

DEP Southwest Regional Office

400 Waterfront Drive

Pittsburgh, Pa 15222

## **SOUTH PARK TOWNSHIPS POLLUTION REDUCTION PLAN**

### **PROPOSAL for NPDES GENERAL PERMIT PAG-13 MS-4**

*South Park Townships projected requirements for "Nutrient" total phosphorous removal based on load requirements a: "see attached Parsing/Load requirements" are a five (5%) reduction or 203.36 lbs. over a five (5) year period. Also see b: "2014 WWTP Upgrades Project Part II NPDES WQM Permit Application Basis of Design Report".*

*South Park Township is proposing to accomplish this total and more thru the following actions:*

- 1) The South Park/Bethel Park Sewage Treatment Plant located in South Park Township and 25% funded by South Park is currently well along with the current \$10.5 million dollar upgrade to the facility, as permitted by DEP. The projection for future sediment /sludge removal, when completed, is to remove an additional 534 lbs. /per day. c: "see attached Bethel Park treatment and Union Township removal calculations" also attached d:"Bethel Park Sewage Treatment Plant Sludge Study"
- 2) Additionally, as many as 24 on-lot sewage facilities in Union Township are in failure or illegally connected to South Park's conveyance system. This presents a problem rectifying and remediation downstream because the illegal flow into Piney Fork Creek is above the denigrated area. South Park Township is also working with DEP and Union Township on this matter, again South Park Township has taken the lead on this matter and discovered this 40 year old problem based upon outfall testing. It is South Parks position this remedy of removing the failed on lot sewage systems will contribute an additional 16 lbs. /day solids removed. Attached e "see Union Township, Estimation of Solids Removed".
- 3) Verification, testing and sampling with the aforementioned will be reported annually to DEP thru the MS\_4 annual compliance report.
- 4) Attachments
  - a: " Parsing/Load requirements"
  - b: "2014 WWTP Upgrades Project Part II NPDES WQM Permit Application Basis of Design Report".
  - c: Bethel Park treatment and Union Township removal calculations"
  - d:"Bethel Park Sewage Treatment Plant Sludge Study"
  - e: Union Township, Estimation of Solids Removed" .
  - f: PRP Map, planning area and existing BMPs.

Thank you for your time and consideration with this process.

Bruce R. Beaver

Director of Public Works

South Park Township

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## **Attachment A: Parsing/ Load Requirements**

## Parsing/Load Requirements

### Load Requirement Calculations

%Pervious in Urbanized Area	81	
% Impervious in Urbanized Area	19	
Total Acres in Urbanized Area	5625.3	
Total Load Requirements	(acres x %Pervious x .84) + (acres x %Impervious x 2.28)	
	(5625.3 x .81*.84) + (5625.3 x .19 x 2.28)	
	6264.33	
5% TP Reduction Required before Parsing	313.2165	

### Parsing

County Park		Bureau of Mines	
County Park (Total Acres)	1363.00	Bureau of Mines Total	242.00
Impervious:		Impervious:	
Roads	81.90	Roads	14.37
Golf Cart Paths	4.06	Other	53.15
Other	36.69	Total Impervious	67.52
Total Imperivous	122.65	Pervious	
Total Pervious:		Total Acres- Impervious Acres	174.48
Total Acres - Total Impervious	1240.35	Impervious Load	153.95
Impervious Load	279.64	Pervious Load	146.56
Pervious Load	1041.89	Total Parsed Load	300.51
Total Parsed Load	1321.54	<b>Waste Management Landfill</b>	
<b>State and County Roads</b>		Landfill Total	96.7
Total Acres	95.05	Impervious	
Total Impervious	95.05	Area 1	1.44
Total Load	216.714	Area 2	1.31
<b>South Park Clubhouse</b>		Total Impervious	2.75
Total	1.65	Pervious:	
Impervious	1.65	Total - Total Impervious	93.95
Total Load	3.762	Impervious Load	6.27
<b>Brownsville Road Developments</b>		Pervious Load	78.92
Total Impervious	7.74	Total Load	85.19
Total Pervious	0.00	<b>Lick Run Treatment Plant</b>	
Gills Café	2.28	Treatment Plant Total	5.29
Post Office	3.28	Impervious	
Bavarian Village	11.17	Area 1	0.99
Dentist Office	0.91	Area 2	0.31
Total Load	17.65	Area 3	1.13
		Total Impervious	2.43
		Pervious:	
		Total-Total Impervious	2.86
		Impervious Load	5.54
		Pervious Load	2.40
		Total Load	7.94

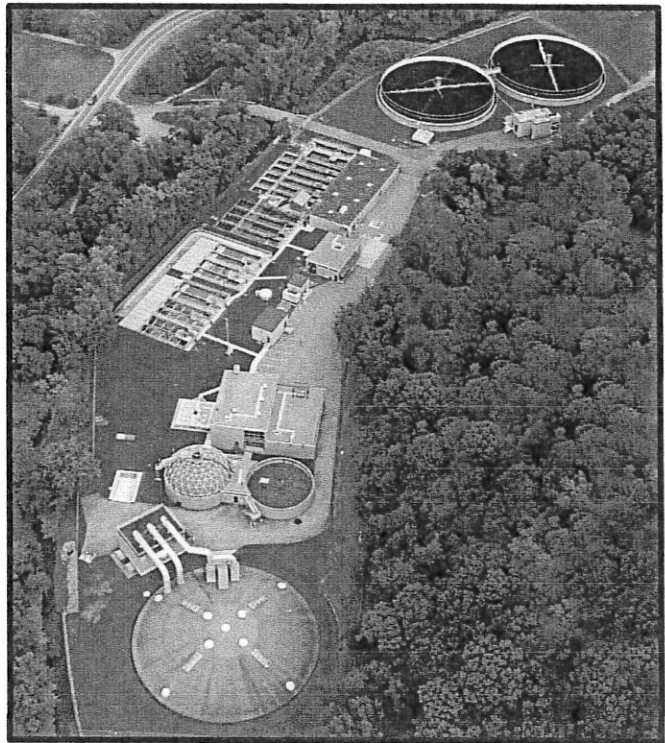
## Existing BMPS

High School ( Dry Detention Pond)		Hidden Ridge (Detention Pond Pond)	
Total	25.00	Total	110.00
Total Impervious	11.18	Impervious	
Total Pervious:	13.82	Roads	13.56
Impervious Load	10.20	Condos	12.75
Pervious Load	4.64	Other	7.75
Total Load	14.84	Total Impervious	34.06
<b>Bonnie Dell (Dry Detention Pond)</b>		Pervious:	
Total	42.40	Total-Total Impervious	75.94
Impervious:		Impervious Load	31.06
Roads	1.22	Pervious Load	25.52
Other	8.14	Total Load	56.58
Total Impervious:	9.36	<b>Piney Ridge (Dry Detention Pond)</b>	
Pervious:		Total	46.00
Total-Total Impervious	33.04	Impervious:	
Impervious Load	8.54	Roads	5.80
Pervious Load	11.10	Other	9.13
Total Load	19.64	Total Impervious	14.93
<b>Imperial Ridge (Dry Detention Pond)</b>		Pervious:	
Total	17.30	Total-Total Impervious	31.07
Impervious:		Impervious Load	13.62
Roads	1.85	Pervious Load	10.44
Other	4.44	Total Load	24.06
Total Impervious	6.29	<b>Elementary School (Dry Detention Pond)</b>	
Pervious:		Total	40.90
Total-Total Impervious	11.01	Total Impervious	6.92
Impervious Load	5.74	Pervious:	
Pervious Load	3.70	Total-Total Impervious	33.98
Total Load	9.44	Impervious Load	6.31
<b>Parkford Apartments (Dry Detention Pond)</b>		Pervious Load	
Total	9.28	Total Load	17.73
Total Impervious	3.25	<b>South Park Trails (Dry Detention Pond)</b>	
Pervious		Total	11.20
Total-Total Impervious	6.03	Total Impervious	2.90
Impervious Load	2.96	Total Pervious	8.30
Pervious Load	2.03	Impervious Load	2.64
Total Load	4.99	Pervious Load	2.79
<b>Heights of South Park (Dry Detention Pond)</b>		Total Load	
Total	8.28	Total Load	5.43
Total Impervious	3.22	<b>Mount Vernon (Dry Detention Pond)</b>	
Total Pervious:	5.06	Total	6.24
Impervious Load	2.94	Total Impervious	2.51
Pervious Load	1.70	Total Pervious	3.73
Total Load	4.64	Impervious Load	2.29
		Pervious Load	1.25

		Total Load	3.54
<b>Glen Creek Tennis Club (Dry Detention Pond)</b>		<b>Brookfield (Dry Detention Pond)</b>	
Total	7.55	Total	21.90
Total Impervious	2.40	Total Impervious	6.37
Total Pervious	5.15	Total Pervious	15.53
Impervious Load	2.19	Impervious Load	5.81
Pervious Load	1.73	Pervious Load	5.22
Total Load	3.92	Total Load	11.03
<b>Della Strada (Dry Detention Pond)</b>		<b>Hurl Plan (Dry Detention Pond)</b>	
Total	19.10	Total	2.52
Total Impervious	2.42	Total Impervious	0.86
Total Pervious	16.68	Total Pervious	1.66
Impervious Load	2.21	Impervious Load	0.78
Pervious Load	5.60	Pervious Load	0.56
Total Load	7.81	Total Load	1.34
<b>Evans Park (Constructed Wetland)</b>		<b>Surry Plan (Dry Detention Pond)</b>	
Total	17.70	Total	12.50
Total Impervious	2.62	Total Impervious	6.15
Total Pervious	14.88	Total Pervious	6.35
Impervious Load	5.08	Impervious Load	5.61
Pervious Load	10.62	Pervious Load	2.13
Total Load	15.70	Total Load	7.74
<b>Bideford Pond (Dry Detention Pond)</b>		<b>GreenBriar Pond (Dry Detention Pond)</b>	
Total	13.30	Total	22.50
Total Impervious	5.32	Total Impervious	8.55
Total Pervious	7.98	Total Pervious	13.95
Impervious Load	4.85	Impervious Load	7.80
Pervious Load	5.70	Pervious Load	4.69
Total Load	10.55	Total Load	12.48
<b>Parsing Totals</b>		<b>High School Stadium (Dry Detention Pond)</b>	
Total Area Parsed	1569.43	Total	25.00
Total Impervious Area Parsed	299.79	Total Impervious	5.99
Total Pervious Area Parsed	1511.64	Total Pervious	19.01
Total Load Parsed	1652.79	Impervious Load	5.46
<b>Load Requirements After Parsing</b>		Pervious Load	6.39
Total Load Before Parsing	6264.33	Total Load	11.85
Total Parsed Load	1652.79		
New Total Load	4611.54		
<b>Existing BMP Reductions</b>			
Total Area within BMPS	458.67		
Total Impervious Area	135.30		
Total Pervious Area	323.17		
Total Load Reduction	243.31		
New Total Load including Parsing and Existing BMPS	4368.23	<b>5% Reduction</b>	<b>218.41</b>

**Attachment B: 2014 WWTP Upgrades Project  
Part II NPDES WQM Permit Application Basis  
of Design Report**

The Bethel Park Municipal Authority  
NPDES Permit No. PA0027618



**2014 WWTP Upgrades Project  
Part II NPDES WQM Permit Application  
Basis of Design Report**

**January 2015**

**Prepared By:**



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**NPDES Permit No. PA0027618  
Part II WQM Permit Application  
2014 WWTP Upgrades Project**

**BASIS OF DESIGN REPORT**

**GENERAL**

Wade Trim, Inc.'s Pittsburgh, Pennsylvania office was retained to complete the second phase of treatment plant upgrades for the Bethel Park Municipal Authority (BPMA)'s Piney Fork treatment plant. Located in Allegheny County, PA, the BPMA WWTP provides wastewater conveyance and treatment services to Bethel Park and South Park Township. The Authority has a 7 member Authority Board, and the Piney Fork WWTP is managed by Scott Dunn, Superintendent. The treatment plant is rated for an average design capacity of 4.92 MGD, and discharges to Piney Fork. The plant utilizes the following existing processes in treatment of domestic, light industrial/commercial wastewater, and landfill leachate from South Hills Landfill:

- Headworks
- Primary clarification (5 rectangular units)
- Trickling filters (2 circular units with plastic media)
- Final clarification (5 rectangular units)
- Disinfection via chlorination
- Dechlorination via sodium bicarbonate

The existing solids handling process includes the following:

- Sludge thickening (1 unit)
- Grit removal of sludge (1 unit)
- Anaerobic digestion (1 primary unit, 1 secondary unit/gas holder)
- Sludge dewatering via belt filter press

The WWTP is currently undergoing construction of Phase I of a multi-phase treatment plant upgrade process. Phase I includes retrofits to the existing digesters as well as various minor improvements throughout the plant. Phase II (current phase) includes the design of two new circular final clarifiers, a new UV system, and design of a new UV building. Phase II also includes the preliminary design of a nutrient removal process that will be receive final design and construction at a later date, as nutrient removal requirements are incorporated into the Authority's permit. Provisions will be included in this phase including equipment and land space for the future nutrient removal processes. These provisions will allow the current design to work adequately with the future nutrient removal design as well as insure that the available facilities and location for those facilities are available. The UV building to be constructed as part of this project will have adequate space for future blowers, pumping, and chemical feed equipment, all anticipated to be needed for the nutrient removal processes.

The project will be bid in two contracts as follows:

- Contract One: General Construction
- Contract Two: Electrical Work

Contract One will also include all necessary HVAC and plumbing work. Contract Two will also include any instrumentation/system integration that is required.

A number of permits have been applied for as part of this project. The application for the Water Obstruction/Encroachment and US Army Corps of Engineers permits have been submitted to the PaDEP as a joint permit package. The permits applied for are as follows:

Permit Name/Number/Description
Joint Permit Package (Army Corps and PaDEP)
Erosion and Sedimentation Control/Stormwater Permit
Grading Permit (South Park Township)
Part II Permit for Construction

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## **EXECUTIVE SUMMARY – PLANT UPGRADES AND IMPROVEMENTS**

The WWTP has an NPDES Permit that is active and due to expire in June of 2018. Operation of the WWTP is currently within the requirements set forth by the WWTP's current permit. The design of the treatment plant upgrades will insure that the facility is able to meet current and future permits at anticipated additional permit requirements (including total nitrogen and total phosphorus). Phase II includes the design of two circular final clarifiers, a new UV system for disinfection, a new UV building, construction of a new plant outfall, and provisions for future nutrient removal processes.

Final clarifiers - The existing final clarifiers are rectangular in shape and mimic the design of the existing primary clarifiers. Although the final clarifiers have performed adequately to this point, they have insufficient capacity for the future nutrient removal requirements that will be imposed on the WWTP. When the nutrient removal process (suspended growth/activated sludge) is implemented, the quantity of solids and solids concentration will be much higher. Larger final clarifiers with an appropriate flocculation zone are required to insure that adequate solids removal is achieved. This project is therefore proposing the installation of two new 100-foot circular clarifiers with a flocculation center well, motor driven spiral mechanisms, and baffling to improve flow distribution patterns, prevent short circuiting, and enhance solids removal. Nutrient removal calculations have been performed and the amount of solids to be generated has been determined. The new final clarifiers have been designed to account for this additional solids generation. The new final clarifiers meet the DEP regulatory requirements for sizing for detention time, surface overflow rate, and solids removal, up to peak flows of 20 MGD.

Ultraviolet (UV) disinfection - The existing disinfection process at the WWTP is a two phased process that includes gaseous chlorine for disinfection and sulfur dioxide for dechlorination. More stringent regulations are anticipated for dechlorination, and the concern for plant workers handling chlorine and sulfur dioxide has warranted the installation of a new UV facility. The UV system will permit the plant to provide the necessary disinfection without the need for dechlorination via sulfur dioxide. The UV process is also much safer for plant workers and removes the need for chemical handling. The UV process chosen for the design is a vertical bank installation. The UV design is adequate up to a peak flow of 25 MGD. The design chosen for the vertical banks includes two channels of two banks in series, for a total of four banks. The UV system will provide a minimum UV dosage of 30,000 uWatts-secs/cm<sup>2</sup> with all lamps in service. Based on the calculations performed for the sizing, configuration, and lamps provided, the UV system is adequate to provide the necessary disinfection to the levels required by the Authority's permit.

UV building – A new UV building is proposed as part of this project, and will house the UV equipment as well as the following additional equipment and future equipment considerations:

- UV equipment
- UV removal mechanism for maintenance

- UV cleaning system
- New waste sludge/scum pumps
- New effluent water pumps
- Space for future chemical removal (phosphorus) equipment
- Space for future RAS pumps for nutrient removal process
- Space for future blowers for nutrient removal process
- Electrical room for electrical equipment, controls, and MCCs
- Bathroom and washdown facilities
- Heating and HVAC

The UV building will also be constructed adjacent to the new plant effluent channel which will convey flow to a newly constructed outfall. The new effluent channel will have a new effluent weir and flow meter to record plant flow. The effluent flow meter will have electrical and control equipment installed in the new electrical room within the UV building.

New Plant Outfall – The discharge from the new UV system will be conveyed to a new outfall that is proposed to be constructed as part of this project. A new outfall is required because the existing outfall is located at the discharge of the chlorine contact tanks, which will no longer be part of the treatment process. The new outfall is located adjacent to the new UV building, approximately 550 feet downstream of the existing outfall. The existing outfall will be decommissioned as part of this project.

Future nutrient removal processes – Based on correspondence received from the PaDEP and meetings regarding current and future projects, the Authority has been informed that the PaDEP is in the process of analyzing the nutrient issues in Pennsylvania’s various receiving waters. Although regulations have been focused more on the central and eastern part of the state because of the condition of the Chesapeake Bay, regulations for nitrogen and phosphorus removal will be imposed in this part of the state in the near future. The effort and cost to address nutrient removal is quite substantial, therefore the Authority felt that a phased approach to construction would space the construction and financial burden over a more reasonable time period. Although this proposed project does not directly address nutrient removal, provisions are being designed into this project that will allow future nutrient removal facilities to be constructed. The calculations have been performed (see Part III – Proposed Facilities) to size the denitrification facilities to determine the space required for those facilities. This required space is being provided for the future denitrification tanks, and space is also being designed into the new UV building for equipment that will provide chemical precipitation for phosphorus removal. Additionally, space is being provided in the new UV building for future pumps and blowers, and some site piping is being constructed now so that the future nutrient removal facilities can be tied into the existing facilities as efficiently as possible.

Existing facilities to remain as part of Phase II – It is being proposed for Phase II of the treatment plant upgrade design that some of the existing facilities currently in operation are capable of handling the flows projected for the WWTP upgrades. These facilities include the headworks, primary clarifiers, and trickling filters.

