW, 4160V, Standby Diesel Engine Generator and 480Y/277V main circuit breaker panelboard

1. Existing 2250kW generator to remain. Refer to the four proposed options in section 4.0 for supplemental emergency standby power options.
2. Replace existing 4160V underground feeder from the generator connection enclosure to the existing or replacement main unit substation switchgear to handle the additional generator capacity.

3. 480Y/277V main circuit breaker panelboard is in good condition and shall remain.

3.0.4 **Existing Blower Building –** 4160V Primary Selector Switches, Motor Control Center, Unit Substation (4160V Primary Switch, 4160V to 480Y/277V Power Transformer), and 480V Motor Control Center

1. Remove existing 4160V primary selector switches and metal enclosed switchgear. Provide replacement 4160V primary switches and switchgear that includes fused switch sections that serve the existing blowers and replacement transformer. The replacement switchgear shall be located inside the existing blower building.
2. Remove existing interior 750kVA transformer and 4160V primary switch and replace with interior 4160V-480Y/277V, 750kVA dry type transformer and 4160V primary switch to serve replaced MCC #10.
3. Remove existing MCC #10 and replace with 9-section, 480V, 1200A frame, 800A MCB, 3P/3W motor control center.
4. Existing feeders from existing outdoor unit substation to existing blower building medium voltage switchgear were replaced within the last 10 years. Test and inspect existing feeders; replace if necessary.
5. Remove and replace existing 480V underground feed from MCC #10 in existing blower building to existing main fused disconnect in existing chemical storage building.
6. Provide VSDs with soft start for existing 4160V, 700 HP – 800 HP blower motors. VSDs shall also be capable of across the line start. VSDs to be located in existing blower building on north wall adjacent to blower motors.

3.0.5 **Existing Raw Waste Pumping Station -** 4160V Primary Selector Switches, 4160V Motor Control Center with Fused Switches, Exterior 4160V to 480V Pad Mounted Transformers, Interior 4160V to 480Y/277V Dry Type Transformer and 480V Motor Control Center (MCC #9)

1. Remove existing 4160V primary selector switches and metal enclosed switchgear. Provide replacement 4160V primary switches and switchgear that includes fused switch sections that serve the existing existing outdoor pad mount transformers and replacement interior 500KVA dry type transformer. The replacement switchgear shall be located inside the existing raw waste pumping station. Some of the features may include incoming live line and incoming line de-energized indications.
2. Maintain the existing primary and secondary feeders to the existing outdoor pad mounted transformers; feeders are less than 5 years old. Re-route and extend existing primary feeders as required to replacement 4160V switchgear location inside the building.

3. Disconnect and remove existing interior 4160V to 480V transformer feeding existing MCC #9 and MCC #8 (via existing fused disconnect switch) inside existing grit building. Existing feeders for both MCCs shall be re-routed/extended to replacement switchboard.
4. Provide replacement interior 500KVA, 4160V-480Y/277V dry type transformer to serve new switchboard.
5. Provide 480Y/277V switchboard served from replacement interior 500KVA transformer inside the existing raw waste pumping station to serve MCC#8 and MCC#9.
5. Remove existing MCC #9 and replace with 5-section, 480V, 1200A frame, 800A MCB, 3P/3W motor control center.
6. Remove existing fused disconnect switch that serves MCC#8 in existing grit building. MCC #8 will be fed from replacement switchboard in existing raw waste pumping station.
7. Remove and replace existing 4160V underground feeders from existing outdoor main unit substation to existing medium voltage motor control center in existing raw waste pumping station.
8. Remove and replace existing 480V underground feed from existing 4160V to 480V interior transformer in existing raw waste pumping station to existing MCC #8 in existing grit building.

3.0.6 **Existing Thickener Complex –** Motor Control Center (MCC #5)

1. Remove existing 480V motor control center (MCC #5)
2. Replace with 480V, 600A main bus with line-up sections that to include the following:
   a. 5-section, 480V, 600A frame, 500A MCB, 3P/3W motor control center.
   b. 30KVa transformer to serve the 208/120V downstream load.
   c. FVNR with solid state motor starters, control power transformers, indication lights, and H-O-A selector switches.
   d. Feeder circuit breakers.

3.0.7 **Existing Grit Building –** Motor Control Center #8 (MCC #8)

1. Remove existing 480V motor control center (MCC #8).
2. Replace with 480V, 600A main bus with line-up sections that to include the following:
   a. 5-section, 480V, 600A frame, 400A MCB, 3P/3W motor control center.
   b. 45kVA transformer to serve the 208/120V downstream loads.
   c. FVNR with solid state motor starters, control power transformers, indication lights, and H-O-A selector switches.
   d. Feeder circuit breakers.

3.0.8 **<u>Existing Chemical Storage Building</u> –** Existing 480V Main Fused Disconnect Switch, 480V to 208Y/120V Power transformer and 208Y/120V Main Circuit Breaker Panelboard.

1. Existing equipment is in excellent condition and will remain.

SECTION 4.0

PROPOSED ELECTRICAL EXPANSION OPTIONS BASED ON SPLIT FLOW DESIGN CONCEPT AND EXISTING CONDITIONS

4.0 PROPOSED ELECTRICAL EXPANSION OPTIONS BASED ON SPLIT FLOW DESIGN CONCEPT AND EXISTING CONDITIONS

NOTES:

1. **Refer to Opinion of Probable Construction Cost in Section 5.0 for each of the four proposed options.**
2. **Refer to One-Line Diagrams in Section 6.0 for each of the four proposed options.**

4.0.1 Option #1:

1. Maintain existing main substation utility receiving structure, power transformers, and the 4160V metal-clad switchgear.
2. Provide 4160V, 2.25MW diesel engine generator to match existing. Provide pad mounted generator paralleling switchgear with draw out vacuum circuit breakers and all required protection relays and controls to synchronize both generators to the main bus. Engine generator and paralleling gear shall be installed with a weather protected and sound attenuated enclosure. Location of paralleling gear and generator will be outside on the south side of existing maintenance garage.
3. Provide outdoor pad mounted 2000kW resistive load bank for generator exercising. Load bank will be connected to generator paralleling gear. Location of load bank will be outside on south side of existing maintenance garage.
4. Upgrade existing 4160V generator feeder from the existing outdoor main substation and connect to generator paralleling switchgear vacuum breaker. Reconfigure all control wiring between the existing outdoor main substation switchgear and generator paralleling gear.
5. Upgrade existing 4160V feeders to the existing raw waste pumping station.
6. Provide outdoor 4160V pad mounted primary switches to serve the new pump station. Location will be outside, adjacent to and on the south side of the new pump station.
7. Provide 1000KVA, 4160-480Y/277 outdoor pad mounted transformer to serve the new pump station. Location will be outside, adjacent to and on the south side of the new pump station.
8. Provide underground feeder splices for the replacement 4160V raw waste pumping station feeders. Provide 4160V feeders to outdoor pad mounted primary switches to serve the new pump station.