### Coordination of Existing Plant Operation During Construction

The new final clarifiers and UV system can be constructed without disturbing the operation of the existing facility. The existing plant will remain in full operation during the entire construction period. Some coordination with sludge handling will be required when the tie-in of the new final clarifier sludge pipe is made to the existing sludge piping system. This tie-in will be performed when sludge removal from the clarifiers is not needed.

The final tie-in required to convey flow to the new facilities when complete will be at the existing secondary trickling filter effluent box. Currently flow is conveyed from this box to the existing final clarifiers. A new pipe will be constructed from the effluent chamber of the secondary trickling filter effluent box to the new final clarifier influent box, and that connection will be made when the final clarifiers and UV system are ready for operation.

### Act 537 Special Study

Per PaDEP requirements, an Act 537 Special Study was completed for the proposed WWTP upgrades. The initial study was submitted in October of 2014, with comments received regarding incompleteness in a letter from the PaDEP dated 10/27/14. Those comments were addressed and the revised Study was submitted to the PaDEP on 11/14/14. As of the date of this Part II Permit submission, a revised response has not been received from the PaDEP. It has been indicated that a revised response shall be forthcoming before the end of January of 2015.

## Part I - Treatment Plant Summary

The WWTP currently operates at an average daily flow (ADF) of 4.92 MGD, with a peak instantaneous flow of approximately 20 MGD. The plant's current permit is due to expire June of 2018. The following table represents BPMA's current permit limits:

### Current Permit Limitations

Parameter	Monthly Average	Weekly Average	Instantaneous Max
CBOD <sub>5</sub> – May 1 to Oct 31	10 mg/L	15 mg/L	
CBOD <sub>5</sub> – May 1 to Oct 31	410 lb/day	615 lb/day	
CBOD <sub>5</sub> – Nov 1 to Apr 30	25 mg/L	38 mg/L	
CBOD <sub>5</sub> – Nov 1 to Apr 30	1026 lb/day	1559 lb/day	
Suspended Solids (TSS)	30 mg/L	45 mg/L	
Suspended Solids (TSS)	1231 lb/day	1847 lb/day	
NH <sub>3</sub> – May 1 to Oct 31	2 mg/L	3 mg/L	
NH <sub>3</sub> – May 1 to Oct 31	82 lb/day	123 lb/day	
NH <sub>3</sub> – Nov 1 to Apr 30	4 mg/L	6 mg/L	
NH <sub>3</sub> – Nov 1 to Apr 30	164 lb/day	246 lb/day	
Total Residual Chlorine	0.16 mg/L		0.53 mg/L
Dissolved Oxygen	6.0 mg/L min.		
Fecal Coliform – May 1 to Sep 30	200/100 mL Geometric mean		1,000/100 mL Geometric mean
Fecal Coliform – Oct 1 to Apr 30	2,000/100 mL Geometric mean		10,000/100 mL Geometric mean
pH	Between 6-9 S.U.		

It should be noted that the Piney Fork treatment plant does accept landfill leachate on a daily basis from the South Park Landfill, owned and operated by Waste Management. There is an agreement in place to limit the amount of flow the treatment plant will accept on a monthly basis. Currently this equates to less than 50,000 gpd, or less than 2% of the plant's flow. It should also be noted that the Authority has the ability to limit the acceptance of leachate from the landfill should there be an issue with the plant's influent loading.

Permit Influent Data, Last Three Years (2012-2014)

Parameter	2012 yearly average (mg/L)	2013 yearly average (mg/L)	2014 yearly average (mg/L)
CBOD <sub>5</sub> – May 1 to Oct 31	118.52	136.53	106.45
CBOD <sub>5</sub> – Nov 1 to Apr 30	85.29	17.3	105.3
Suspended Solids (TSS)	106	127	123
NH <sub>3</sub> – May 1 to Oct 31	22.2	22.5	19.9
NH <sub>3</sub> – Nov 1 to Apr 30	16.4	17.3	22.1

Permit Effluent Data, Last Three Years (2012-2014)

Parameter	2012 yearly average (mg/L)	2013 yearly average (mg/L)	2014 yearly average (mg/L)
CBOD <sub>5</sub> – May 1 to Oct 31	7.39	10.34	8.96
CBOD <sub>5</sub> – Nov 1 to Apr 30	6.49	7.54	9.63
Suspended Solids (TSS)	15	17	21
NH <sub>3</sub> – May 1 to Oct 31	0.62	0.78	0.83
NH <sub>3</sub> – Nov 1 to Apr 30	0.43	0.45	0.91
Total Residual Chlorine	0.11	0.08	0.05
Fecal Coliform – May 1 to Sep 30	6	14	12
Fecal Coliform – Oct 1 to Apr 30	7	15	9

Design Loading and Facilities Data

The following table represents the design loading and facilities design data for the existing and proposed facilities:

**Design Loading Data\***

Design Parameter	Existing Facilities Design	Present Operating Data	Proposed Facilities Design
<b>Hydraulic Design Capacity</b>			
a. Domestic (mgd)	4.92		4.92
b. Industrial (mgd)			
c. Total (mgd)	4.92		4.92
Average Annual Flow (mgd)		3.8	
Design Year for Operating Data		2013	
Flow from Landfill Leachate (mgd)	0.05	0.05	0.05
<b>Total Design Wastewater Data</b>			
a. Peak instantaneous flow (mgd)	20	20	20
b. Peak hourly flow (mgd)			
c. Max daily flow (mgd)			
d. Max monthly avg flow (mgd)		5.6	
e. Organic design capacity Max daily (lbs BOD <sub>5</sub> /day)	6,155	5,316	6,155
f. Max daily TSS conc. (mg/L)	45		45

*\* Note – presenting operating data based on data obtained in 2013.*

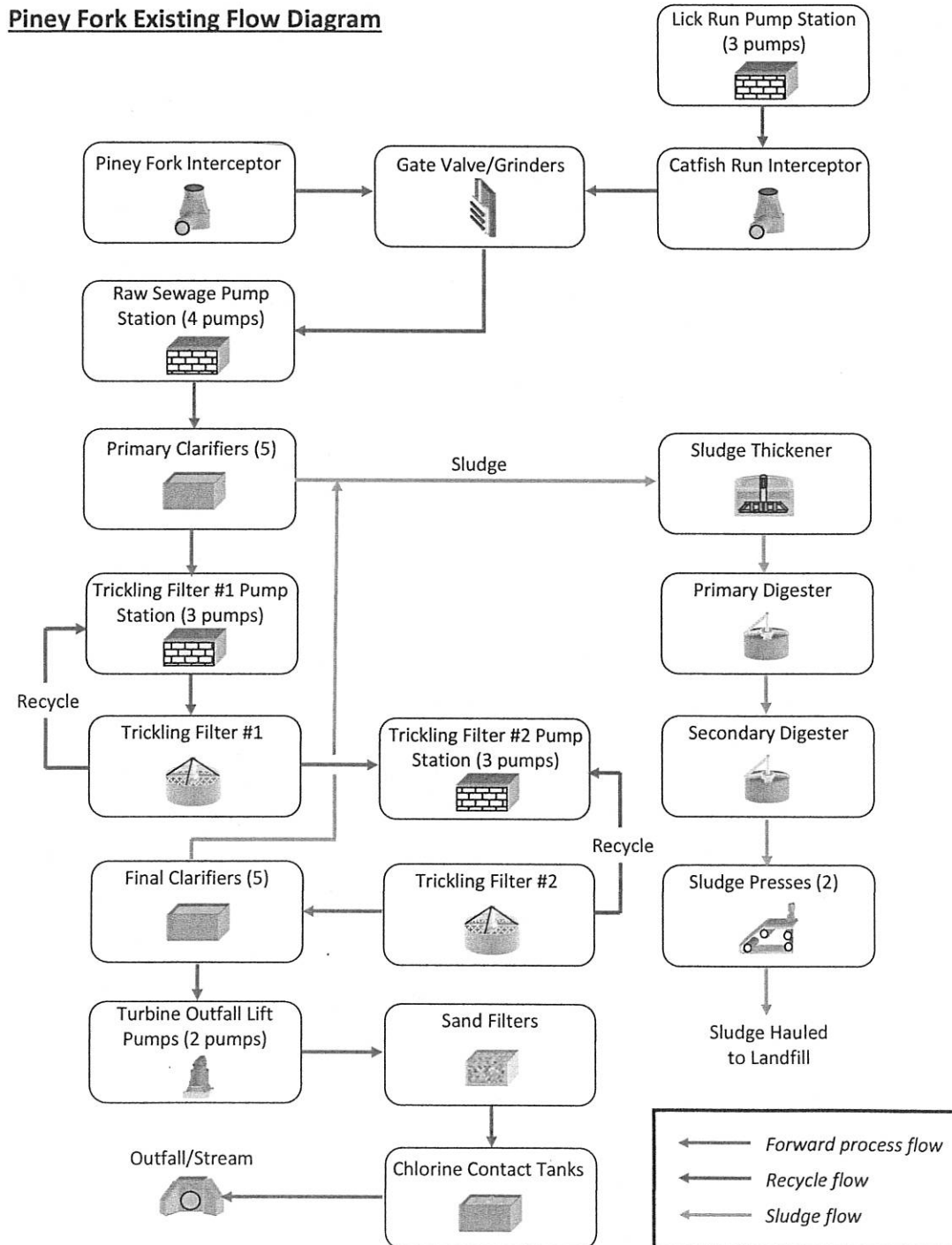
**Facilities Design Data**

Treatment Unit	Number of units		
	Existing	To be Abandoned	Proposed New Units (on-site)
Raw Sewage Pumps	4	0	0
Screens	2	0	0
Grit Removal	1	0	0
Primary Clarifiers	5	0	0
Trickling Filter Pumps	3	0	0
Trickling Filters	2	0	0
Final Clarifiers	5	5	2
Chlorine Contact Tanks	2	2	0
UV Disinfection System	0	0	1

## Part II – Existing Facilities

### Existing Flow Schematic and Hydraulic Profile

The flow schematic for the current operation of the Piney Fork treatment plant as well as the hydraulic profile diagram of the existing plant are provided as follows:





**Raw Sewage Pump Stations (Existing Off-Site and Existing On-Site)**

Existing Facilities (Off-Site)

The existing off-site Lick Run sewage pump station currently houses 3 pumps, and their characteristics and operating capacity are summarized in the table below. The average flow in 2013 for the Lick Run Pump Station was 0.352 MGD, well below the design capacity of the pump station.

<b>Pump No.</b>	<b>Pump capacity (gpm)</b>	<b>Pump horsepower (hp)</b>
1	750	75
2	750	75
3	2080	200
Total pump station capacity =		2.5 MGD

Existing Facilities (On-Site)

There are two main pump stations that exist at the plant (on-site). The first pump station is the raw sewage pump station, which contains four pumps and was recently upgraded in 2010. The second pump station is the trickling filter pump station, which has three pumps that control flow to and from the trickling filters. The trickling filter pumps were installed in 2006 as part of the Secondary Upgrade Project. The characteristics of these pump stations are summarized in the tables below.

*Raw Sewage Pump Station*

<b>Pump No.</b>	<b>Pump capacity (gpm)</b>	<b>Pump horsepower (hp)</b>
1	5,000	60
2	5,000	60
3	5,000	60
4	5,000	60
Total pump station capacity =		21.6 MGD

*Trickling Filter Pump Station*

<b>Pump No.</b>	<b>Pump capacity (gpm)</b>	<b>Pump horsepower (hp)</b>
1	6,950	150
2	6,950	150
3	6,950	150
Total pump station capacity =		20 MGD

## **Existing Headworks**

The existing headworks for the treatment facility consist of two screening devices (comminutors) in the influent channel, with the individual screening channels being split off of the main influent channel. The comminutors catch large debris, rags, and other significant solids that could damage downstream equipment. There are no true grit removal facilities installed at the plant for raw sewage flow. After the flow passes through the comminutors, the flow is pumped up to the primary clarifiers. It is not being proposed to modify the existing headworks system as part of this project.

## Existing Primary Clarification

### Description

The BPMA WWTP currently has 5 rectangular primary clarifiers. Under normal operating conditions, only three of the primary clarifiers are used to treat flow, with the remaining primary clarifier being used only under peak flow conditions.

### Existing Facilities – Design Average flow of 4.92 MGD (3 clarifiers on-line)

*Primary Clarifiers No. 1 – 5, 74 feet long by 27.5 feet wide, avg. 10 foot SWD*

Surface area (each) = 2035 ft<sup>2</sup>

Volume (each) = 152,210 gal

Total weir length (each) = 94.7 feet

Detention time @ 1.64 MGD (3 clarifiers on-line) = 2.23 hours

SOR @ 1.64 MGD = 805.7 gal/day/ft<sup>2</sup>

Weir loading rate @ 1.64 MGD = 17,336 gal/day/ft

### Existing Facilities – Peak flow of 20 MGD (5 clarifiers on-line)

*Primary Clarifiers No. 1 – 5, 74 feet long by 27.5 feet wide, avg. 10 foot SWD*

Surface area (each) = 2035 ft<sup>2</sup>

Volume (each) = 152,210 gal

Total weir length (each) = 94.7 feet

Detention time @ 4 MGD (5 clarifiers on-line) = 0.91 hours

SOR @ 4 MGD = 1965.2 gal/day/ft<sup>2</sup>

Recommended minimum detention time and maximum surface overflow rate for peak flows for primary clarifiers are 1 hour and 2000 gal/day/ft<sup>2</sup>, respectively. As can be seen from the previous calculations, the primary clarifiers are providing slightly less than the 1 hour minimum recommended detention time but are within the recommended surface overflow rate peak of 2000 gal/day/ft<sup>2</sup>. The plant has seen numerous peak flow events and the primary clarifiers have performed adequately. The existing facilities should be adequate to continue to treat peak flows during high flow events.

## Existing Trickling Filters

### Description

Prior to the completion of the Secondary Plant Upgrade Project (2006), BOD and NH<sub>3</sub>-N were removed by processing the wastewater in the trickling filters through rock media and distributors that were originally installed in 1957. Over time, the rock underdrain system within the filters was crushed and did not operate as intended. The rock media provided limited surface area for biological growth, and there was no means to control the speed of the rotary distributors, which were hydraulically driven.

The Secondary Upgrade Project provided for a major renovation of the trickling filters, including a new secondary trickling filter pump station. The existing trickling filter pump station, which was originally designed to pump to both trickling filters, was renovated so that it could only pump to the primary trickling filter. Dedicating a pump station to each trickling filter allowed for a more definitive flow path to each trickling filter, much better control of return flows, and the elimination of short circuiting of flow that could occur when only one pump station existed to pump to both trickling filters. This also allowed the primary filter to become a BOD removal filter and the secondary filter to remove NH<sub>3</sub>-N. In addition to the separation of filters, the rock media were replaced with plastic cross-flow media, the media was raised up to allow for natural ventilation, and motorized rotary distributors were installed that better allow the plant to control the speed of the distributors and also control the rate of flow distribution to the filter media.

The trickling filters have been providing anticipated BOD and NH<sub>3</sub> removals since the upgrade. It is not intended to modify the existing trickling filters as part of this project. It should be noted that flow will be routed to the new final clarifiers from the existing secondary trickling filter flow distribution box, which currently routes flow to the existing final clarifiers. Testing has been done on the trickling filters by an independent laboratory over a six month period to continually monitor their performance. The testing was also performed to assess the levels of nutrients in the effluent of the trickling filters. The table below summarizes the testing performed and those results:

Sampling parameter	2/17/12	3/28/12	5/23/12	6/26/12	8/29/12
CBOD	24.1 mg/L	10.6 mg/L	4.1 mg/L	30.1 mg/L	26.5 mg/L
TSS	90 mg/L	42 mg/L	95 mg/L	89 mg/L	56 mg/L
NH <sub>3</sub> (as N)	0.37 mg/L	0.64 mg/L	0.26 mg/L	0.46 mg/L	0.31 mg/L
Total P	3.4 mg/L	2.05 mg/L	3.55 mg/L	5.1 mg/L	3.74 mg/L
Total Nitrogen as N	11.6 mg/L	4.8 mg/L	8.6 mg/L	6.5 mg/L	6.09 mg/L

## Final Clarifiers

### Description

The BPMA WWTP currently has 5 rectangular final (secondary) clarifiers. The existing final clarifiers are original to the plant and were constructed in 1957. They are the exact same configuration as the primary clarifiers in terms of number and dimensions. Although the final clarifiers have performed adequately for the flows and NPDES requirements for the plant, the final clarifier configuration is not optimal for removal of secondary process solids.

### Existing Facilities – Design Average flow of 4.92 MGD (all clarifiers on-line)

#### **Secondary Clarifiers No. 1 – 5, 74 feet long by 27.5 feet wide, avg. 10 foot SWD**

Surface area (each) = 2035 ft<sup>2</sup>

Volume (each) = 152,210 gal

SOR @ 0.98 MGD = 481.5 gal/day/ft<sup>2</sup>

### Existing Facilities – Peak flow of 20 MGD (all clarifiers on-line)

#### **Secondary Clarifiers No. 1 – 5, 74 feet long by 27.5 feet wide, avg. 10 foot SWD**

Surface area (each) = 2035 ft<sup>2</sup>

Volume (each) = 152,210 gal

SOR @ 4 MGD = 1965.2 gal/day/ft<sup>2</sup>

Although solids loading rate is a standard design factor considered for secondary clarifiers following aeration tanks in the activated sludge process, solids loading is not typically evaluated as a design criteria for secondary clarifiers following fixed film media. This is typically because the solids loading is 20 to 40 times less than that of activated sludge, and the type of sludge is quite different. Surface overflow rates and appropriate flocculation center well sizing are the most common factors considered for clarifiers following trickling filters. The recommended the surface overflow rate at average flow and maximum surface overflow rate for peak flows for secondary clarifiers following trickling filters (Clarifier Design, MOP FD-8, Water Environment Federation) are 680 gal/day/ft<sup>2</sup> and 1300 gal/day/ft<sup>2</sup>, respectively. As can be seen from the previous calculations, the maximum SOR value is currently exceeded at a peak instantaneous flow of 20 MGD.

### ***Solids Loading Rate, Existing Clarifiers (future nutrient removal system)***

One of the options considered as part of the planning for this project was the ability of the existing final clarifiers to provide adequate treatment for the future nutrient removal process. Of critical concern is the solid loading rate that would result on the existing clarifiers when the activated sludge process is placed on-line. The following analysis examines the solids loading rate on the existing final clarifiers if the clarifiers were to remain in service:

Assumptions:

4 of 5 final clarifiers on-line

MLSS to final clarifiers of 2500 mg/L from activated sludge process

Peak flow of 20 MGD (assume no RAS) = 5 MGD per tank

Calculated solids loading would be:

$$(5 \text{ MGD} * 2500 \text{ mg/L} * 8.34) / 2035 \text{ ft}^2 = 51.2 \text{ lbs/day/ft}^2$$

Per the WEF Manual of Practice (Chapter 14), the peak loading rates for final clarifiers should range from 40-50 lb/day/ft<sup>2</sup>. In addition to exceeding the recommended surface overflow rate and solids loading rate, the existing final clarifiers are not of the proper configuration to provide proper settling for activated sludge solids. The tanks are short, rectangular, and shallow, and offer no method for proper flocculation of the solids, which is critical for final clarifiers to achieve proper solids settling. The existing clarifiers already exhibit short circuiting of solids during periods of higher flows.