4.0.2 **Option #2:**

1. Maintain existing main substation utility receiving structure, power transformers, and the 4160V metal-clad switchgear.
2. Provide 4160V, 2.25MW diesel engine generator to match existing. Provide pad mounted generator paralleling switchgear with draw out vacuum circuit breakers and all required protection relays and controls to synchronize both generators to the main bus. Engine generator and paralleling gear shall be installed with a weather protected and sound attenuated enclosure. Location of pad mounted paralleling gear and generator will be outside, adjacent to and on the south side of existing maintenance garage.
3. Provide outdoor pad mounted 2000kW resistive load bank for generator exercising. Load bank will be connected to generator paralleling gear. Location of load bank will be outside on south side of existing maintenance garage.
4. Upgrade existing 4160V generator feeder from the existing outdoor main substation and connect to the generator paralleling switchgear vacuum breaker. Reconfigure all control wiring between the existing outdoor main substation switchgear and generator paralleling gear.
5. Remove one of the existing 4160V feeders that feed the existing raw waste pumping station and one of the existing feeders that feed the existing blower building. Two existing air circuit breakers in the existing outdoor main substation switchgear will be made available.
6. One of the 4160V air circuit breakers in the existing outdoor main substation switchgear shall be utilized to serve one of the new pump station’s outdoor pad mounted primary switches and transformer. The second breaker in the existing outdoor unit substation shall labeled as spare.
7. Provide outdoor 4160V pad mounted primary switches to serve the new pump station. Location will be outside, adjacent to and on the south side of the new pump station.
8. Provide 1000kVA, 4160-480Y/277 outdoor pad mounted transformer to serve the new pump station. Location will be outside, adjacent to and on the south side of the new pump station.
9. Provide emergency 4160V feeders from the pad mounted generator paralleling switchgear to the existing raw waste pumping station, existing blower building and one of the new pump station outdoor pad mounted switches.
10. Provide 4160V feeder from the existing outdoor substation switchgear to one of the new pump station’s outdoor pad mounted switches.

4.0.3 **Option #3:**

1. Maintain the existing main substation utility receiving structure and power transformers.
2. Remove existing 4160V metal-clad switchgear and replace with 4160V, main-tie-main, metal-clad switchgear. Replacement metal-clad switchgear will have weather proof enclosure, 1200Amps draw-out vacuum circuit breakers, all required protective relays, current transformers, potential transformers, utility metering compartments and auxiliary devices. The replacement switchgear will be connected to existing transformers via busway. Replacement gear will be located outside in existing substation yard.
3. Provide outdoor pad mounted 2000kW resistive load bank for generator exercising. Load bank will be connected to generator paralleling gear. Location of load bank will be outside on south side of existing maintenance garage.
4. The two existing outdoor power transformers shall share the facility load via two bus sections with an open tie breaker within the replacement 4160V metal-clad switchgear. If one of the existing power transformers fails, or fails to de-energize one of the existing transformers for maintenance, the second existing transformer shall serve the entire facility load through the closed bus tie breaker.
5. Provide 4160V, 2.25MW diesel engine generator to match existing. Provide pad mounted generator paralleling switchgear with draw out vacuum circuit breakers and all required protection relays and controls to synchronize both generators to the main bus. Engine generator and paralleling gear shall be installed with a weather protected and sound attenuated enclosure. Location of pad mounted paralleling gear and generator will be outside, adjacent to, and on the south side of existing maintenance garage.
6. Provide 4160V emergency feeders from the existing outdoor main substation 4160V switchgear and connect to the outdoor pad mounted generator paralleling switchgear's vacuum breakers. Provide all required control wiring between the switchgears.
7. Replace existing 4160V feeders to the replacement outdoor substation metal-clad switchgear in the substation yard.
8. Provide 4160V feeders from the replacement outdoor substation switchgear to new pump station's pad mounted switches.
9. Provide 4160V outdoor pad mounted primary switches to serve the new pump station. Location will be outside, adjacent to and on the south side of the new pump station.
10. Provide 1000kVA, 4160-480Y/277 outdoor pad mounted transformer to serve the new pump station. Location will be outside, adjacent to and on the south side of the new pump station.

4.0.4 Option #4 (Recommended):

1. Remove existing utility receiving structures and replace with utility receiving structures with 46kV, group operated air break switches. Located in existing substation yard.
2. Provide circuit switchers mounted on a concrete pad with equipment mounted control panel. Located outside in existing substation yard.
3. Remove existing outdoor power transformers and replace with two liquid filled power transformers rated for 46kV primary-4160Y/2400V secondary, 7.5MVA/9.3MVA @65 degree temperature rise with forced air fans and automatic load tap changers. Located outside in existing substation yard next to replacement metal-clad switchgear.
4. Remove existing 4160V metal-clad switchgear and replace with 4160V, main-tie-main, metal-clad switchgear. Replacement metal-clad switchgear will have weather proof enclosure with 2000Amps draw-out vacuum circuit breakers (Main, generator and tie breakers) and 1200A feeder distribution vacuum circuit breakers. Provide all required protective relays, current transformers, potential transformers, utility metering compartments and auxiliary devices. Located outside in existing substation yard.
5. Provide 4160V, 2.25MW diesel engine generator to match existing. Provide pad mounted generator paralleling switchgear with draw out vacuum circuit breakers and all required protection relays and controls to synchronize both generators to the main bus. Engine generator and paralleling gear shall be installed with a weather protected and sound attenuated enclosure. Location of pad mounted paralleling gear and generator will be outside, adjacent to and on the south side of existing maintenance garage.
6. Provide outdoor pad mounted 2000kW resistive load bank for generator exercising. Load bank will be connected to generator paralleling gear. Location of load bank will be outside on south side of existing maintenance garage.
7. Provide 4160V emergency feeders from the main substation 4160V switchgear and connect to outdoor generator paralleling switchgear vacuum breakers. Provide all required control wiring between the switchgears.
8. Replace existing 4160V feeders to replacement substation metal-clad switchgear location.
9. Provide 4160V feeders from the replacement substation switchgear to the new pump station's outdoor pad mounted switches.
10. Provide 4160V pad mounted primary switches to serve the new pump station. Location will be outside, adjacent to and on the south side of the new pump station.
11. Provide 1000kVA, 4160-480Y/277 pad mounted transformer to serve the new pump station. Location will be outside, adjacent to and on the south side of the new pump station.