The existing final clarifiers are proposed to be decommissioned as part of this project, with the new circular clarifiers replacing them. The mechanisms will be stripped from these tanks, and the tanks will be used instead as effluent water storage tanks that will continue to feed existing effluent water systems at the plant.

## Solids Handling – Existing Sludge Thickener and Digesters

### Description

The plant currently has one existing sludge thickener for thickening sludge prior to the sludge being conveyed to the digesters. The sludge thickener accepts sludge from the primary and secondary clarifiers. It is anticipated that the amount of sludge sent to the sludge thickener will not change. Although new secondary clarifiers are being added, the old secondary clarifiers are being removed from the process and the loadings are not changing, therefore the demand on the sludge thickener should not change. No alterations are proposed to the existing sludge thickener as part of this project.

The plant has two anaerobic digesters (primary and secondary) that are used to stabilize and break down sludge prior to dewatering. The digesters are original to the plant and were constructed in 1957, but have seen recent upgrades. The secondary digester had its membrane cover (Dystor) replaced this year, and many piping upgrades have been performed on the digesters to replace worn out piping and valves. Additionally, vertical draft tube mixers have been added to the digesters this year that provide for much more efficient mixing and heating of the sludge.

Solids handling facilities are generally evaluated on the basis the period of time when the maximum solids generation is expected. At the Bethel Park WWTP, the period of maximum solids production occurs during the seasonal “unloading” of the trickling filters. When the seasons change in the spring and fall, the change in ambient temperature causes a rapid loss of the attached biomass that can overwhelm the solids handling treatment processes at the plant. The loss of biomass can be sustained for a period of two to four weeks depending on the weather conditions. Therefore, the balance of the analysis will be based upon worst case conditions characterized as time period of trickling filter “unloading”.

Based upon the current influent flow of 4.1 MGD, the current influent pollutant loadings of 3,800 lbs/day of CBOD / 3,600 lbs/day of TSS, and seasonal trickling filter “unloading” conditions, then the following is the calculated current generation of biosolids from the treatment process under worst case type conditions.

	Current
Annual Average Flow, MGD	4.1
Primary Sludge, lbs/day	2,400
Secondary Sludge w/Unloading, lbs/day	3,900
Total Sludge, lbs/day	6,300
Digestion, lbs/day	-1,950
Total for Disposal, lbs/day	4,350

Current Gravity Thickener/Digester Performance

***Current Gravity Thickener Performance***

Parameter	Value	Typical Range
Solids Loading, lbs/sq ft day	6.1	12 to 20
Overflow Rate, gpd/sq ft	155	150 to 300
Side Water Depth, ft	9.75	10 to 13

***Current Anaerobic Primary Digester Performance***

Parameter	Value	Typical Range
Solids Loading, lbs VSS/1000 cu ft day	55	< 40 for Poor Mixing, < 80 for Good Mixing
Solids Retention Time, day	12	15 to 20

Note: Typical range provide is for mesophilic, high-rate, complete-mix anaerobic digester.

## Disinfection and Outfall – Existing Chlorine/Dechlorination System

### Description

The BPMA currently uses gaseous chlorine for disinfection followed by sulfur dioxide for reduction of the chlorine residual. There are two existing chlorine contact tanks that are approximately 64-feet long by 32-feet wide by 10.5-feet deep. The plant's current NPDES permit for chlorine residual is 0.53 mg/L (instantaneous max), therefore dechlorination via sulfur dioxide is required to lower the chlorine residual to the permitted level.

As part of this project it is being proposed to no longer use the chlorine contact tanks and sulfur dioxide system for disinfection. Chlorine is dangerous to handle and is a safety concern for employees. Additionally, it is expected that the chlorine residual requirement will be lowered on future permits, therefore the use of chlorine and dechlorination will become more challenging and expensive for the Authority. For these reasons, the Authority plans to switch to an ultraviolet (UV) light system for disinfection.

The chlorine contact tanks are proposed to remain and be used as effluent water storage tanks. Effluent from the new UV system will be pumped up to the chlorine contact tanks, where the existing effluent water pumps reside. The existing effluent water pumps will then be used to supply effluent water to the plant as the pumps do currently. This allows the Authority to minimize the upgrade requirements for the plant's effluent water system and use as much of the existing infrastructure as possible.

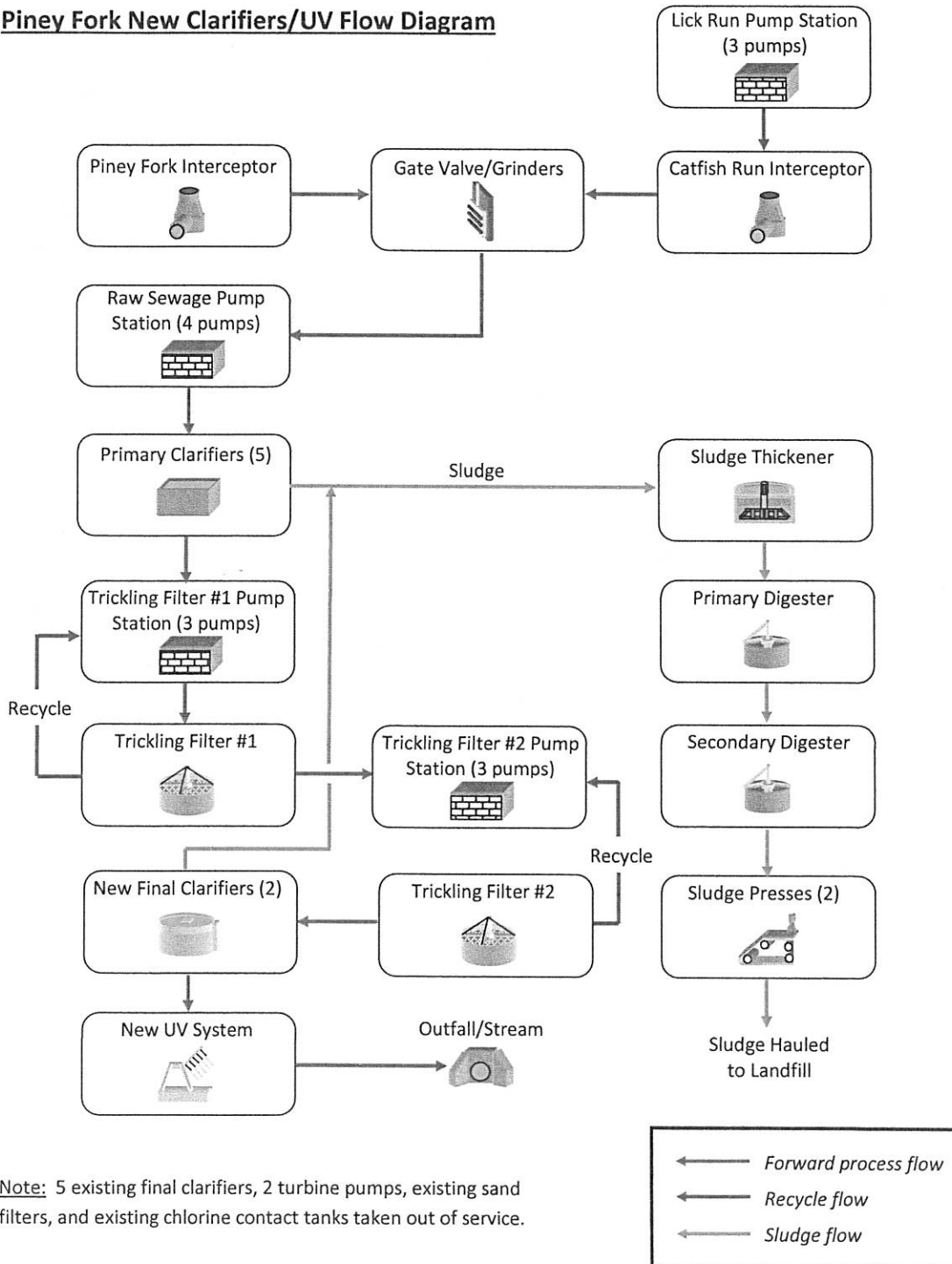
In its current flow configuration, effluent from the chlorine contact tanks is conveyed to Piney Fork through the current outfall just downstream of the chlorine contact tanks. The existing outfall is proposed to be decommissioned as part of this project, and a new outfall constructed downstream at the location of the proposed UV facilities.

# Part III – Proposed Facilities

## Proposed Flow Schematic and Hydraulic Profile

The flow schematic for the proposed operation of the Piney Fork treatment plant as well as the hydraulic profile diagram for the upgraded plant are provided as follows:

**Piney Fork New Clarifiers/UV Flow Diagram**



W.L. EL. 917.50  
WEIR EL. 917.25

18" PRIMARY  
CLARIFIER  
INFLUENT  
& EL. 917.00



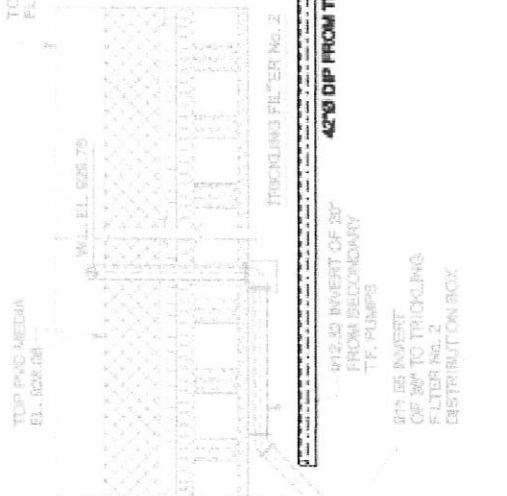
WET DISTRIBUTION BOX

42" DIA. DIP (INV. EL. 907.26)  
TO FINAL CLARIFIER INFLUENT  
DISTRIBUTION BOX

TRICKLING FILTER NO. 2  
DISTRIBUTION BOX

W.L. EL. 915.94  
TOP OF WEIR EL. 915.00  
30" FROM TRICKLING  
FILTER NO. 2 @ EL. 906.75

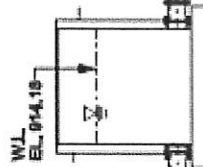
TOP STEEL TANK  
EL. 903.50



24" TO TRICKLING FILTER NO. 2  
20" PUMP SLUICING @ EL. 905.75

SECONDARY TRICKLING  
FILTER PUMPS

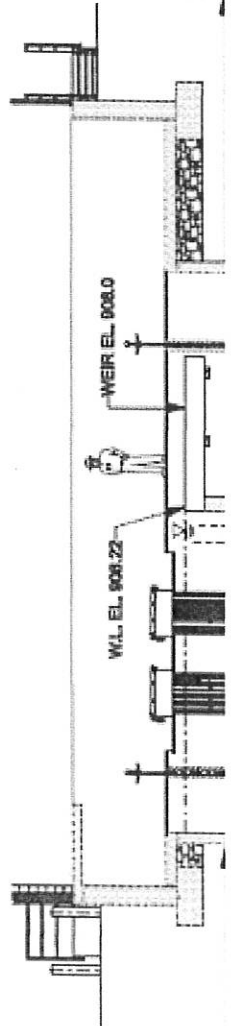
SECONDARY  
WET WELL



TO FINAL  
CLARIFIERS  
(BELOW LEFT)

42" DIA. DIP FROM TF DISTRIBUTION BOX

FINAL CLARIFIERS  
INFLUENT  
DISTRIBUTION BOX



W.L. EL. 908.43  
WEIR EL. 905.0

W.L. EL. 904.31  
INV. EL. 903.0

W.L. EL. 903.31  
INV. EL. 902.0

## Design of Future Nutrient Removal System

### Nitrogen Removal Design Calculations

Following example for Separate Stage Suspended Growth Denitrification Example in Chapter 7 of WEF Publication - Biological and Chemical Systems for Nutrient Removal, 1998

### Design Assumptions

	Current (MGD)	Design (MGD)
Permitted Capacity AADF	4.92	4.92
Annual Average Daily Flow	3.79	
Average Dry Weather Flow		2.5
Maximum Month Ave Daily Flow	5.97	5
Maximum Day Ave Daily Flow	15.7	20
Instantaneous Peak Flow		22

### Pollutant Design Conditions

		Influent	Nitrified Effluent	Final Effluent
Carbonaceous Oxygen Demand (CBOD)	(mg/L)	110	5	< 5
Total Suspended Solids (TSS)	(mg/L)	104	75	10
Volatile Suspended Solids (VSS)	(mg/L)		20	
Total Kjeldahl Nitrogen (TKN)	(as N mg/L)	25	1.5	1.25
Ammonia (NH <sub>3</sub> -N)	(as N mg/L)	20	0.5	0.25
Biodegradable Organic N	(as N mg/L)	4	0	0
Non-Biodegradable Organic N	(as N mg/L)	1	1	1
Nitrate/Nitrite (NO <sub>3</sub> -N (plus NO <sub>2</sub> -N))	(as N mg/L)	0	5	1
Total Nitrogen (TN)	(as N mg/L)	25	6.5	< 3 to 5

## Design Parameters

Yield Coefficient	Y	0.18	(g VSS/g COD)
Decay Coefficient	b	0.04	(days <sup>-1</sup> )
Half-rate Constant	K <sub>s</sub>	9.1	(g COD/m <sup>3</sup> )
Maximum Substrate Removal Rate at 20°C	q <sub>max</sub>	10.3	(g COD/g VSS d)
Temperature Correction Coefficient	Θ	1.12	
TKN:VSS Ratio in Solids		0.12	(g N/g VSS)
COD:Methanol Weight Ratio		1.5	
Lowest Reactor Temperature		55	(°F)
Lowest Reactor Temperature		12.5	(°C)
Biological Design Safety Factor (Assumed)		1.1	
Max Month N Loading to Ave N Loading Peaking Factor*		1.5	
Max Day N Loading to Ave N Loading Peaking Factor*		2.3	
Nitrified Effluent Dissolved Oxygen Concentration		12	(mg DO/L)

\* Per Data Review

### Step 1. Determine Operational Design Criteria

Design Mixed Liquid Suspended Solids (MLSS) = 2500 mg TSS/L

VSS:TSS Ratio = 0.75 g VSS/g TSS

Design Mixed Liquid Volatile Suspended Solids (MLVSS) = 1875 mg VSS/L

### Step 2. Determine Maximum Nitrate in Final Effluent to Meet Total Nitrogen Requirement

Effluent Particulate N = [Eff TSS x (VSS:TSS Ratio) x (TKN:VSS Ratio for Solids)]

Effluent TSS = 10 mg TSS/L

VSS:TSS Ratio = 0.75 g VSS/g TSS

TKN:VSS Ratio in Solids = 0.12 g N/g VSS

**Effluent Particulate N = 0.9 as N mg/L**

Effluent TKN = [Eff Particulate N + Eff Nonbiodegradable Organic N + Eff Ammonia]

Effluent Particulate N = 0.9 as N mg/L

Nonbiodegradable N = 1 as N mg/L

Effluent Ammonia = 0.25 as N mg/L

**Effluent TKN = 2.15 as N mg/L**

Allowable Nitrate in Effluent = [Design Eff Total N - Eff TKN]

Design Effluent Total N = 5 as N mg/L

Effluent TKN = 2.15 as N mg/L

**Effluent Nitrate = 2.85 as N mg/L**

Step 3. Determine Methanol Concentration in Reactor

Reactor Methanol Conc = [Eff COD / COD:MeOH WT Ratio]

Use a design Eff COD conc = 5 mg/L , so Eff CBOD5 conc  $\approx$  3.0 mg/L

Effluent COD Conc (Se) = 5 mg COD/L

COD:MeOH WT Ratio = 1.5

**Reactor MeOH = 3.33 mg MeOH/L**

Step 4. Calculate Organic Substrate Removal Rate at 20°C (q<sub>20</sub>)

$$q = q_{\max} \left( \frac{S_e}{K_s + S_e} \right)$$

Assume Methanol is Rate Limiting

Maximum Substrate Removal Rate (q<sub>max</sub>) = 10.3 g COD/g VSS d

Half-Rate Constant (K<sub>s</sub>) = 9.1 g COD/m<sup>3</sup>

Eff COD Con (Se) = 5.00 mg COD/L

**Organic Substrate Removal Rate at 20°C (q<sub>20</sub>) = 3.65 g COD/g VSS d**

Step 5. Calculate Substrate Removal Rate at Lowest Temperature (q<sub>T</sub>)

$$q_T = q_{20} \theta^{t-20}$$

Lowest Reator Temperature = 12.5 °C

Organic Substrate Removal Rate at 20°C (q<sub>20</sub>) = 3.65 g COD/g VSS d

Temperature Correction Coefficient (θ) = 1.12

**Organic Substrate Removal Rate at Lowest Temp (q<sub>T</sub>) = 1.56 g COD/g VSS d**

Step 6. Calculated Reactor Minimum Solids Retention Time (SRT<sub>min</sub>)

$$\frac{1}{SRT_{min}} = Y * q_t - b$$

Yield Coefficient (Y) = 0.18 g VSS/g COD

Organic Substrate Removal Rate at Lowest Temp (q<sub>T</sub>) = 1.56 g COD/g VSS d

Decay Coefficient (b) = 0.04 days<sup>-1</sup>

**Minimum Solids Retention Time (SRT<sub>min</sub>) = 4.15 days**

Step 7. Calculate Design Solids Retention Time (SRT<sub>design</sub>)

$$SRT_{design} = SRT_{min} \times Safety\ Factor \times Peaking\ Factor$$

Minimum Solids Retention Time (SRT<sub>min</sub>) = 4.15 days

Safety Factor = 1.1

Peaking Factor = 1.5

**Design Solids Retention Time (SRT<sub>design</sub>) = 6.85 days**

Step 8. Calculate Methanol Dose

Nitrate Removal = Influent Nitrate - Effluent Nitrate

*Annual Average Conditions*

Annual Ave Inf Nitrate = 5 as N mg/L

Eff Nitrate = 2.85 as N mg/L

**Annual Ave Nitrate Removal = 2.15 as N mg/L**

*Maximum Day Conditions*

Max Day to Ave Day Nitrate Peaking Factor = 2.3

Max Day Inf Nitrate = 11.5 as N mg/L

Eff Nitrate = 2.85 as N mg/L

**Max Day Nitrate Removal = 8.65 as N mg/L**

Methanol Dose = (2.47\*Nitrate Removal) + (0.87\*DO) + Design Reactor Conc

*Annual Average Conditions*

Annual Ave Nitrate Removal = 2.15 as N mg/L

Nitrified Effluent Dissolved Oxygen Conc = 12 mg DO/L

Design Reactor MeOH Conc = 3.33 mg MeOH/L

**Methanol Dose (as MeOH) = 19.08 mg MeOH/L**

Methanol Dose (as COD) = 28.63 mg COD/L

*Maximum Day Conditions*

Max Day Nitrate Removal = 8.65 as N mg/L

Nitrified Effluent Dissolved Oxygen Conc = 12 mg DO/L

Design Reactor MeOH Conc = 3.33 mg MeOH/L

**Methanol Dose (as MeOH) = 35.14 mg MeOH/L**

Methanol Dose (as COD) = 52.71 mg COD/L

Step 9. Calculated Anoxic Hydraulic Residence Time (HRT)

$$HRT_{anoxic} = \frac{V}{Q} = \frac{Y(S_0 - S_e)SRT_{design}}{(1 + b * SRT_{design})MLVSS} + \frac{(InfVSS)SRT_{design}}{MLVSS}$$

Yield Coefficient (Y) = 0.18 g VSS/g COD

Influent COD Conc (S<sub>0</sub>) = 52.71 mg COD/L

Effluent COD Conc (S<sub>e</sub>) = 5 mg COD/L

Decay Coefficient (b) = 0.04 days<sup>-1</sup>

Mixed Liquor Volatile Suspended Solids Conc (MLVSS) = 1875 mg VSS/L

Nitrified Effluent TSS Conc = 75 mg TSS/L

Nitrified Effluent VSS Conc = 20 mg VSS/L

Design Solids Retention Time (SRT<sub>design</sub>) = 6.85 days

First Term = 0.0246146

Second Term = 0.073025295

Anoxic Hydraulic Residence Time (HRT<sub>anoxic</sub>) = 0.0976 days

**Anoxic Hydraulic Residence Time (HRT<sub>anoxic</sub>) = 2.3 Hrs**

Step 10. Select Aerobic/Postaeration Hydraulic Residence Time (HRT)

**Assume Aerobic HRT = 0.75 hrs**

Step 11. Calculate Total Reactor Volume

Total HRT = HRT<sub>anoxic</sub> + HRT<sub>aerobic</sub>

Anoxic Hydraulic Residence Time (HRT<sub>anoxic</sub>) = 2.3 Hrs

Aerobic Hydraulic Residence Time (HRT<sub>aerobic</sub>) = 0.75 Hrs

Total Hydraulic Residence Time (HRT<sub>total</sub>) = 3.1 Hrs

Total Reactor Volume (V) = HRT<sub>total</sub> \* Max Month Design Flow

Total Hydraulic Residence Time (HRT<sub>total</sub>) = 3.1 Hrs

Max Month Design Flow = 5 MGD

Reactor Volume (V) = 644,449 gal

Reactor Volume (V) = 86,156 CF

*Assumptions:*

Number of Trains = 4, Side Water Depth = 10, Length to Width Ratio = 4

**Width (each) = 23.2 ft**

**Length (each) = 92.8 ft**

Based on the sizing requirements determined for the future nutrient removal tanks, adequate space has been allocated for those tanks to be installed at a future time when nutrient removal requirements are imposed. The drawing set provided with the Part II Permit submission shows a layout of these future tanks with the anticipated area for installation. Additionally, future piping connections have been provided that will allow flow to be taken from the existing trickling filters, through the new nutrient removal facilities, and then to the proposed final clarifiers when the nutrient removal facilities are installed.

## Phosphorus Removal Design Calculations

Based on the process selected for advanced total nitrogen removal, it is anticipated that chemical precipitation of phosphorus will be required to achieve anticipated future phosphorus limits. In order to determine the size of the chemical facilities/equipment that will be required to provide the necessary phosphorus reduction, dosage equations are presented below. It is assumed that ferric chloride will be the iron salt used for precipitation of phosphorus, although it is possible that alum (aluminum sulfate) may be used. For that reason, the chemical storage feed quantities and storage requirements for both ferric chloride and alum are presented as follows.

### Plant Data

Anticipated feed point = Final clarifier influent  
Peak average flow rate = 5.5 MGD

### **Ferric Chloride Calculations**

#### Assumptions

Average Daily Flow (MGD)	4.1 MGD (current)	5.5 MGD (future)
Chemical to be used:	Ferric Chloride (FeCl <sub>3</sub> )	
Chemical Concentration:	30%	
Specific Gravity:	1.36	
Density:	11.2 lbs/gal	
Molecular Weight:	162.3 g/mole	
Iron/Phosphorus Ratio:	5	

(Taken from Fig 343 in Biological and Chemical Systems for Nutrient Removal, WEF, 1998)

Total Phosphorus Concentration after Secondary Treatment: 3 mg/L

Ferric Dose: Fe/P Ratio x mg/L P = 15 mg/L as Fe

Mass of Ferric to be Added:  $8.34 \times \text{Fe Dose (mg/L)} \times Q \text{ (MGD)} = \underline{688.1 \text{ lbs/day}}$  as Fe @ Future ADF

Weight % Fe in Dry Ferric Chloride:  $\text{Weight Fe} / \text{Weight Ferric Chlorine} = (55.85 * 100) / 162.3 = \underline{34.4\%}$

Mass of Ferric Chloride to be Added:  $\text{Fe Added} / \% \text{ Fe in Ferric Chloride} = 688.1 / 0.344 = \underline{2,000 \text{ lbs/day Ferric}}$

Ferric Chloride Feed Rate:  $\text{Mass of Ferric} / (\% \text{ Conc} \times \text{Density}) = \underline{595 \text{ gal/day of 30\% Ferric}}$

Ferric Storage Volume for 30 Days On-Site Storage:  $\text{Ferric Feed Rate} \times 30 = \boxed{17,858 \text{ gallons}}$

### Alum Calculations

#### Assumptions

Average Daily Flow (MGD)	4.1 MGD (current)	5.5 MGD (future)
Chemical to be used:	Aluminum Sulfate (Alum, $\text{Al}_2(\text{SO}_4)_3 \cdot 14 \text{ H}_2\text{O}$ )	
Chemical Concentration:	50%	
Specific Gravity:	1.33	
Density:	11.1 lbs/gal	
Molecular Weight:	594.3 g/mole	
Iron/Phosphorus Ratio:	2	

(Taken from Fig 3.3 in Biological and Chemical Systems for Nutrient Removal, WEF, 1998)

Total Phosphorus Concentration after Secondary Treatment: 3 mg/L

Aluminum Dose:  $\text{Al/P Ratio} \times \text{mg/L P} = 6 \text{ mg/L as Fe}$

Mass of Alum to be Added:  $8.34 \times \text{Al Dose (mg/L)} \times \text{Q (MGD)} = \underline{275.2 \text{ lbs/day}}$  as Al @ Future ADF

Weight % Fe in Dry Alum:  $\text{Weight Al} / \text{Weight Alum} = (2 * 27 * 100) / 594.3 = \underline{9.08\%}$

Mass of Alum to be Added:  $\text{Al Added} / \% \text{ Al in Alum} = 275.2 / 0.0908 = \underline{3,031 \text{ lbs/day Alum}}$

Alum Feed Rate:  $\text{Mass of Alum} / (\% \text{ Conc} \times \text{Density}) = \underline{546 \text{ gal/day of 50\% Alum}}$

Alum Storage Volume for 30 Days On-Site Storage:  $\text{Alum Feed Rate} \times 30 = \boxed{16,384 \text{ gallons}}$

The UV bldg. has been designed to facilitate the installation of chemical feed equipment capable of providing the required dosage for phosphorus removal as well as the necessary space for 30 days of chemical storage.

## **Design of New Final Clarifiers**

As was mentioned in the previous section, the existing final clarifiers are not well designed for the removal of solids, especially at higher flows. With the upcoming nutrient removal requirements that are anticipated to be imposed in the not too distant future, the existing final clarifiers will not have enough capacity to remove the additional solids generated from the selected nutrient removal process. A more conventional rapid removal circular clarifier design will be required to provide the necessary settling and solids removal capacity when nutrient removal requirements are imposed. Based on the amount of work and cost to install the anticipated nutrient removal facilities, final clarifiers, and any other facilities that may be required, the Authority has decided to phase the construction and install the final clarifiers as part of this initial phase of construction.

### Proposed Facilities

#### ***Clarifier layout***

For the design, two (2) new 100-foot circular final clarifiers are proposed to remove solids from the existing trickling filters and from the future nutrient removal process that will be constructed when nutrient limits are imposed.

The clarifier floors will be sloped at  $\frac{1}{4}$ " per foot. Each clarifier will have an access bridge spanning half of the diameter of the clarifier, and will be supported by a center pier. A square platform will be located at the center of each clarifier to allow for maintenance to each clarifier drive.

The clarifier launders will be two foot wide and run the entire perimeter of the clarifier, with v-notch weirs attached to the launders to provide effluent control. Additionally, the launders will be covered with a fiberglass reinforced plastic cover to control algae growth, which is very common in final clarifiers. The covers will be of the lift up type that will allow operators to access the launders/weirs and provide the necessary maintenance, inspection, and cleaning.

Two sludge pipes will be installed as part of this project for each clarifier. An 8-inch sludge withdrawal pipe will be installed that will allow the new waste sludge pumps to remove sludge from the final clarifiers and be pumped to the existing sludge thickener. In addition, a 16-inch future return activated sludge (RAS) line will be installed for the future nutrient removal process. This line will run from the center well of the clarifier to the outside wall of the clarifier and be capped on both ends. Installing this line now will prevent part of the floor of the clarifier from having to be removed to install a future RAS line when nutrient removal requirements are imposed.

#### ***Clarifier mechanism/components***

The clarifier mechanism will be a rapid sludge removal header mechanism (Tow-Bro type mechanism). Plant flow will enter from the final clarifier influent distribution box to the center well located beneath the support pier. Flow exits the clarifier over weirs located at the launder effluent channel. Scum will be removed from the surface of the water via two scum arms and

will be collected into a scum beach. A density current baffle will be installed to facilitate scum removal, and launder covers will be provided to prevent algae growth. Scum will be piped from the 6-foot long scum beach to a scum box located adjacent to the final clarifier effluent box. Pumps for waste sludge are located at the UV building lower floor (reference below for sizing of waste sludge/scum pumps). The sludge pumps will pump the clarifier sludge/scum to the existing sludge thickener.

The clarifier mechanism will be equipped with a 24'-6" diameter stilling well. Flow will enter the stilling well via a 36" center pier. The stilling well is also equipped with FEDWA baffles, which are rectangular baffles provided around the perimeter of the stilling well center pier. The baffles are positioned and sized to promote flocculation, which in turn improves settleability.

Effluent flow will be collected via effluent weirs and troughs located at the perimeter of the clarifier. The weirs will be 90-degree v-notch weirs constructed of steel plate material. The launder troughs will be concrete and will be 2-feet in width. Flow will be conveyed over the effluent weirs into the launders, where it will then flow to the clarifier effluent drop box. From here flow is conveyed to the final clarifier effluent box.

***Clarifier Design Criteria – Design Average Flow (4.92 MGD)***

Secondary Clarifiers No. 1 – 2, 100 foot diameter, 17 foot SWD  
Proposed Clarifiers On-Line = 1  
Surface area (each) = 7854 ft<sup>2</sup>  
Volume (each) = 998,715 gal  
Detention time = 4.87 hours  
SOR @ 4.92 MGD = 626 gal/day/ft<sup>2</sup>

***Clarifier Design Criteria – Peak Instantaneous Flow (20 MGD)***

Secondary Clarifiers No. 1 – 2, 100 foot diameter, 17 foot SWD  
Proposed Clarifiers On-Line = 2  
Surface area (each) = 7854 ft<sup>2</sup>  
Volume (each) = 998,715 gal  
Detention time = 2.4 hours  
SOR @ 10 MGD = 1273 gal/day/ft<sup>2</sup>

## **Design of New Disinfection/UV System**

As part of this project, it is being proposed that a new UV disinfection system be installed to replace the existing chlorine/dechlorination process. The new UV system will be safer to operate, does not require chemicals in addition to chlorine for dechlorination, and will increase the disinfection capacity at the plant. As permit regulations become more stringent for removal of pathogens, it is imperative that the plant install a process that will meet those requirements.

The Authority is designing its UV system around the Ozonia Aquaray UV system. The following paragraphs provide a description of the components and well as design considerations evaluated for the basis of design.

### UV lamp module arrangement

The lamp modules are arranged with thirty-six (36) high output, low pressure amalgam lamps vertically mounted in a uniform staggered array of six rows of six lamps each. Each lamp in the system is guaranteed for 12,000 hours of operation. At the end of its lamp life, the UV lamps will have a minimum of 90% its original output. The lamps are housed in quartz jackets, which have dome-shaped test tube ends supported by neoprene holders. The top end of the quartz jacket is open and terminates inside at the top of the enclosure. It is sealed using a neoprene compression gland.

The UV module support structure includes a top enclosure, four support legs and a bottom support pan, which are all manufactured from 316 stainless steel. All electrical connections are out of the water and within the protection of the NEMA 4X Power Center Panels. One UV intensity monitor is provided per bank of modules which continuously monitors and stores the UV intensity data and the status of each lamp. Data collected includes the number of hours in operation of each lamp, the number of on and off cycles, and the date it was installed. This information is passed on to the control system and held in memory for subsequent recovery and integration.

### Energy conservation and flow pacing

The UV System conserves energy via a combination of turning on/off and dimming of UV lamps. The UV system is able to switch UV lamps on or off and vary between lamp output (dimming lamps from 100% to 60%) in response to the flow pattern of the wastewater treatment plant through a 4-20mA signal received from the plant flow meter. This feature minimizes power consumption of the system and extends lamp life by ensuring that only the minimum number of lamps are used to treat a certain flow at a preset UV dose.

### Level control

The UV System has to react to a wide variety of flows, from minimum to peak wet weather.

Regardless of this wide flow variation, a constant water level has to be maintained in the disinfection channels. For this project, fixed straight edge weirs will be used to insure a constant water level in the UV channels.

#### Quartz jacket fouling, reduction, and removal

In any UV system the primary cause of fouling is the deposition of minerals under the influence of heat from the ultraviolet lamp. Calcium bicarbonate forms a precipitate that is deposited as scale on the outside of the lamp's quartz jacket. The rate of deposition is a site-specific and is temperature dependent: the higher the temperature, the greater the deposition. For low-pressure high output amalgam lamps, the wasted energy given off as heat is minimal, and therefore fouling is significantly lessened. Furthermore, the UV system is equipped with an automatic mechanical wiping system to remove the deposits on the quartz jacket. Under normal operating conditions, it is recommended to utilize the manufacturer supplied in-channel cleaning system two wipes per day (default factory setting). The automatic wiping system is driven by an electrical motor located in each module and has been field tested and proven to maintain a fouling factor in excess of 95%.

#### Design basis

Peak Flow - 25 MGD

Minimum Flow - 2 MGD

UV Transmittance - 65% (assumed)

Influent SS, mg/L - less than 30 mg/l

Influent, MPN/100ml:

- Fecal Coliform - 200,000 (Assumed)

Required Effluent, MPN/100ml:

- Fecal Coliform - less than 200 MPN (30 day geometric mean)

Minimum UV Dose - 30,000  $\mu$ watt-secs/cm<sup>2</sup>

Number of Channels - 2

Number of Modules Across - 1

Number of Modules in Series - 2

Channel Width - 29.5 inches

Channel Length - 19 feet

Channel Depth - 84 inches

Water Depth - Range 62-69 inches

Total Number of Modules - 4

Number of Lamps/Module - 36

Total Number of Lamps - 144

End of Lamp Life factor, Fp - 0.90

Fouling factor, Ft - 0.95

Headloss at Peak Flow - 4.33 Inches @ 25 MGD

Power Consumption per Lamp - 406 watts

Power Consumption at Peak Flow - 58.4 kW @ 25 MGD

#### Power Consumption at Low Flow - 10.4 kW @ 2 MGD

Based on the information summarized above, the system proposed will provide a minimum UV dosage of 30,000 uWatts-secs/cm<sup>2</sup> with all lamps in service. The dosage calculation takes into account several factors including the end of lamp life and the quartz sleeves transmittance factor at the peak capacity. Based on a Peak Flow of 25 MGD and UV transmittance of 65%, two (2) UV disinfection channels are proposed, each with two (2) UV modules mounted one (1) across by two (2) banks in series. The total number of modules is four (4) each with 36 low pressure high output amalgam lamps.