SECTION 5.0

OPINION OF PROBABLE CONSTRUCTION COST

PROJECT: Oneida County WWTP

Existing Administration Building - RECOMMENDED

COST ESTIMATE

Item	Description	Qty	Unit	Unit Cost	Cost	Total Cost
1	Removal work (double ended sub st., swgr and MCC#2)	1	ls	$40,000.00	$40,000.00	$40,000.00
2	750kva dry type indoor transformer	2	ea	$90,000.00	$180,000.00	$180,000.00
3	4.16KV indoor fused switches	2	ea	$35,000.00	$70,000.00	$70,000.00
4	1600A, 480V, main-tie-main swgr w/ drawout breakers	1	ls	$400,000.00	$400,000.00	$400,000.00
5	5KV Feeder termination	1	ls	$10,000.00	$10,000.00	$10,000.00
6	Grounding	1	ls	$10,000.00	$10,000.00	$10,000.00
7	Provide equipment pads	1	ls	$8,000.00	$8,000.00	$8,000.00
8	Miscellaneous wiring	1	ls	$5,000.00	$5,000.00	$5,000.00
9	Replace existing 5KV feeders	1	ls	$125,000.00	$125,000.00	$125,000.00
10	Replace existing 480V feeder to MCC#5	1	ls	$80,000.00	$80,000.00	$80,000.00
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	SUBTOTAL				$928,000.00	$928,000.00
	CONTINGENCY	10%				$92,800.00
	OVERHEAD AND PROFIT	15%				$139,200.00
	TOTAL					**$1,160,000.00**

Trade:	**1 of 1**
Phase: Conceptual	

PROJECT: Oneida County WWTP

Existing Blower Building - RECOMMENDED

			COST ESTIMATE			
Item	**Description**	**Qty**	**Unit**	**Unit Cost**	**Cost**	**Total Cost**
1	Removal work	1	ls	$45,000.00	$45,000.00	$45,000.00
2	750kva dry type indoor transformer	1	ea	$90,000.00	$90,000.00	$90,000.00
3	4.16KV indoor fused switches	1	ls	$250,000.00	$250,000.00	$250,000.00
4	4.16Kv blower speed drives/soft start	4	ea	$150,000.00	$600,000.00	$600,000.00
5	Test existing 5KV feeders	1	ls	$10,000.00	$10,000.00	$10,000.00
6	5KV Feeder termination	1	ls	$10,000.00	$10,000.00	$10,000.00
7	Grounding	1	ls	$6,000.00	$6,000.00	$6,000.00
8	Provide equipment pads	1	ls	$8,000.00	$8,000.00	$8,000.00
9	Miscellaneous wiring	1	ls	$50,000.00	$50,000.00	$50,000.00
10	Test existing 480V feeders to chem. storage bldg	1	ls	$2,000.00	$2,000.00	$2,000.00
11	Replace existing 480V feeder	1	ls	$25,000.00	$25,000.00	$25,000.00
12	Replace existing MCC#10	1	ls	$100,000.00	$100,000.00	$100,000.00
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	SUBTOTAL				$1,196,000.00	$1,196,000.00
	CONTINGENCY	10%				$119,600.00
	OVERHEAD AND PROFIT	15%				$179,400.00
	TOTAL					**$1,495,000.00**

	Trade:	**1 of 1**
	Phase: Conceptual	

PROJECT: Oneida County WWTP

Existing Raw Waste Pump Station - RECOMMENDED

COST ESTIMATE

Item	Description	Qty	Unit	Unit Cost	Cost	Total Cost
1	Removal work	1	ls	$45,000.00	$45,000.00	$45,000.00
2	500kva dry type indoor transformer	1	ea	$55,000.00	$55,000.00	$55,000.00
3	4.16KV indoor fused switches	1	ls	$250,000.00	$250,000.00	$250,000.00
4	Test existing 5KV feeders	1	ls	$10,000.00	$10,000.00	$10,000.00
5	Extend existing 5KV feeders & terminations	1	ls	$35,000.00	$35,000.00	$35,000.00
6	Grounding	1	ls	$6,000.00	$6,000.00	$6,000.00
7	Provide equipment pads	1	ls	$8,000.00	$8,000.00	$8,000.00
8	Miscellaneous wiring	1	ls	$50,000.00	$50,000.00	$50,000.00
9	Test existing 480V feeders	1	ls	$3,000.00	$3,000.00	$3,000.00
10	Replace existing 5KV feeders to main substation	1	ls	$150,000.00	$150,000.00	$150,000.00
11	Replace existing 480V feeder to MCC#8	1	ls	$80,000.00	$80,000.00	$80,000.00
12	480/277V Switchboard	1	ls	$30,000.00	$30,000.00	$30,000.00
13	Replace existing MCC#9	1	ls	$70,000.00	$70,000.00	$70,000.00
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	SUBTOTAL				$792,000.00	$792,000.00
	CONTINGENCY	10%				$79,200.00
	OVERHEAD AND PROFIT	15%				$118,800.00
	TOTAL					**$990,000.00**

Trade:	**1 of 1**
Phase: Conceptual	

PROJECT: Oneida County WWTP

Existing Thickener Complex - RECOMMENDED

COST ESTIMATE

Item	Description	Qty	Unit	Unit Cost	Cost	Total Cost
1	Removal work	1	ls	$10,000.00	$10,000.00	$10,000.00
2	Replace existing MCC#5	1	ls	$75,000.00	$75,000.00	$75,000.00
3	Provide equipment pads	1	ls	$8,000.00	$8,000.00	$8,000.00
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	SUBTOTAL				$93,000.00	$93,000.00
	CONTINGENCY	10%				$9,300.00
	OVERHEAD AND PROFIT	15%				$13,950.00
	TOTAL					**$116,250.00**

Trade:	**1 of 1**
Phase: Conceptual	

PROJECT: Oneida County WWTP

Existing Grit Building - RECOMMENDED

COST ESTIMATE						
Item	**Description**	**Qty**	**Unit**	**Unit Cost**	**Cost**	**Total Cost**
1	Removal work	1	ls	$10,000.00	$10,000.00	$10,000.00
2	Replace existing MCC#8	1	ls	$75,000.00	$75,000.00	$75,000.00
3	Provide equipment pads	1	ls	$8,000.00	$8,000.00	$8,000.00
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	SUBTOTAL				$93,000.00	$93,000.00
	CONTINGENCY	10%				$9,300.00
	OVERHEAD AND PROFIT	15%				$13,950.00
	TOTAL					**$116,250.00**

Trade:	**1 of 1**
Phase: Conceptual	

PROJECT: Oneida County WWTP

46KV Substation, Generator and New Pump Station (Option-4) - RECOMMENDED

COST ESTIMATE

Item	Description	Qty	Unit	Unit Cost	Cost	Total Cost
1	72.5KV Circuit switcher	2	ea	$80,000.00	$160,000.00	$160,000.00
2	5KV Outdoor Switchgear lineup	1	ls	$1,150,000.00	$1,150,000.00	$1,150,000.00
3	7.5MVA Substation Power Transformers	2	ea	$400,000.00	$800,000.00	$800,000.00
4	Grounding & Lightning protection	1	ls	$100,000.00	$100,000.00	$100,000.00
5	Fencing	1	ls	$100,000.00	$100,000.00	$100,000.00
6	46KV OH conductors & surge arrestors	2	ea	$40,000.00	$80,000.00	$80,000.00
7	5KV Transformer secondary Bus	2	ea	$100,000.00	$200,000.00	$200,000.00
8	Power Manholes	5	ea	$8,000.00	$40,000.00	$40,000.00
9	46KV group operated switch	2	ea	$50,000.00	$100,000.00	$100,000.00
10	PT's and CT's	2	ea	$15,000.00	$30,000.00	$30,000.00
11	Miscellaneous power & control wiring	1	ls	$350,000.00	$350,000.00	$350,000.00
12	Equipment pads	1	ls	$60,000.00	$60,000.00	$60,000.00
13	General Site civil work	1	ls	$300,000.00	$300,000.00	$300,000.00
14	Removal work	1	ls	$250,000.00	$250,000.00	$250,000.00
15	Testing	1	ls	$40,000.00	$40,000.00	$40,000.00
16	Sump pumps	1	ls	$10,000.00	$10,000.00	$10,000.00
17	Pole lights	1	ls	$80,000.00	$80,000.00	$80,000.00
18	Extend existing feeders	1	ls	$150,000.00	$150,000.00	$150,000.00
19	Steel structures	2	ea	$150,000.00	$300,000.00	$300,000.00
20	Transformer neutral grounding	2	ea	$25,000.00	$50,000.00	$50,000.00
21	5KV Feeder termination	1	ls	$40,000.00	$40,000.00	$40,000.00
22	National Grid Service allowance	1	ls	$100,000.00	$100,000.00	$100,000.00
23	New 4.16kv, 2.25MW diesel generator	1	ls	$700,000.00	$700,000.00	$700,000.00
24	New 4.16kv generator parallel switchgear	1	ls	$450,000.00	$450,000.00	$450,000.00
25	New 4.16kv feeders from parallel gear to subst.	1	ls	$80,000.00	$80,000.00	$80,000.00
26	New 4.16 feeders to pump st.	1	ls	$150,000.00	$150,000.00	$150,000.00
27	1000kva pad mount transformer	1	ea	$80,000.00	$80,000.00	$80,000.00
28	4.16kv pad mount fused switches	1	ea	$60,000.00	$60,000.00	$60,000.00
29	Secondary feeders to pump station	1	ls	$40,000.00	$40,000.00	$40,000.00
30	New MCC pump station w/VFD's & FVNR starters	1	ea	$150,000.00	$150,000.00	$150,000.00
31	Miscellaneous pump st. bldg devices and lighting	1	ls	$20,000.00	$20,000.00	$20,000.00
32	New switchboard in pump station	1	ea	$80,000.00	$80,000.00	$80,000.00
33	Low voltage panels & transformers	1	ls	$30,000.00	$30,000.00	$30,000.00
	SUBTOTAL				$6,330,000.00	$6,330,000.00
	CONTINGENCY	10%				$633,000.00
	OVERHEAD AND PROFIT	15%				$949,500.00
	TOTAL					**$7,912,500.00**

Trade:	**1 of 1**
Phase: Conceptual	

PROJECT: Oneida County WWTP

46KV Substation, Generator and New Pump Station (Option-3)

COST ESTIMATE

Item	Description	Qty	Unit	Unit Cost	Cost	Total Cost
1	5KV Outdoor Switchgear lineup	1	ls	$1,150,000.00	$1,150,000.00	$1,150,000.00
2	Grounding	1	ls	$30,000.00	$30,000.00	$30,000.00
3	Fencing	1	ls	$25,000.00	$25,000.00	$25,000.00
4	5KV Transformer secondary Bus	2	ea	$80,000.00	$160,000.00	$160,000.00
5	Power Manholes	5	ea	$8,000.00	$40,000.00	$40,000.00
6	Miscellaneous power & control wiring	1	ls	$250,000.00	$250,000.00	$250,000.00
7	Equipment pads	1	ls	$40,000.00	$40,000.00	$40,000.00
8	General Site civil work	1	ls	$80,000.00	$80,000.00	$80,000.00
9	Removal work	1	ls	$100,000.00	$100,000.00	$100,000.00
10	Testing	1	ls	$25,000.00	$25,000.00	$25,000.00
11	Sump pumps	1	ls	$10,000.00	$10,000.00	$10,000.00
12	Pole lights	1	ls	$30,000.00	$30,000.00	$30,000.00
13	Extend existing feeders	1	ls	$150,000.00	$150,000.00	$150,000.00
14	5KV Feeder termination	1	ls	$40,000.00	$40,000.00	$40,000.00
15	National Grid Service allowance	1	ls	$50,000.00	$50,000.00	$50,000.00
16	New 4.16kv, 2.25MW diesel generator	1	ls	$700,000.00	$700,000.00	$700,000.00
17	New 4.16kv generator parallel switchgear	1	ls	$450,000.00	$450,000.00	$450,000.00
18	New 4.16kv feeders from parallel gear to subst.	1	ls	$80,000.00	$80,000.00	$80,000.00
19	New 4.16 feeders to pump st.	1	ls	$150,000.00	$150,000.00	$150,000.00
20	1000kva pad mount transformer	1	ea	$80,000.00	$80,000.00	$80,000.00
21	4.16kv pad mount fused switches	1	ea	$60,000.00	$60,000.00	$60,000.00
22	Secondary feeders to pump station	1	ls	$40,000.00	$40,000.00	$40,000.00
23	New MCC pump station w/VFD's & FVNR starters	1	ea	$150,000.00	$150,000.00	$150,000.00
24	New switchboard in pump station	1	ea	$80,000.00	$80,000.00	$80,000.00
25	Low voltage panels & transformers	1	ls	$30,000.00	$30,000.00	$30,000.00
26	Miscellaneous pump st. bldg devices and lighting	1	ls	$20,000.00	$20,000.00	$20,000.00
27						
28						
29						
30						
	SUBTOTAL				$4,020,000.00	$4,020,000.00
	CONTINGENCY	10%				$402,000.00
	OVERHEAD AND PROFIT	15%				$603,000.00
	TOTAL					**$5,025,000.00**