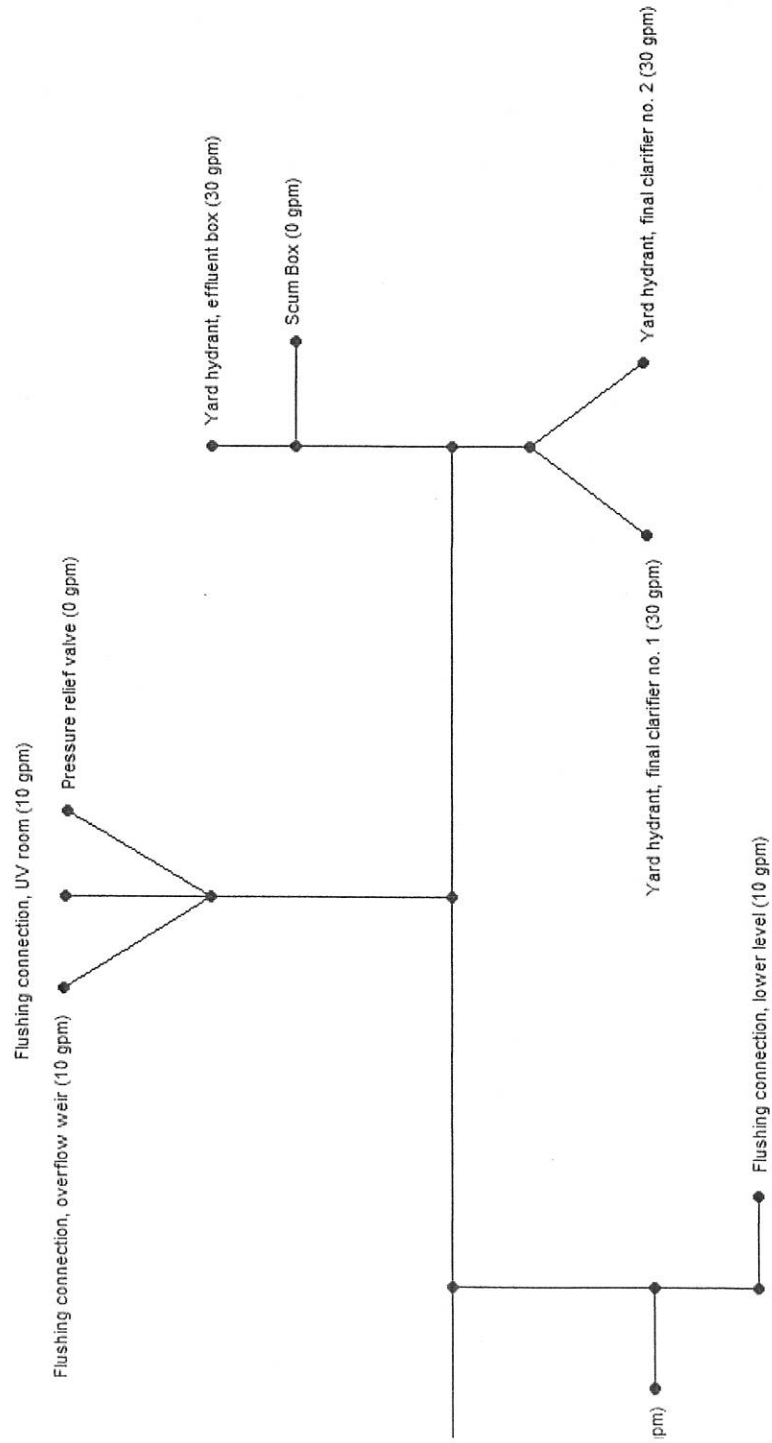
## **Design of New Effluent Water System/Pumps**

Part of this project involves the design and installation of new effluent water pumps and effluent lines that will run to the proposed facilities. Because the existing chlorine contact tanks are no longer to be used for disinfection, they will be turned into effluent water storage tanks. The existing effluent water pumps currently pull flow from the chlorine contact tanks, so using these tanks for effluent water will allow those existing pumps to provide effluent water in the plant's current configuration.

The new effluent water pumps will be installed on the intermediate level of the new UV building and will draw water from the effluent channels downstream of the UV system. This will allow the water to receive full disinfection prior to being distributed to the plant for general use. In addition to supplying effluent water to the chlorine contact tanks that are being converted to effluent water storage tanks, the new effluent water pumps will also provide effluent flow to hose bibs located within the UV building as well as hydrants located within the new project area. The new hydrants will be located around the final clarifiers and at the new sludge/scum box strategically so that the clarifiers and sludge/scum box can be cleaned on a regular basis. EPANET was used to determine the pressures in the new effluent water system and size the pumps to provide enough flow and pressure at its discharge locations. The effluent water system schematic is provided on Drawing M-17.

Based on the demand on the effluent water system and pressures desired at the various nodes of discharge, the pump size that is required will be 350 gpm at 150' TDH. A schematic of the effluent water system and pump curve for the selected pumps is provided on the following pages.

Three effluent water pumps are being provided. Two pumps will be required during the maximum draw on the effluent water system, and one pump will be available for backup. It is anticipated that only one pump will be required for the majority of plant operations, and the second pump would most likely be required during the cleaning of the equalization tank, which creates the largest draw on the effluent water system. Pressures of approximately 50 psi will be maintained at all discharge points within the water system during the period of maximum draw. Pressures as high as 80 psi will exist in the system when the use of the system is minimal.