Trade:	**1 of 1**
Phase: Conceptual	

PROJECT: Oneida County WWTP

46KV Substation, Generator and New Pump Station (Option-2)

COST ESTIMATE

Item	Description	Qty	Unit	Unit Cost	Cost	Total Cost
1	New 4.16kv, 2.25MW diesel generator	1	ls	$700,000.00	$700,000.00	$700,000.00
2	New 4.16kv generator parallel switchgear	1	ls	$600,000.00	$600,000.00	$600,000.00
3	New 4.16kv feeders from parallel gear to sub. st./site	1	ls	$250,000.00	$250,000.00	$250,000.00
4	New 4.16 feeders to pump station	1	ls	$150,000.00	$150,000.00	$150,000.00
5	Power Manholes	4	ea	$6,000.00	$24,000.00	$24,000.00
6	Miscellaneous power & control wiring	1	ls	$200,000.00	$200,000.00	$200,000.00
7	Equipment pads	1	ls	$35,000.00	$35,000.00	$35,000.00
8	General Site civil work	1	ls	$80,000.00	$80,000.00	$80,000.00
9	Grounding	1	ls	$10,000.00	$10,000.00	$10,000.00
10	Testing	1	ls	$20,000.00	$20,000.00	$20,000.00
11	Sump pumps	1	ls	$10,000.00	$10,000.00	$10,000.00
12	1000kva pad mount transformer	1	ea	$80,000.00	$80,000.00	$80,000.00
13	4.16kv pad mount fused switches	1	ea	$60,000.00	$60,000.00	$60,000.00
14	5KV Feeder termination	1	ls	$40,000.00	$40,000.00	$40,000.00
15	Removal work	1	ls	$20,000.00	$20,000.00	$20,000.00
16	Secondary feeders to pump station	1	ls	$40,000.00	$40,000.00	$40,000.00
17	New MCC pump station w/VFD's & FVNR starters	1	ea	$150,000.00	$150,000.00	$150,000.00
18	New switchboard in pump station	1	ea	$80,000.00	$80,000.00	$80,000.00
19	Low voltage panels & transformers	1	ls	$30,000.00	$30,000.00	$30,000.00
20	Miscellaneous pump st. bldg devices and lighting	1	ls	$20,000.00	$20,000.00	$20,000.00
21						
22						
23						
24						
25						
26						
27						
28						
29						
30						
	SUBTOTAL				$2,599,000.00	$2,599,000.00
	CONTINGENCY	10%				$259,900.00
	OVERHEAD AND PROFIT	15%				$389,850.00
	TOTAL					**$3,248,750.00**

Trade:	**1 of 1**
Phase: Conceptual	

PROJECT: Oneida County WWTP

46KV Substation, Generator and New Pump Station (Option-1)

COST ESTIMATE						
Item	**Description**	**Qty**	**Unit**	**Unit Cost**	**Cost**	**Total Cost**
1	New 4.16kv, 2.25MW diesel generator	1	ls	$700,000.00	$700,000.00	$700,000.00
2	New 4.16kv generator parallel switchgear	1	ls	$350,000.00	$350,000.00	$350,000.00
3	New 4.16kv feeders from parallel gear to sub. st.	1	ls	$50,000.00	$50,000.00	$50,000.00
4	New 4.16 feeders to new pump station	1	ls	$150,000.00	$150,000.00	$150,000.00
5	Power Manholes	4	ea	$6,000.00	$24,000.00	$24,000.00
6	Miscellaneous power & control wiring	1	ls	$200,000.00	$200,000.00	$200,000.00
7	Equipment pads	1	ls	$25,000.00	$25,000.00	$25,000.00
8	General Site civil work	1	ls	$80,000.00	$80,000.00	$80,000.00
9	Grounding	1	ls	$10,000.00	$10,000.00	$10,000.00
10	Testing	1	ls	$20,000.00	$20,000.00	$20,000.00
11	Sump pumps	1	ls	$10,000.00	$10,000.00	$10,000.00
12	1000kva pad mount transformer	1	ea	$80,000.00	$80,000.00	$80,000.00
13	4.16kv pad mount fused switches	1	ea	$60,000.00	$60,000.00	$60,000.00
14	5KV Feeder termination	1	ls	$25,000.00	$25,000.00	$25,000.00
15	Removal work	1	ls	$20,000.00	$20,000.00	$20,000.00
16	Secondary feeders to pump station	1	ls	$40,000.00	$40,000.00	$40,000.00
17	New MCC pump station w/VFD's & FVNR starters	1	ea	$150,000.00	$150,000.00	$150,000.00
18	New switchboard in pump station	1	ea	$80,000.00	$80,000.00	$80,000.00
19	Low voltage panels & transformers	1	ls	$30,000.00	$30,000.00	$30,000.00
20	Miscellaneous pump st. bldg devices and lighting	1	ls	$20,000.00	$20,000.00	$20,000.00
21						
22						
23						
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27						
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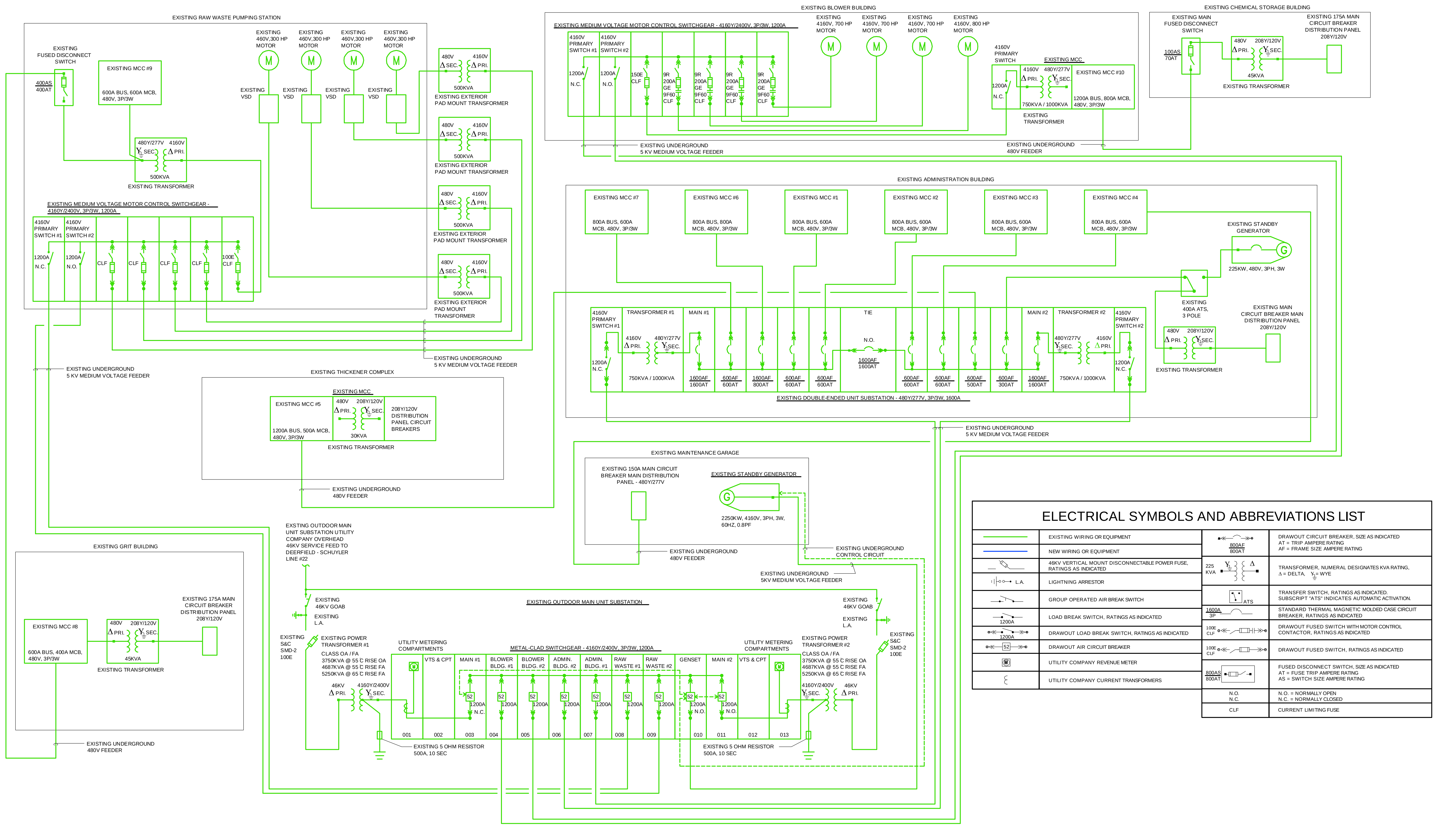
29						
30						
	SUBTOTAL				$2,124,000.00	$2,124,000.00
	CONTINGENCY	10%				$212,400.00
	OVERHEAD AND PROFIT	15%				$318,600.00
	TOTAL					**$2,655,000.00**