**EPANET calculations of effluent water system**

Page 1

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*****
*                               E P A N E T                               *
*                               Hydraulic and Water Quality                 *
*                               Analysis for Pipe Networks                   *
*                               Version 2.0                                 *
*****
  
```

Link - Node Table:

Link ID	Start Node	End Node	Length ft	Diameter in
3	2	4	15	6
4	4	5	1000	12
5	4	6	14	3
6	6	7	5	1.5
7	6	8	13	3
8	5	9	50	3
9	9	10	30	1.5
10	9	11	16	1.5
11	9	12	5	1.5
12	5	13	70	12
17	13	21	20	12
18	21	17	5	1.5
19	21	20	5	3
20	13	22	100	3
21	22	14	40	1.5
22	22	15	40	1.5
23	8	23	12	1.5
27	29	28	80	6
1	1	2	500	6
15	28	2	#N/A	#N/A Pump

Energy Usage:

Pump	Usage Factor	Avg. Effic.	Kw-hr /Mgal	Avg. Kw	Peak Kw	Cost /day
15	100.00	75.00	590.63	13.47	13.47	0.00
Demand Charge:						0.00
Total Cost:						0.00

## Node Results:

Node ID	Demand GPM	Head ft	Pressure psi	Quality
2	0.00	1048.39	60.11	0.00
4	0.00	1048.35	59.51	0.00
5	0.00	1048.28	59.48	0.00
6	0.00	1048.32	61.67	0.00
7	10.00	1048.21	63.14	0.00
8	0.00	1048.31	67.29	0.00
9	0.00	1048.16	57.70	0.00
10	10.00	1047.54	56.13	0.00
11	10.00	1047.83	57.99	0.00
12	0.00	1048.16	59.86	0.00
13	0.00	1048.28	59.48	0.00
14	30.00	1040.11	49.44	0.00
15	30.00	1040.11	49.44	0.00
17	30.00	1047.50	53.94	0.00
20	0.00	1048.28	59.48	0.00
21	0.00	1048.28	59.48	0.00
22	0.00	1046.35	58.65	0.00
23	10.00	1048.06	69.57	0.00
28	0.00	907.39	2.34	0.00
1	250.00	1043.77	57.96	0.00
29	-380.00	909.00	2.60	0.00 Tank

## Link Results:

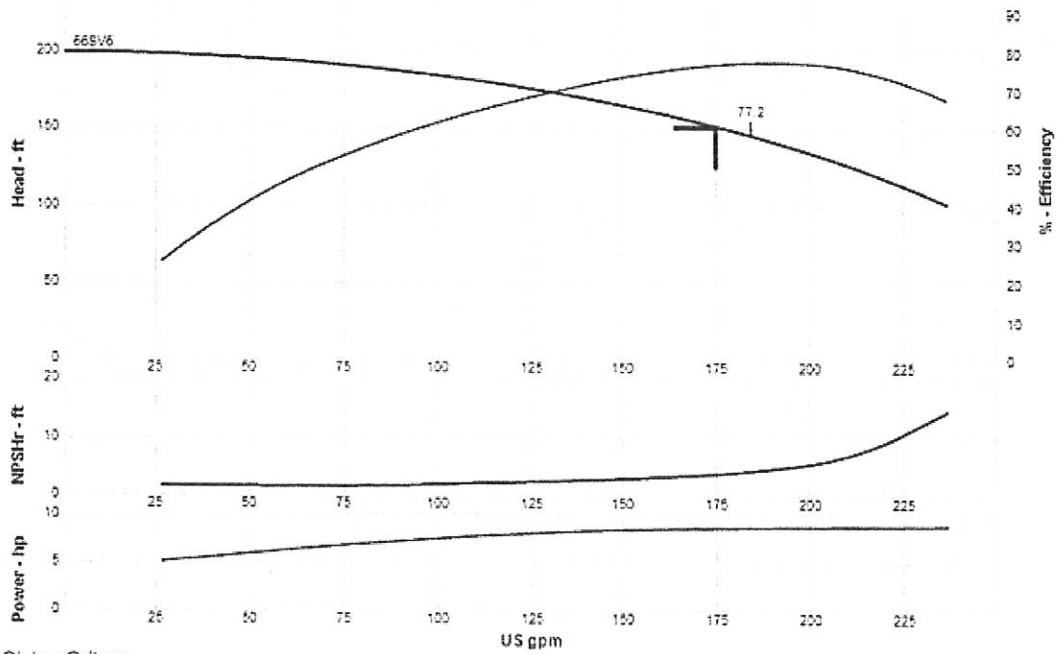
Link ID	Flow GPM	Velocity fps	Unit Headloss ft/Kft	Status
3	130.00	1.48	2.76	Open
4	110.00	0.31	0.07	Open
5	20.00	0.91	2.52	Open
6	10.00	1.82	20.41	Open
7	10.00	0.45	0.69	Open
8	20.00	0.91	2.52	Open
9	10.00	1.82	20.41	Open
10	10.00	1.82	20.41	Open
11	0.00	0.00	0.00	Open
12	90.00	0.26	0.05	Open
17	30.00	0.09	0.01	Open
18	30.00	5.45	156.15	Open
19	0.00	0.00	0.00	Open
20	60.00	2.72	19.26	Open
21	30.00	5.45	156.14	Open
22	30.00	5.45	156.14	Open
23	10.00	1.82	20.41	Open
27	380.00	4.31	20.09	Open
1	-250.00	2.84	9.25	Open
15	380.00	0.00	-141.00	Open Pump

**Effluent water system pump curve**



**PERFORMANCE CURVE**

Quote Number: 9001-141022-036  
 66SV60GJ8F64 - 10HP 4/60/3 TEPE 362 6STG



**Sizing Criteria**

Series:	E-SV	Maximum Pressure:	86 psi
Size:	66SV	Maximum Head:	199 ft
Number of Impellers:	6	Head at Design:	150 ft
Number of Trimmed Impellers:	0	Head on Performance Curve:	151 ft
Trimmed Impeller Size:	N/A	Best Efficiency:	77.2 %
Speed:	1800 RPM	Efficiency at Design:	76.4 %
Curve Type:	Standard NPSH	NPSHr:	4.0 ft
Curve ID:	66SV6	Power at Design:	8.72 Hp
Maximum Flow:	237 USGPM	NOL Power:	8.96 Hp
Flow at Design:	176 USGPM	Fluid Type:	WATER
Flow at BEP:	184 USGPM	Temperature:	70.0 °F

DO NOT USE FOR CONSTRUCTION UNLESS CERTIFIED	
Certified by	
Date of Certification	
Pump Serial Number	
Project Name	P-7004
Tag	

## Design of New Final Clarifier Sludge/Scum Pumps

Final clarifier waste sludge pumps will be installed on the lower level of the UV building in order to facilitate sludge and scum removal from the two final clarifiers. Sludge will be withdrawn via two 8" suction lines. These lines will pull from a sludge hopper located at the center of each clarifier. Scum is piped from each scum beach to a scum box that is located adjacent to the final clarifier effluent box. The scum box will also accept the sump pump flow from the UV building basement. The waste sludge pumps will also be utilized to empty the scum box as needed.

Two double disc positive displacement pumps will have the capability to send flow to the administration building basement where waste sludge can either be diverted to the thickener or to either digester, if required. Certain minor piping modifications will be done in the administration building basement as a part of this project to accomplish this task. Piping modifications will be required in the pipe tunnel as well to tie into existing rectangular secondary clarifier sludge removal piping.

After running the pumps through a pump sizing program, the performance requirements for the pumps will be 250 gpm at 30' TDH.

Below is a summary report for the sizing of the waste sludge pumps.

### Design Information:

Final Clarifier W.L.:	912.14 el.
Pump Centerline:	889.16 el.
Thickener W.L.:	914 el.
Digester Sludge Outlet:	924.16 el. (use digester, worst case scenerio)
Static Discharge:	35 el.
Static Suction:	22.98 el.
Flow:	250 gpm
Friction loss (Haz. Wil)	100 (for discharge, existing pipe)
Friction Loss (Haz. Wil)	110 (for suction)

### Final Clarifier No. 2 Sludge to Digester:

#### Suction (8"):

DIP Length:	207
22.5° Elbow:	1
45° Elbow:	2
90° Elbow:	1
90° Long Rad Elbow:	1
Tee (R-R):	1
Tee (R-B):	1
Wye:	1
Reducer (8->6):	1
Plug Valve:	3

#### Discharge (6"):

DIP Length:	1095
22.5° Elbow:	3
45° Elbow:	9
90° Elbow:	5
Tee (R-R):	8
Tee (R-B):	1
Tee (B-R):	3
Plug Valve:	7
Check Valve:	1
Exit:	1

Design Form	Results	Theory/Assumptions	Specialties														
<p>Data Provided</p> <p><b>Data Provided:</b></p> <ul style="list-style-type: none"> <li>Suction friction loss: 0.4 ft</li> <li>Suction fitting loss: 0.27 ft</li> <li>Max. suction velocity: 1.6 ft/s</li> <li>Max. vapor pressure: 0.59 ft</li> <li>Discharge friction loss: 10.21 ft</li> <li>Discharge fitting loss: 2.65 ft</li> <li>Discharge velocity: 2.84 ft/s</li> <li>Discharge pressure: 20.72 psi</li> </ul>	<p>Graph - Flow vs. Head</p> <table border="1" style="margin-top: 10px; width: 100%; border-collapse: collapse;"> <caption>Data points for Flow vs. Head graph</caption> <thead> <tr> <th>Flow (gpm)</th> <th>Total dynamic head (ft)</th> </tr> </thead> <tbody> <tr><td>83.3</td><td>15</td></tr> <tr><td>166.7</td><td>20</td></tr> <tr><td>250</td><td>25</td></tr> <tr><td>333.3</td><td>35</td></tr> <tr><td>416.7</td><td>45</td></tr> <tr><td>500</td><td>65</td></tr> </tbody> </table>			Flow (gpm)	Total dynamic head (ft)	83.3	15	166.7	20	250	25	333.3	35	416.7	45	500	65
Flow (gpm)	Total dynamic head (ft)																
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250	25																
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Results																	
<table border="1" style="width: 100%; border-collapse: collapse;"> <thead> <tr> <th style="width: 33%;"><u>NPSH available:</u></th> <th style="width: 33%;"><u>Total discharge head:</u></th> <th style="width: 33%;"><u>Total dynamic head</u></th> </tr> </thead> <tbody> <tr> <td style="text-align: center;">55.68 ft</td> <td style="text-align: center;">47.86 ft</td> <td style="text-align: center;">25.64 ft</td> </tr> </tbody> </table>				<u>NPSH available:</u>	<u>Total discharge head:</u>	<u>Total dynamic head</u>	55.68 ft	47.86 ft	25.64 ft								
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55.68 ft	47.86 ft	25.64 ft															
			<div style="border: 1px solid black; padding: 5px; display: inline-block;">           Calculation Summary Report         </div>														

## **Part IV – Project Schedule**

### ***Design/Construction Schedule***

The following is an updated design schedule through completion of this project.

<b>Milestone</b>	<b>Approximate Date</b>
Design NTP	January 23, 2014
Re-submission of Act 537 Special Study	November 2014
Part II Permit Submission	January 2015
Award Bids	June 2015
Complete Construction	June 2017

Not shown in the schedule are progress meetings with the Authority and presentations to the Authority Board. Review periods by PaDEP and Bethel Park Municipal Authority may impact the timeline.

**APPENDIX I – HYDRAULIC PROFILE CALCULATIONS REPORT**

# Visual Hydraulics Summary Report - Hydraulic Analysis



Project: Bethel Phase II WWTP Improvements hydraulics with effluent weir 9-17-

Company:

Date:

## Current flow conditions

<b>Forward Flow =</b>	22 mgd
<b>Return I Flow =</b>	4 mgd
<b>Return II Flow =</b>	-----
<b>Return III Flow =</b>	-----

## Section Description

## Water Surface Elevation

**Piney Fork water elevation**

900

**Effluent weir**

906.43

Weir invert (top of weir) = 905

Weir length = 6 ft

Weir 'C' coefficient = 3.33

Flow over weir = 22 mgd

Weir submergence = unsubmerged

Head over weir = 1.43 ft

**UV effluent channel**

906.43

Channel shape = Rectangular

Manning's 'n' = 0.014

Channel length = 50 ft

Channel width/diameter = 6 ft

Flow = 22 mgd

Downstream channel invert = 903

Channel slope = 0 ft/ft

Channel side slope = not applicable

Area of flow = 20.58 ft<sup>2</sup>

Hydraulic radius = 1.6

Normal depth = infinite

Critical depth = 1 ft

Depth downstream = 3.43 ft

Bend loss = 0 ft

Depth upstream = 3.43 ft

Velocity = 1.66 ft/s

Flow profile = Horizontal

**Section Description****Water Surface Elevation****Entrance between UV channel and UV effluent channel 2**

906.57

Pipe shape = Circular  
Diameter = 36 in  
Length = 2 ft  
Flow = 11 mgd  
Friction method = Hazen Williams  
Friction factor = 120  
Total fitting K value = 1.5  
Pipe area = 7.07 ft<sup>2</sup>  
Pipe hydraulic radius = 0.75  
Age factor = 1  
Solids factor = 1  
Velocity = 2.41 ft/s  
Friction loss = 0 ft  
Fitting loss = 0.13 ft  
Total loss = 0.14 ft

**Entrance between UV channel and UV effluent channel 1**

906.57

Pipe shape = Circular  
Diameter = 36 in  
Length = 2 ft  
Flow = 11 mgd  
Friction method = Hazen Williams  
Friction factor = 120  
Total fitting K value = 1.5  
Pipe area = 7.07 ft<sup>2</sup>  
Pipe hydraulic radius = 0.75  
Age factor = 1  
Solids factor = 1  
Velocity = 2.41 ft/s  
Friction loss = 0 ft  
Fitting loss = 0.13 ft  
Total loss = 0.14 ft

**UV effluent channel 1 weir**

908.22

Weir invert (top of weir) = 908  
Weir length = 48 ft  
Weir 'C' coefficient = 3.33  
Flow over weir = 11 mgd  
Weir submergence = unsubmerged  
Head over weir = 0.22 ft

**UV system in channel 1**

908.72

Change in elevation = 0.5 ft

**UV channel 1 influent gate**

908.78

Opening type = rectangular gate

**Section Description**

**Water Surface Elevation**

Opening diameter/width = 30 in  
Gate height = 72 in  
Invert = 903  
Number of gates = 1  
Flow through gate(s) = 11 mgd  
Total area of opening(s) = 15 ft<sup>2</sup>  
Velocity through gate(s) = 1.13 ft/s  
Flow behavior = orifice, downstream control  
Gate loss = 0.05 ft  
Downstream water level = 908.72  
Upstream water level = 908.78

**UV effluent channel 2 weir** **908.22**

Weir invert (top of weir) = 908  
Weir length = 48 ft  
Weir 'C' coefficient = 3.33  
Flow over weir = 11 mgd  
Weir submergence = unsubmerged  
Head over weir = 0.22 ft

**UV system in channel 2** **908.72**

Change in elevation = 0.5 ft

**UV channel 2 influent gate** **908.78**

Opening type = rectangular gate  
Opening diameter/width = 30 in  
Gate height = 72 in  
Invert = 903  
Number of gates = 1  
Flow through gate(s) = 11 mgd  
Total area of opening(s) = 15 ft<sup>2</sup>  
Velocity through gate(s) = 1.13 ft/s  
Flow behavior = orifice, downstream control  
Gate loss = 0.05 ft  
Downstream water level = 908.72  
Upstream water level = 908.78

**UV channel flow split** **908.78**

User defined loss for flow split = 0 ft  
Total flow through flow split = 22 mgd

**UV influent channel** **908.78**

Channel shape = Rectangular  
Manning's 'n' = 0.014  
Channel length = 25 ft  
Channel width/diameter = 4 ft  
Flow = 22 mgd  
Downstream channel invert = 903

**Section Description**

**Water Surface Elevation**

Channel slope = 0 ft/ft  
Channel side slope = not applicable  
Area of flow = 23.11 ft<sup>2</sup>  
Hydraulic radius = 1.486  
Normal depth = infinite  
Critical depth = 1.31 ft  
Depth downstream = 5.78 ft  
Bend loss = 0 ft  
Depth upstream = 5.78 ft  
Velocity = 1.47 ft/s  
Flow profile = Horizontal

**Final clarifier combined effluent pipe**

909.27

Pipe shape = Circular  
Diameter = 42 in  
Length = 120 ft  
Flow = 22 mgd  
Friction method = Hazen Williams  
Friction factor = 120  
Total fitting K value = 1.9  
Pipe area = 9.62 ft<sup>2</sup>  
Pipe hydraulic radius = 0.875  
Age factor = 1  
Solids factor = 1  
Velocity = 3.54 ft/s  
Friction loss = 0.12 ft  
Fitting loss = 0.37 ft  
Total loss = 0.49 ft

**Final clarifier 1 effluent pipe**

909.42

Pipe shape = Circular  
Diameter = 36 in  
Length = 30 ft  
Flow = 11 mgd  
Friction method = Hazen Williams  
Friction factor = 120  
Total fitting K value = 1.5  
Pipe area = 7.07 ft<sup>2</sup>  
Pipe hydraulic radius = 0.75  
Age factor = 1  
Solids factor = 1  
Velocity = 2.41 ft/s  
Friction loss = 0.02 ft  
Fitting loss = 0.13 ft  
Total loss = 0.15 ft

**Final clarifier 1 effluent launder**

911.43

**Section Description**

**Water Surface Elevation**

Launder invert = 910  
Launder length = 155 ft  
Launder width = 2 ft  
Launder slope = 0 ft/ft  
Flow through launder = 5.5 mgd  
Critical depth = 0.83 ft  
Downstream depth = 0.83 ft  
Upstream depth = 1.43 ft

**Final clarifier 1 effluent v-notch weir**

912.16

Invert of V notch = 912  
Angle of V notch = 90 degrees  
Number of notches = 628  
Total flow over weir = 11 mgd  
Weir submergence = unsubmerged  
Head over weir = 0.16 ft

**Final clarifier 1 influent ports**

912.35

Opening type = rectangular orifice  
Opening diameter/width = 8 in  
Opening height = 12 in  
Invert = 908  
Number of openings = 12  
Flow through opening(s) = 11 mgd  
Total area of opening(s) = 8 ft<sup>2</sup>  
Velocity through opening(s) = 2.13 ft/s  
Flow behavior = orifice, downstream control  
Orifice loss = 0.18 ft  
Downstream water level = 912.16  
Upstream water level = 912.35

**Final clarifier 1 influent pipe**

912.59

Pipe shape = Circular  
Diameter = 36 in  
Length = 100 ft  
Flow = 11 mgd  
Friction method = Hazen Williams  
Friction factor = 120  
Total fitting K value = 2  
Pipe area = 7.07 ft<sup>2</sup>  
Pipe hydraulic radius = 0.75  
Age factor = 1  
Solids factor = 1  
Velocity = 2.41 ft/s  
Friction loss = 0.06 ft  
Fitting loss = 0.18 ft  
Total loss = 0.24 ft

**Section Description****Water Surface Elevation****Final clarifier 2 effluent pipe**

909.42

Pipe shape = Circular  
Diameter = 36 in  
Length = 30 ft  
Flow = 11 mgd  
Friction method = Hazen Williams  
Friction factor = 120  
Total fitting K value = 1.5  
Pipe area = 7.07 ft<sup>2</sup>  
Pipe hydraulic radius = 0.75  
Age factor = 1  
Solids factor = 1  
Velocity = 2.41 ft/s  
Friction loss = 0.02 ft  
Fitting loss = 0.13 ft  
Total loss = 0.15 ft

**Final clarifier 2 effluent launder**

911.43

Launder invert = 910  
Launder length = 155 ft  
Launder width = 2 ft  
Launder slope = 0 ft/ft  
Flow through launder = 5.5 mgd  
Critical depth = 0.83 ft  
Downstream depth = 0.83 ft  
Upstream depth = 1.43 ft

**Final clarifier 2 effluent v-notch weir**

912.16

Invert of V notch = 912  
Angle of V notch = 90 degrees  
Number of notches = 628  
Total flow over weir = 11 mgd  
Weir submergence = unsubmerged  
Head over weir = 0.16 ft

**Final clarifier 2 influent ports**

912.35

Opening type = rectangular orifice  
Opening diameter/width = 8 in  
Opening height = 12 in  
Invert = 908  
Number of openings = 12  
Flow through opening(s) = 11 mgd  
Total area of opening(s) = 8 ft<sup>2</sup>  
Velocity through opening(s) = 2.13 ft/s  
Flow behavior = orifice, downstream control  
Orifice loss = 0.18 ft  
Downstream water level = 912.16  
Upstream water level = 912.35

**Section Description**

**Water Surface Elevation**

**Final clarifier 2 influent pipe** **912.59**

Pipe shape = Circular  
Diameter = 36 in  
Length = 100 ft  
Flow = 11 mgd  
Friction method = Hazen Williams  
Friction factor = 120  
Total fitting K value = 2  
Pipe area = 7.07 ft<sup>2</sup>  
Pipe hydraulic radius = 0.75  
Age factor = 1  
Solids factor = 1  
Velocity = 2.41 ft/s  
Friction loss = 0.06 ft  
Fitting loss = 0.18 ft  
Total loss = 0.24 ft

**Final clarifier flow split** **912.59**

User defined loss for flow split = 0 ft  
Total flow through flow split = 22 mgd

**Final clarifier flow splitter weir** **914.18**

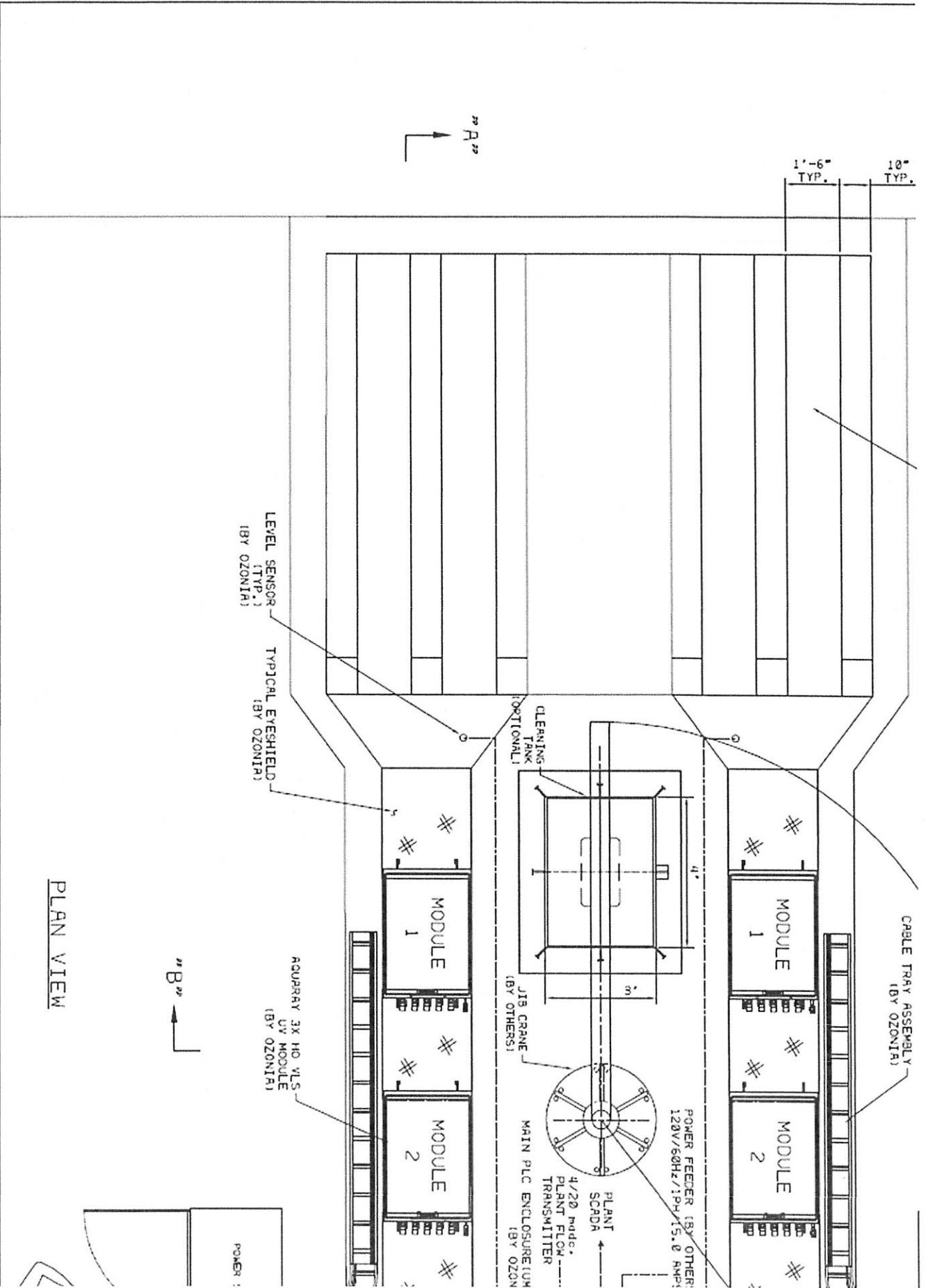
Weir invert (top of weir) = 913  
Weir length = 8 ft  
Weir 'C' coefficient = 3.33  
Flow over weir = 22 mgd  
Weir submergence = unsubmerged  
Head over weir = 1.18 ft

**42-inch trickling filter effluent pipe to final clarifiers** **915.28**

Pipe shape = Circular  
Diameter = 42 in  
Length = 325 ft  
Flow = 22 mgd  
Friction method = Hazen Williams  
Friction factor = 120  
Total fitting K value = 3.96  
Pipe area = 9.62 ft<sup>2</sup>  
Pipe hydraulic radius = 0.875  
Age factor = 1  
Solids factor = 1  
Velocity = 3.54 ft/s  
Friction loss = 0.34 ft  
Fitting loss = 0.77 ft  
Total loss = 1.11 ft

**APPENDIX II - EQUIPMENT PRODUCT INFORMATION/CUT  
SHEETS**

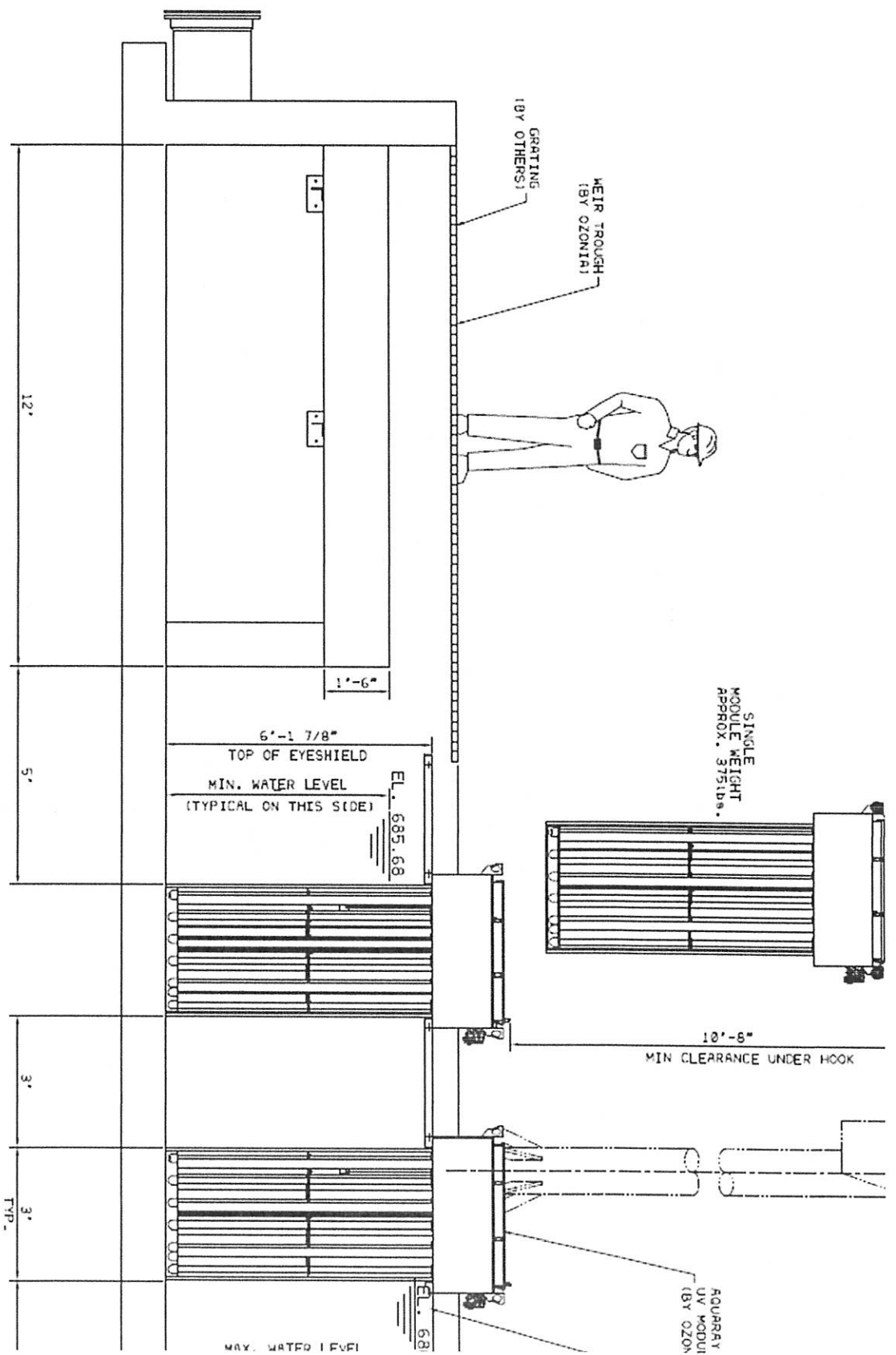
# UV SYSTEM



PLAN VIEW

REV.	DESCRIPTION	DRAWN	APPD.	DATE	APPROVED:	SCALE:	TITLE:
						NONE	PLANT SYSTEM
						03/08/12 HD	AQUARIY 3X HO VLS D
						CHECKED: 03/08/12 PDC	

# UV SYSTEM CONTINUED



SECTION <sup>HP</sup> A-A<sup>HP</sup>

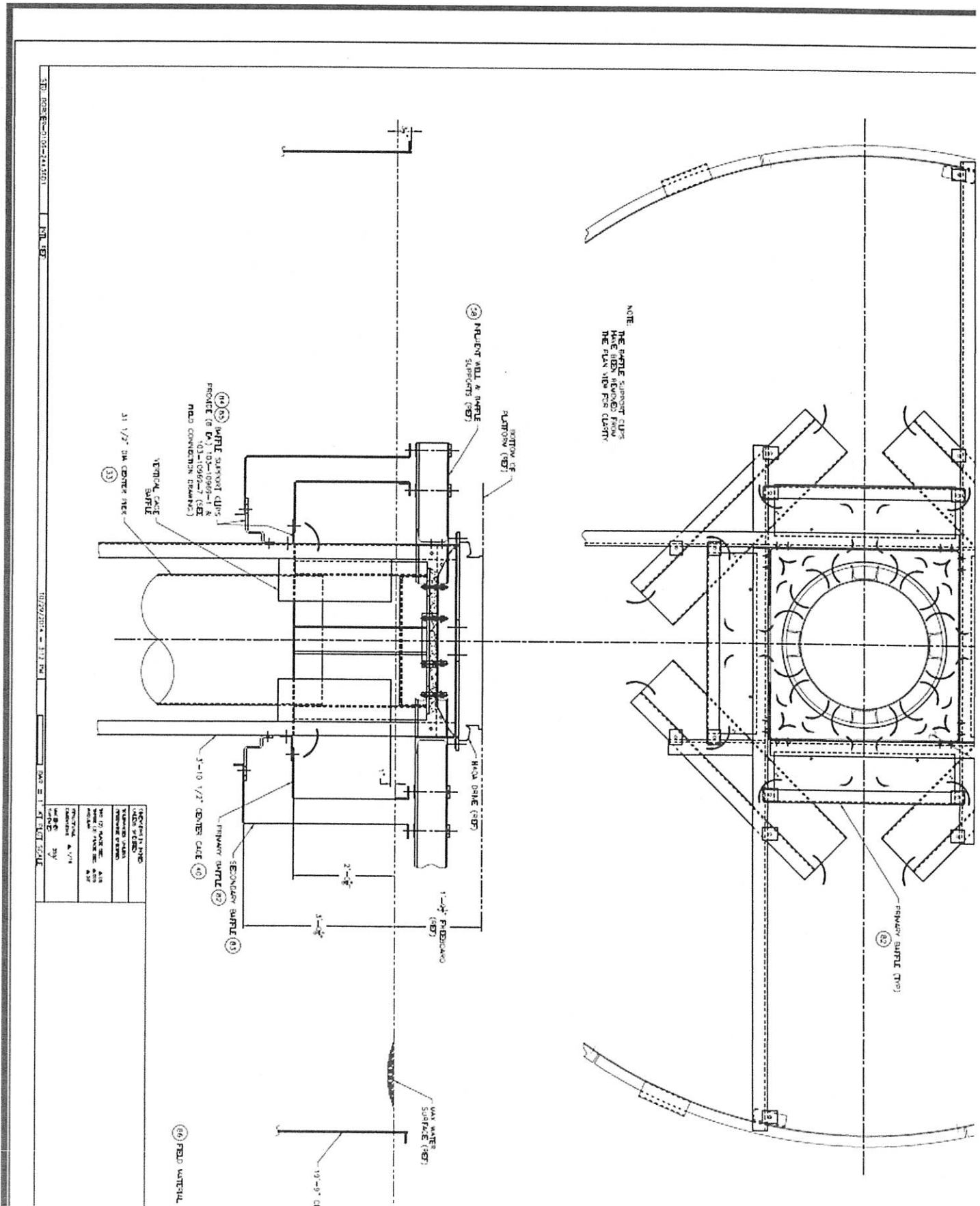
REV.	DESCRIPTION	DRAWN	APPD.	DATE	SCALE:	TITLE:
					NONE	SECTION AND DET
					DRAWN: 03/08/12 HD	AQUARIUM 3X HO VLS DIS
					CHECKED: 03/08/12 PDC	
					APPROVED:	







# FINAL CLARIFIER MECHANISMS CONTINUED





# EFFLUENT WATER PUMPS



## PUMP SUBMITTAL DATA

Quote Number: 9001-141022-036  
66SV60GJ8F64 - 10HP 4/60/3 TEPE 362 6STG

### ENGINEERING DATA

#### Hydraulics

FLOW AT DESIGN	176 USGPM
TOTAL DYNAMIC HEAD AT DESIGN	150 ft
NPSHr	4.0 ft

#### Pump

PRODUCT NUMBER	66SV60GJ8F64
SIZE	66SV
NUMBER OF STAGES	6
MATERIAL	Cast Iron/304SS
SUCTION SIZE	4 inch
DISCHARGE SIZE	4 inch
POWER AT DESIGN	8.72 Hp
EFFICIENCY AT DESIGN	76.4 %
APPROXIMATE NET WEIGHT	398 lbs

#### Motor

PART NUMBER	V12A34E5BE2S
NAMEPLATE HORSEPOWER	10 Hp (7.46 kW)
PHASE	3
MAXIMUM FREQUENCY	60 Hz
VOLTAGE	208-230/460 V
SPEED	1800 RPM
ENCLOSURE	TPE
FRAME SIZE	215TC

#### Impeller

IMPELLER TYPE	Radial Impeller
QUANTITY	6

#### Mechanical Seal

SHAFT SEAL	Silicon Carbide/Silicon Carbide/EPR
------------	-------------------------------------

#### Options

LOW NPSH	No
HIGH PRESSURE	No

#### Certified Testing

None

### STANDARD EQUIPMENT / CAPABILITY

#### Pump

The e-SV pump is a non-self-priming vertical multistage pump coupled to a standard motor. The liquid end, located between the upper cover and the pump casing, is held in place by tie rods. The pump casing is available with different configurations and connection types.

- Delivery; up to 700.0 gpm/ Head: up to 1200 ft
- Temperature of pumped liquid: -20.0°F to 250.0°F standard version
- Direction of rotation; clockwise looking at the pump from the top down (marked with an arrow on the adapter and on the coupling).
- G - Cast Iron/304SS ANSI Flange with 200 psi g MAWP
- Seal housing chamber designed to prevent the accumulation of air in the critical area next to the mechanical seal
- Mechanical seal according to EN 12756 (ex DIN 24960) and ISO 3069
- Versions with ANSI flanges that can be coupled to ANSI counter-flanges
- Threaded oval counter-flanges made of stainless steel are standard supply for the T versions
- Easy maintenance. No special tools required for assembly or disassembly
- Standard version for temperatures ranging from: 0°F to 250°F (optional to 300°F)

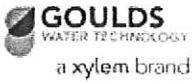
#### Motor

- Standard NEMA 215TC Frame motor premium efficient totally enclosed fan cooled
- 1800 RPM nominal
- 3 phase version, 4 pole: 208-230/460 V, 60Hz, 10 Hp
- Reduced axial thrusts enable the use of standard NEMA TC motors that are easily found in the market

#### DO NOT USE FOR CONSTRUCTION UNLESS CERTIFIED

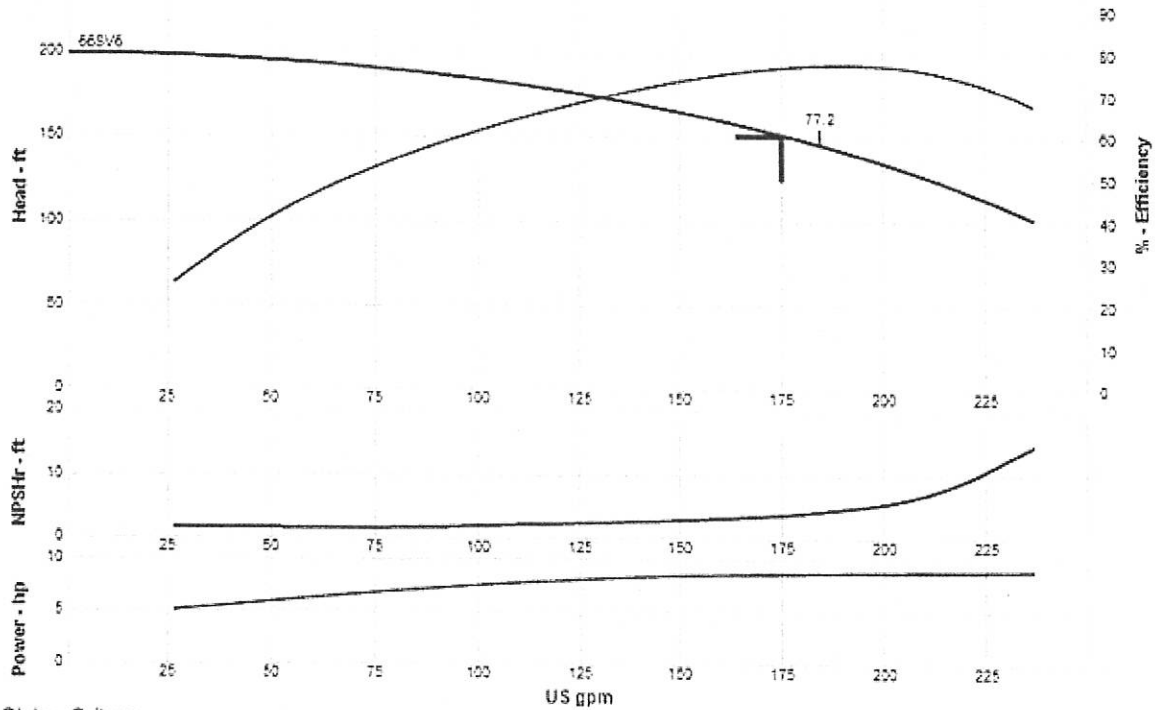
Certified by	
Date of Certification	
Pump Serial Number	
Project Name	P-7004
Tag	

# EFFLUENT WATER PUMPS CONTINUED



## PERFORMANCE CURVE

Quote Number: 9001-141022-036  
 66SV60GJ8F64 - 10HP 4/60/3 TEPE 362 6STG



### Sizing Criteria

Series:	E-SV	Maximum Pressure:	86 psi
Size:	66SV	Maximum Head:	199 ft
Number of Impellers:	6	Head at Design:	150 ft
Number of Trimmed Impellers:	0	Head on Performance Curve:	151 ft
Trimmed Impeller Size:	N/A	Best Efficiency:	77.2 %
Speed:	1800 RPM	Efficiency at Design:	76.4 %
Curve Type:	Standard NPSH	NPSHr:	4.0 ft
Curve ID:	66SV6	Power at Design:	8.72 Hp
Maximum Flow:	237 USGPM	NOL Power:	8.96 Hp
Flow at Design:	176 USGPM	Fluid Type:	WATER
Flow at BEP:	184 USGPM	Temperature:	70.0 °F

**DO NOT USE FOR CONSTRUCTION UNLESS CERTIFIED**

Certified by	
Date of Certification	
Pump Serial Number	
Project Name	P-7004
Tag	

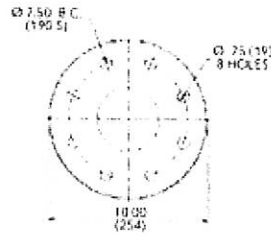
# EFFLUENT WATER PUMPS CONTINUED



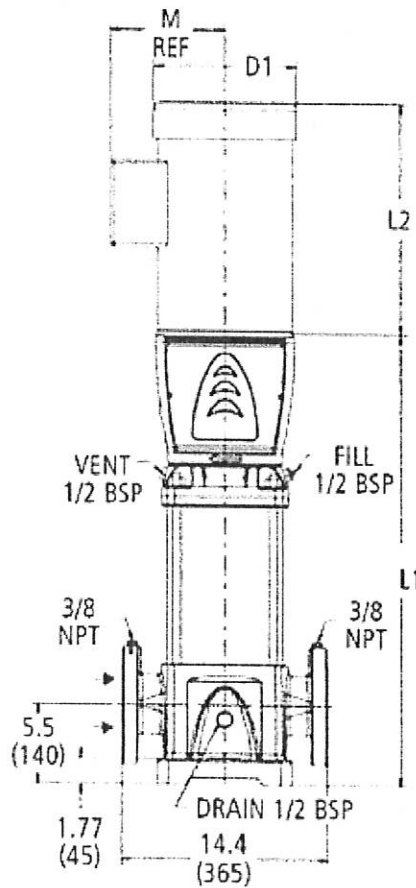
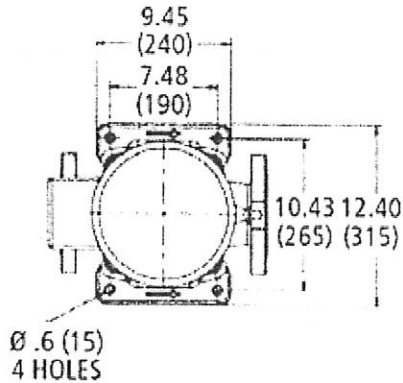
## OUTLINE DRAWING

Quote Number: 9001-141022-036  
 66SV60GJ8F64 - 10HP 4/60/3 TEPE 362 6STG

Weight	Dimensions	
398 lbs	L1	40.72 inch
	L2	30.89 inch
	D1	19.25 inch



Cast Iron  
 4" Class 125 F.F.



**DO NOT USE FOR CONSTRUCTION UNLESS CERTIFIED**

Certified by	
Date of Certification	
Pump Serial Number	
Project Name	P-7004
Tag	

**SLIDE GATES**



STAINLESS STEEL

## SERIES 15

STANDARD FLOW CONTROL SLIDE GATES / PENSTOCKS

### General description

Fontaine's SERIES 15 (see FIG 1) is a highly STANDARD versatile flow control slide gate. Adaptable to different applications, the SERIES 15 is designed to withstand maximum heads both upstream and downstream with the following performance.

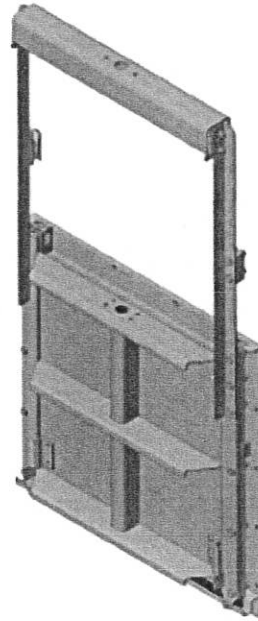
- 30 (9.0m) for 6" to 12" (150mm to 300mm). 0.04 USGPM/ft<sup>1</sup> (0.5L/min)<sup>2</sup>
- 25 (7.5m) for 16" to 24" (400mm to 600mm). 0.05 USGPM/ft (0.6 L/min)
- 20 (6.0m) for 28" to 40" (700mm to 1000mm). 0.06 USGPM/ft (0.7L/min)

The leakage rate is maintained inside boundaries of maximum allowable leakage recommended by AWWA C561 and BS7775 (2005). The SERIES 15 is available in eleven standard sizes from 6" up to 40" (150 mm up to 1000 mm). The design is suitable for square or round applications.