Trade:	**1 of 1**
Phase: Conceptual	

SECTION 6.0

ONE-LINE DIAGRAMS



DRAWING NOT TO SCALE

SHUMAKER Consulting Engineering & Land Surveying, P.C.

Brown AND Caldwell

GHD

WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

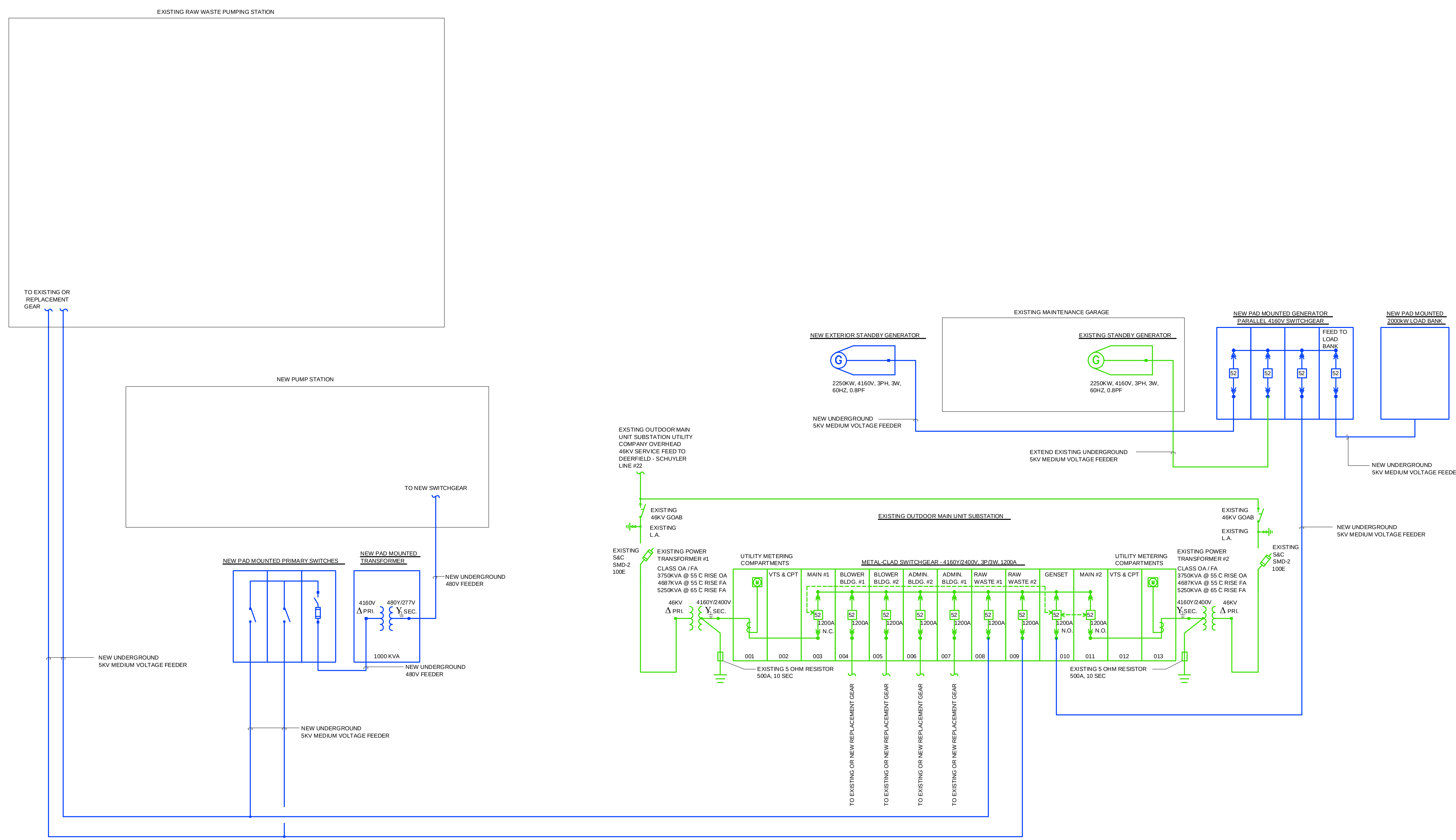
EXISTING ELECTRICAL ONE-LINE DIAGRAM

Job Number	8614782
Revision	A
Date	AUGUST 2012

Figure 01

200 John James Audubon Parkway Suite 101, Amherst NY 14228 USA T 1 716 691 8503 F 1 716 691 8506 E amhmail@ghd.com W www.ghd.com

Plot Date: 30 August 2012 - 3:24 PM Cad File No: G:\86\14782\CADD\Drawings\Electrical\Electrical One-Line Diagram.dwg



DRAWING NOT
TO SCALE

WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

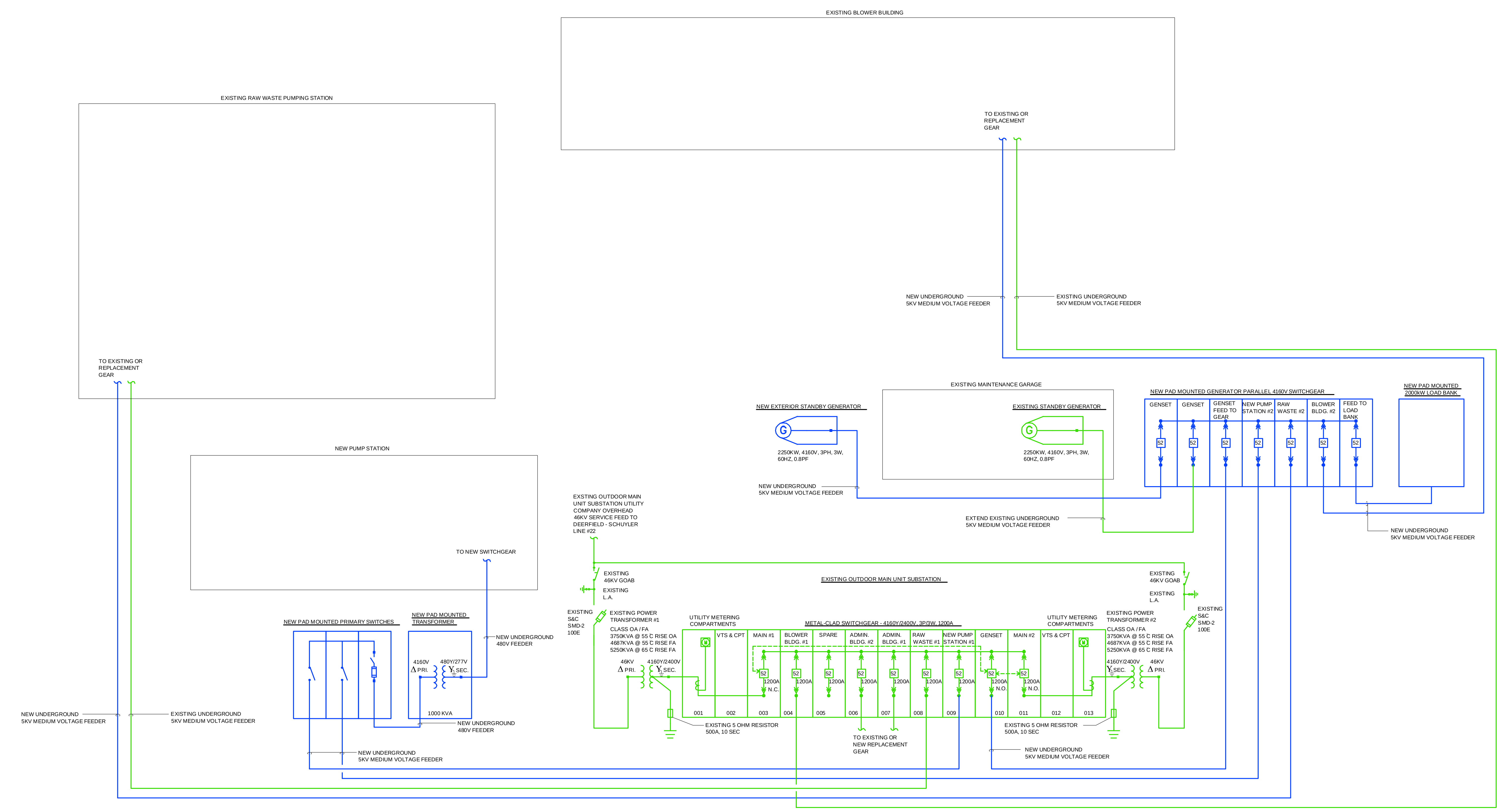
ELECTRICAL EXPANSION
OPTION #1

Job Number	8614782
Revision	A
Date	AUGUST 2012

Figure 02

200 John James Audubon Parkway Suite 101, Amherst NY 14228 USA **T** 1 716 691 8503 **F** 1 716 691 8506 **E** amhmail@ghd.com **W** www.ghd.com

Plot Date: 30 August 2012 - 3:25 PM Cad File No: G:\86\14782\CADD\Drawings\Electrical\Electrical One-Line Diagram.dwg



DRAWING NOT TO SCALE

SHUMAKER Consulting Engineering & Land Surveying, P.C.

Brown AND Caldwell

GHD

WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

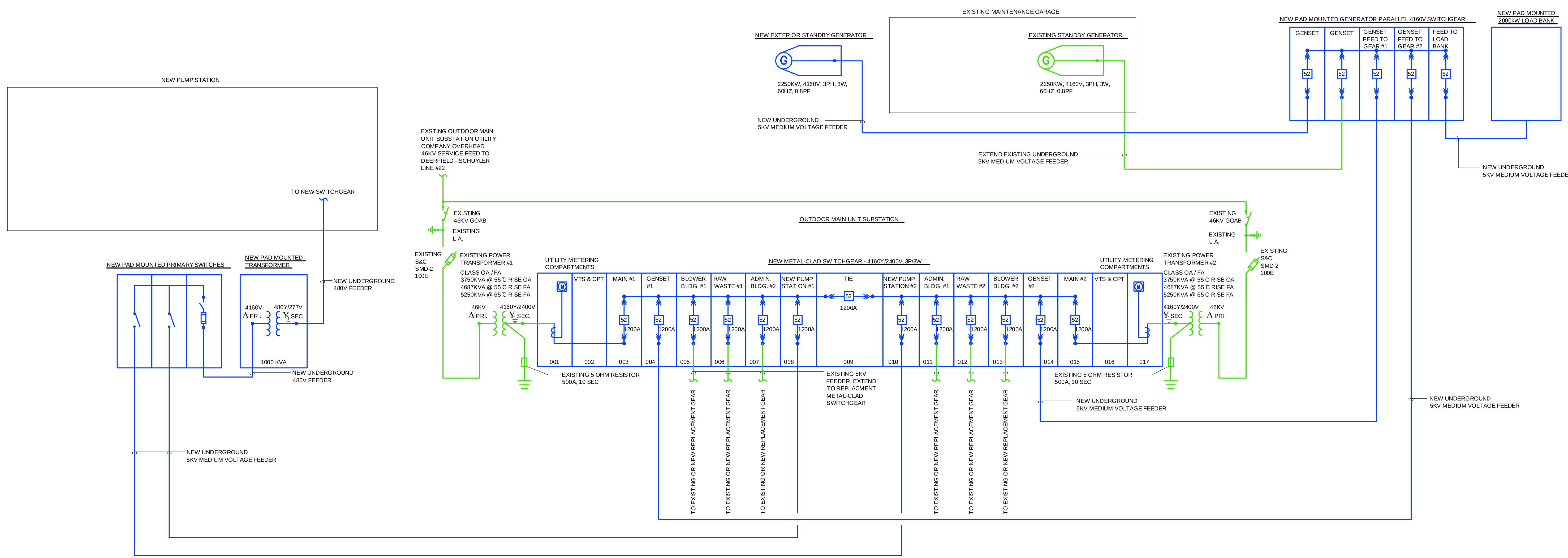
ELECTRICAL EXPANSION
OPTION #2

Job Number	8614782
Revision	A
Date	AUGUST 2012

Figure 03

200 John James Audubon Parkway Suite 101, Amherst NY 14228 USA **T** 1 716 691 8503 **F** 1 716 691 8506 **E** amhmail@ghd.com **W** www.ghd.com

Plot Date: 30 August 2012 - 3:25 PM Cad File No: G:\86\14782\CADD\Drawings\Electrical\Electrical One-Line Diagram.dwg



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Brown AND Caldwell

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WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

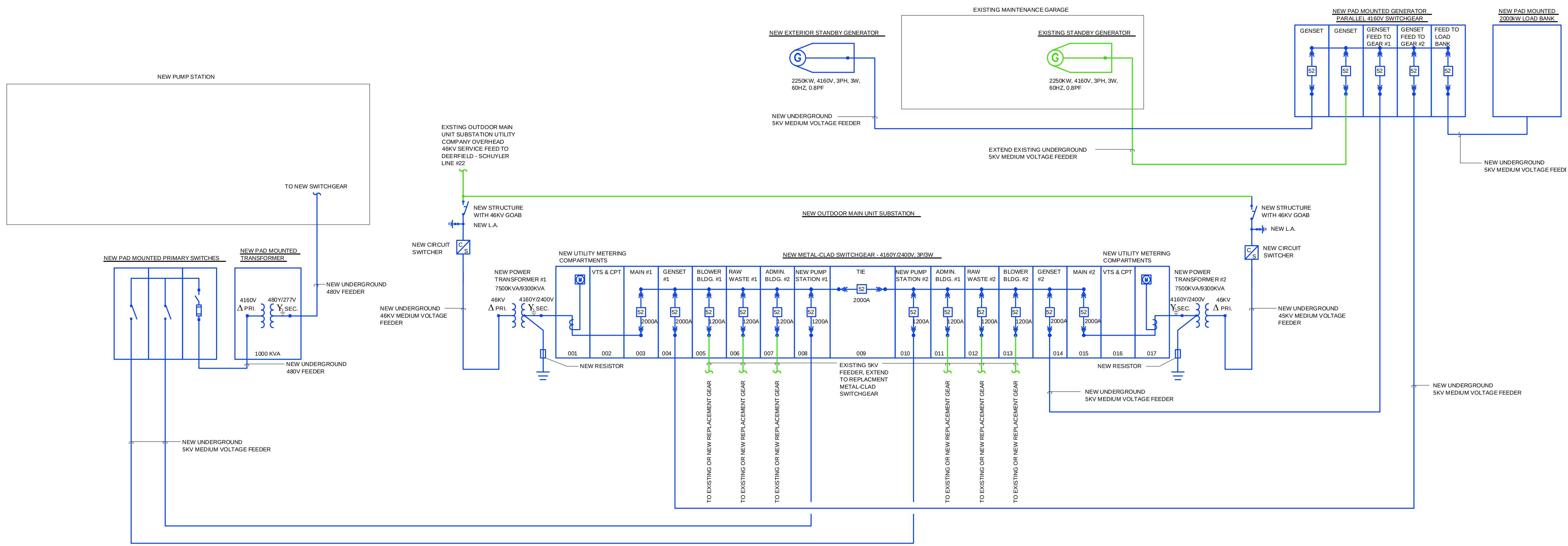
ELECTRICAL EXPANSION
OPTION #3

Job Number	8614782
Revision	A
Date	AUGUST 2012

Figure 04

200 John James Audubon Parkway Suite 101, Amherst NY 14228 USA **T** 1 716 691 8503 **F** 1 716 691 8506 **E** amhmail@ghd.com **W** www.ghd.com

Plot Date: 30 August 2012 - 3:25 PM Cad File No: G:\86\14782\CADD\Drawings\Electrical\Electrical One-Line Diagram.dwg



DRAWING NOT TO SCALE

WPCP AND SCPS EVALUATION
ONEIDA COUNTY, NY

ELECTRICAL EXPANSION
OPTION #4 - RECOMMENDED

Job Number	8614782
Revision	A
Date	AUGUST 2012

Figure 05

200 John James Audubon Parkway Suite 101, Amherst NY 14228 USA **T** 1 716 691 8503 **F** 1 716 691 8506 **E** amhmail@ghd.com **W** www.ghd.com

Plot Date: 30 August 2012 - 3:25 PM Cad File No: G:\86\14782\CADD\Drawings\Electrical\Electrical One-Line Diagram.dwg