### Construction

The SERIES 15 is made of 316L stainless steel providing high corrosion and erosion resistance, and can be operated for many years with a minimum of maintenance. Stainless steel provides virtually limitless design flexibility. The result is a lighter weight and easier-to-install gate.

In accordance with our high standards of quality, the slide gates are completely factory assembled as well as tested for operation and leakage before being shipped. It eliminates the needs for any on-site assembly and adjustments.



### Mounting

The SERIES 15 is designed for concrete wall (CW).

FIG 1 S152 General view

### AWWA C561, BS Standards

SERIES 15 slide gates are designed and manufactured to meet or exceed the AWWA C561 and the main European standards for leakage, design safety factors, stem guide positioning, and buckling on lifting mechanisms. Leakage and operation tests are performed on all Fontaine SERIES 15 before shipping.

### Seals

SERIES 15 seals are made of an ultra high molecular weight polyethylene (UHMWPE) allowing no metal to metal contact. Its low coefficient of friction makes the gate easier to open even when not operated for long periods of time. The self-adjusting feature is obtained by a continuous compression cord that ensures a tight seal between the gate and the frame in both seating and unseating conditions. This system guarantees a perfect seal between the gate and the frame for water pressure conditions upstream or downstream. The continuous wedging action of the compression cord on the side seals enables the SERIES 15 slide gates to control flow by allowing water only through the open portion of the gate. The neoprene seal on the bottom is a flat type and does not obstruct water flow.

<sup>1</sup> Leakage rate per U.S. gallon per minute per foot of seal of perimeter

<sup>2</sup> Leakage rate per liter per minute per meter of seal of perimeter





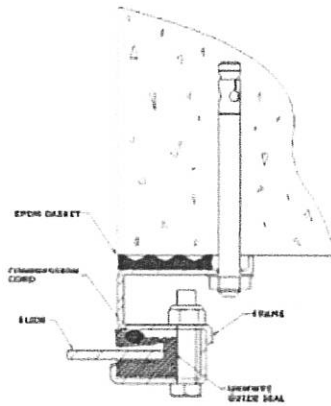
STAINLESS STEEL

## SERIES 15

STANDARD FLOW CONTROL SLIDE GATES / PENSTOCKS

### Seals arrangement

The side and top seals (SECTION A A & B B) of the SERIES 15 are made of self lubricating UHMWPE, allowing no metal-to-metal contact.



SECTION A-A: Side seal arrangement

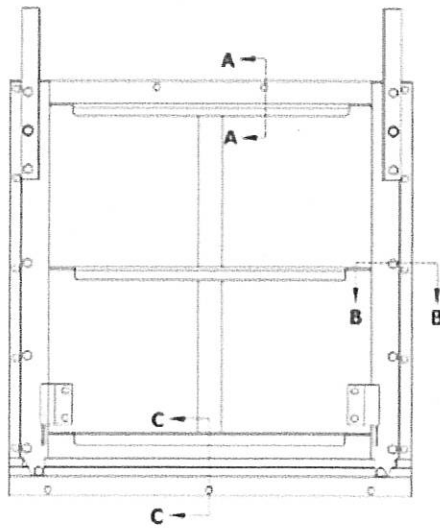
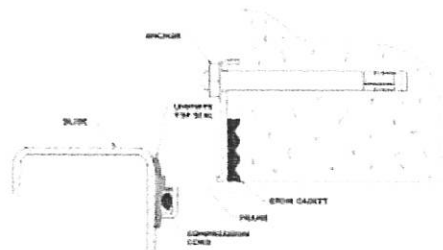
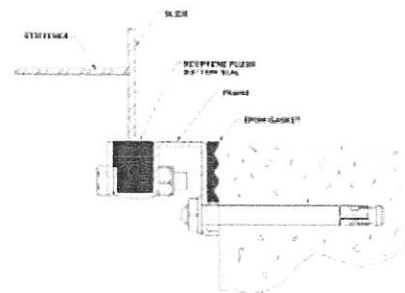


FIG 3 Section view details



SECTION B-B: Top seal arrangement

The flush-bottom seal (SECTION C-C), made of resilient neoprene, leaves the opening unobstructed when the gate is in the open position.



SECTION C-C: Frame bottom (all)





STAINLESS STEEL

## SERIES 15

STANDARD FLOW CONTROL SLIDE GATES / PENSTOCKS

### Dimensions - 152-CW

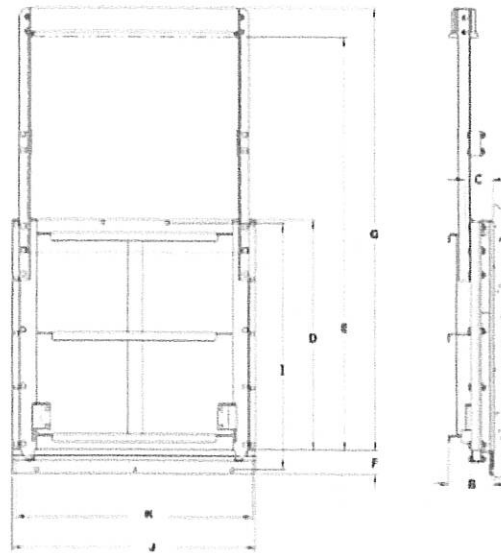


FIG 4 152 CW. General layout (see TABLE 2 p 11)

### Specific layouts

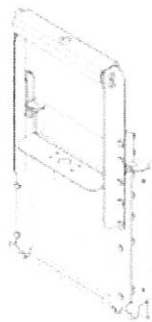


FIG 5 152 CW 6' 12" (150mm 300mm)

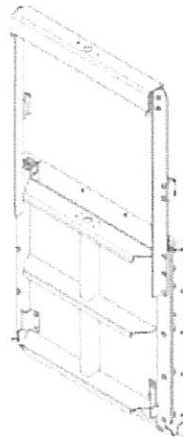


FIG 6 152 CW 16' 24" (400mm 600mm)

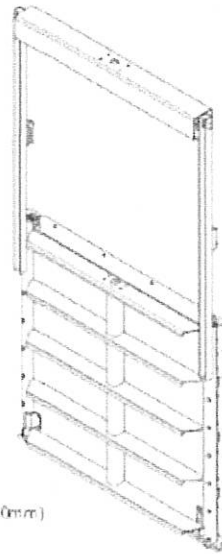


FIG 7 152 CW 28' 40" (700mm 1000mm)



STAINLESS STEEL

## SERIES 15

STANDARD FLOW CONTROL SLIDE GATES / PENSTOCKS

### Dimensions - 154-CW

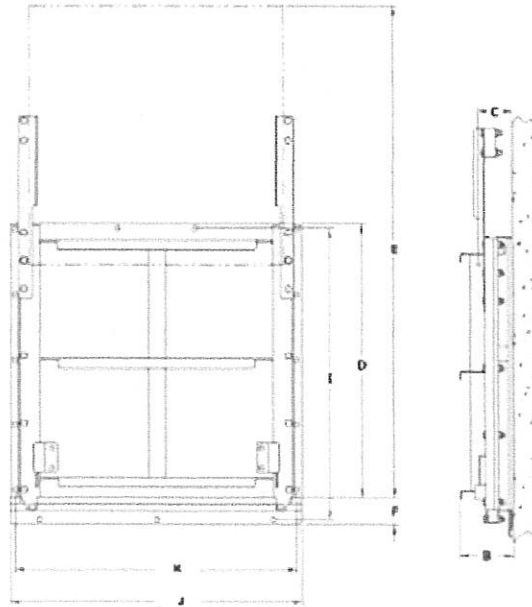


FIG.8 154 CW. General layout (see TABLE 2 p. 10)

### Specific layouts

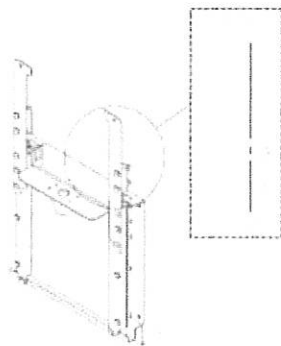


FIG.9 154 CW 6' 12" (150mm x 300mm)

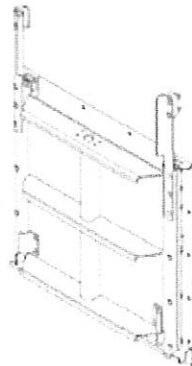


FIG.10 154 CW 16' 24" (400mm x 600mm)

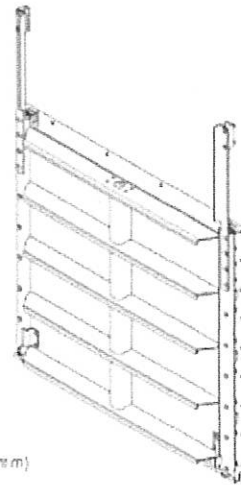


FIG.11 154 CW 28' 40" (700mm x 1000mm)



**SLUICE GATES**

## STAINLESS STEEL SLUICE GATES



# Fontaine

## SERIES 20

### FLOW CONTROL SLUICE GATES

*Designed to be adapted to all applications  
Maximum leakage rate is less than half of the AWWA allowable  
Low-maintenance gates*

\*\*\*

### General Description

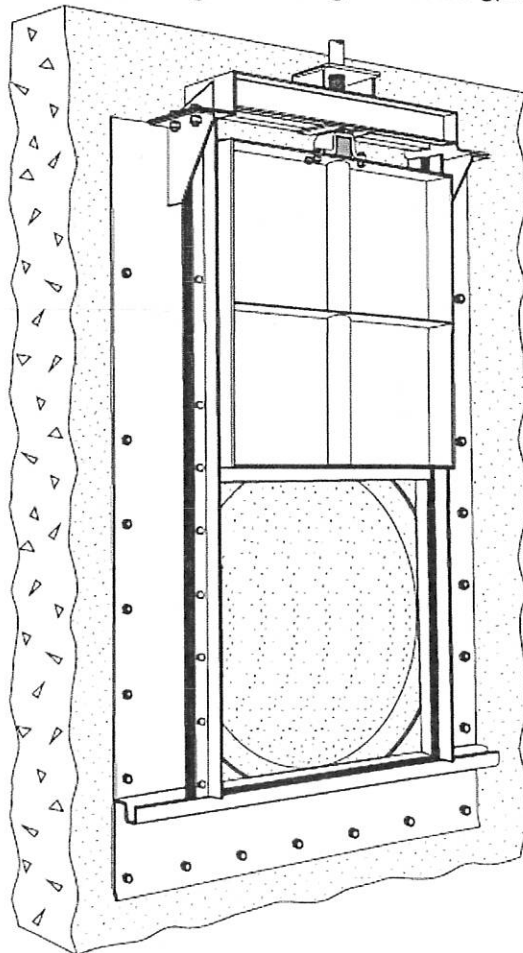
Fontaine's SERIES 20 Sluice Gate is a highly versatile flow control gate with various types of mountings. Adaptable to different applications, the Series 20 can be designed to withstand seating and unseating heads of up to 60 feet (18 m). When required, the SERIES 20 can be designed for higher water heads. The seal design keeps the allowable leakage rate to 0.05 U.S. gpm per foot (0.60 l/min per meter) of seating perimeter for seating head conditions. The unseating head leakage is 0.1 U.S. gpm (1.25 l/min per meter) per foot of perimeter up to 20 feet (6 m). Over 20 feet (6m) of unseating head, the leakage is corrected at a rate of 0.0025 U.S. gpm (0.03 l/min per meter) per foot of perimeter for each additional foot (meter) over 20 feet (6 m). The leakage rate is 50 % of the maximum allowable leakage recommended by AWWA. It is available in sizes from 6" (152 mm) up to 120" (3048 mm). For larger sizes, please contact a Fontaine representative. The design is suitable for square, rectangular or round applications.

### Stainless Steel Construction

Because of its stainless steel construction, the SERIES 20 has high corrosion and erosion resistance, and can be operated for many years with a minimum maintenance. Stainless steel provides virtually limitless design flexibility. The result is a lighter weight and easier-to-install gate.

### AWWA Standards

SERIES 20 Sluice Gates are built to meet or exceed AWWA C561 latest revision standards pertaining to design safety factors, stem and stem guides positioning, manual lifting devices, leakage, etc. As specified in the AWWA standard, all Fontaine SERIES 20 water gates are tested for leakage and operation before shipping.



STAINLESS STEEL SLUICE GATES



**Fontaine**

**SERIES 20**

FLOW CONTROL SLUICE GATES

No.	Part	Material
1	Frame	Stainless steel ASTM A-240 Type 304L or 316L
2	Guides and Side seals	Ultra high molecular weight polyethylene (UHMWPE) ASTM D-4020
3	Compression cord	Nitrile ASTM D-2000 M6BG 708, A14, B14, E014, E034
4	Bottom seal	Neoprene ASTM D-2000 Grade 2 BC-510
5	Slide	Stainless steel ASTM A-240 Type 304L or 316L
6	Top seal	Ultra high molecular weight polyethylene (UHMWPE) ASTM D-4020
7	Yoke	Stainless steel ASTM A-240 Type 304L or 316L

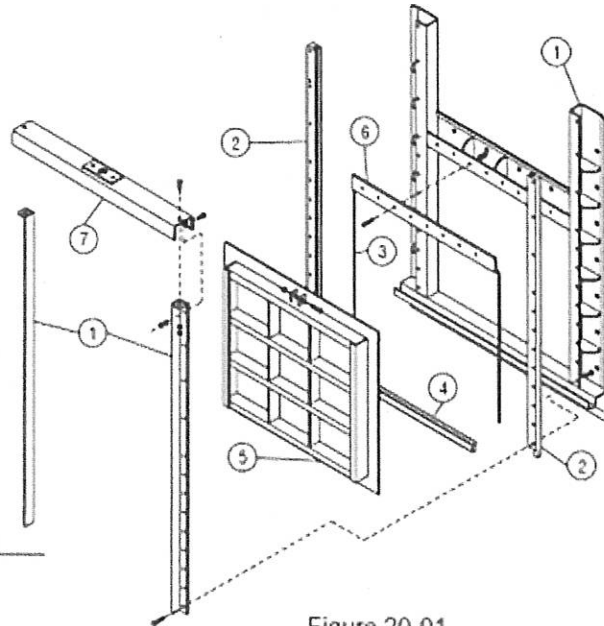


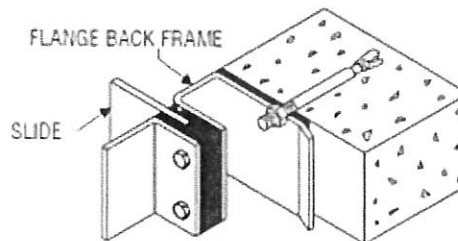
Figure 20-01  
Exploded view of a Series 20 Model 203

**Flange Back Frame**

The stainless steel frame on the SERIES 20 is a flange back type (Detail 20-01) available in open or self-contained configurations, providing a solid one-piece gate. The rigidity provided by the flange back frame makes it easier to handle in transportation and installation with less risk of distortion. The seal bolting is completely separated from the flange anchoring, allowing the flange to be modified to better suit all particular applications. This feature also allows the gate to be completely factory assembled as well as tested for operation and leakage before being shipped. It also eliminates any on-site assembly and adjustments.

**Reinforced Slide**

The slide is a stainless steel plate reinforced with members welded to the plate, making it a solid single piece.



Detail 20-01  
Flange back frame

STAINLESS STEEL SLUICE GATES



**Fontaine**

**SERIES 20**

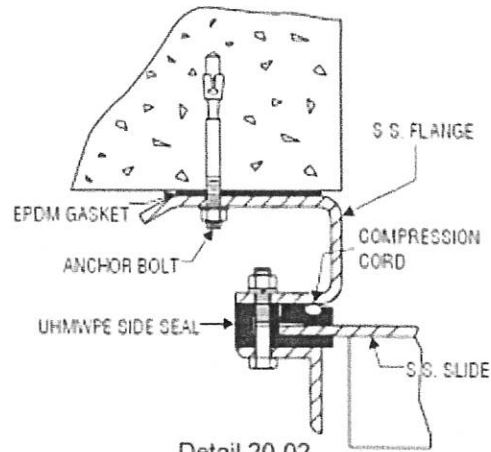
FLOW CONTROL SLUICE GATES

Following Details 20-02 through 20-05 refer to figures 20-08, 09, 10 on pages 6, 7 and 8.

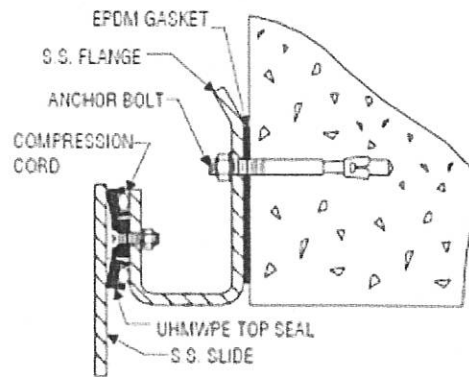
**UHMWPE Seals  
(U.S. and Canadian Patents)**

The side and top seals (Detail 20-02, 03) of the SERIES 20 are made of a self-lubricating ultra high molecular weight polyethylene (UHMWPE), allowing no metal-to-metal contact. With a friction coefficient of 0.2, the seals make the gate easier to open even when not operated for a long period of time. The self-adjusting feature is obtained by a continuous compression cord that ensures a tight seal between the slide and the frame in both seating and unseating conditions. The continuous wedging action of the compression cord on the slide enables the SERIES 20 Sluice Gate to control flow by allowing water only through the open portion of the gate.

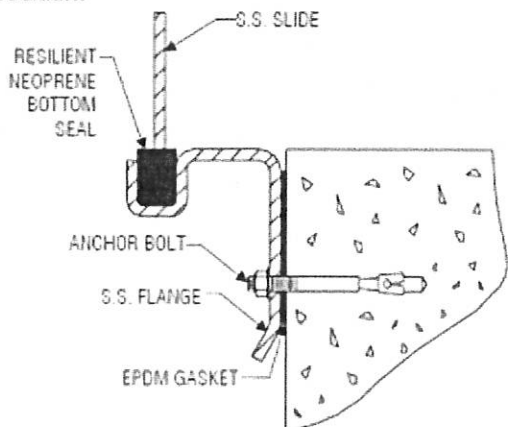
The flush-bottom seal (Detail 20-04, 05), made of resilient neoprene, leaves the opening unobstructed when the slide is in the open position.



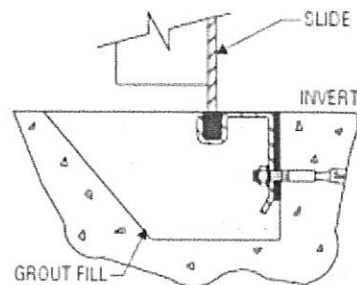
Detail 20-02  
Section "A-A" of the side frame



Detail 20-03  
Section "B-B" of the top frame



Detail 20-04  
Section "C-C" of the flush-bottom frame



Detail 20-05  
Flush-bottom frame installed in a concrete box out

STAINLESS STEEL SLUICE GATES



**Fontaine**

**SERIES 20**

FLOW CONTROL SLUICE GATES

**Mountings**

The Series 20 can be mounted in almost any application. Figures 20-02 through 20-07 show the different mountings (for more mounting details, refer to "Mountings" in the Introduction section):



Figure 20-02  
Directly mounted on a concrete wall  
(CW)

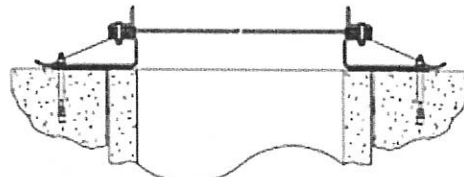


Figure 20-03  
In front of a flush pipe on a concrete wall  
(CWX)

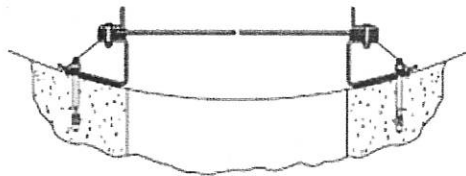


Figure 20-04  
Inside a round manhole  
(RM)

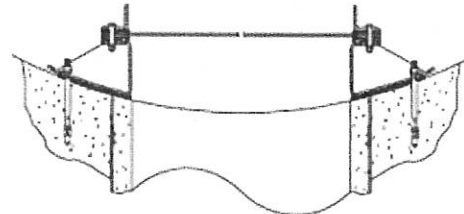


Figure 20-05  
In front of a flush pipe  
inside a round manhole  
(RMX)

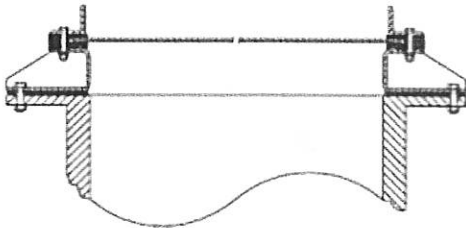


Figure 20-06  
On a standard flange  
(SF)

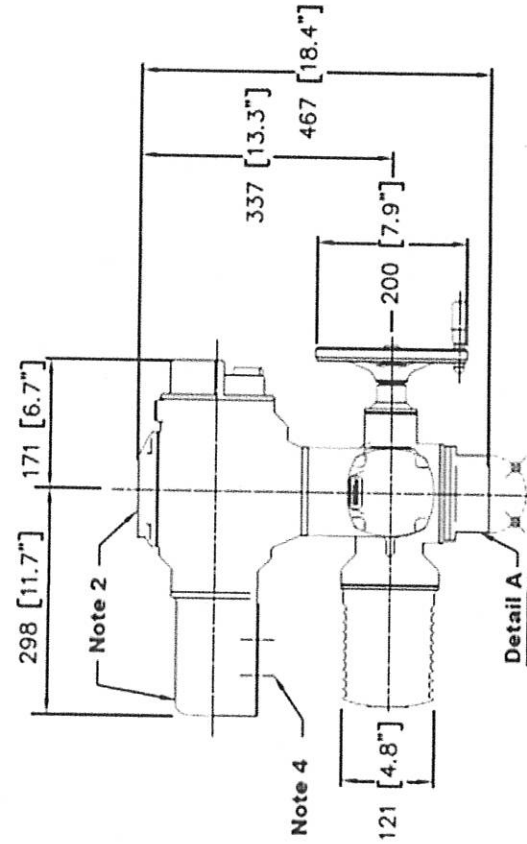
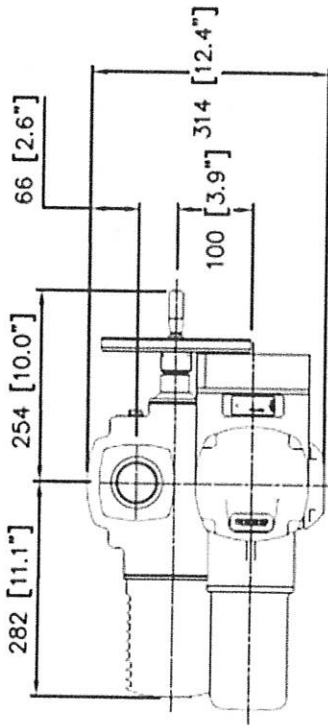


Figure 20-07  
On a wall thimble  
(WT)

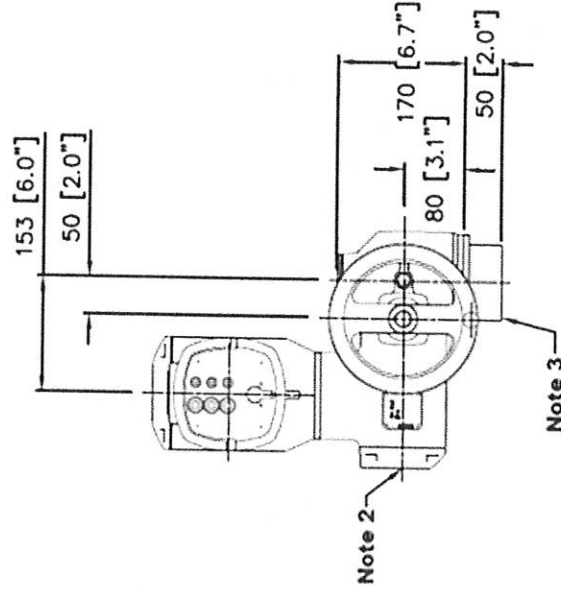
**ELECTRIC ACUTATORS**

**Notes:**

1. Metric tolerance per ISO 2768-m. Dimensions in brackets [ ] are in inches and rounded to one decimal place.
2. Seven inch minimum clearance recommended for removal of access cover and equipment adjustment.
3. See appropriate drive drawing for detail.
4. See data sheets for conduit entries.



Detail A



 AUMA ACTUATORS, INC.	A - drive No gearbox	SA(R) 10.2 Motor 3 phase Handwheel - 7.9"	AM01.1 P&S - 100mm	PART ISSUE DESCRIPTION	E. DATE A. DATE M. DATE S. DATE	DDS100001CAADA011
	Description of the drawing content, including part name and issue information.					

**Attachment C: Bethel Park Treatment and  
Union Township Removal Calculations**

## Bethel Park Treatment Plant and Union Township Solids Removal Calculations

### Treatment Plant - Existing Final Clarifiers

Assumptions:

4 of 5 final clarifiers on-line

Suspended solids to final clarifiers of 80 mg/L from trickling filter process (determined from field testing of trickling filter effluent)

Average flow of 4 MGD

Typical measured clarifier effluent (existing) = 30 mg/L

### ***Solids removal calculation***

Solids removed (lbs/day) =  $(80 \text{ mg/L} - 30 \text{ mg/L}) * 4 \text{ MGD (average flow)} * 8.34 = 1,668 \text{ lbs/day}$

### Treatment Plant - New Final Clarifiers

Assumptions:

Both final clarifiers on-line

Suspended solids to final clarifiers of 80 mg/L from trickling filter process (determined from field testing of trickling filter effluent)

Average flow of 4 MGD

Predicted clarifier effluent (based on modeling) = 14 mg/L

### ***Solids removal calculation***

Solids removed (lbs/day) =  $(80 \text{ mg/L} - 14 \text{ mg/L}) * 4 \text{ MGD (average flow)} * 8.34 = 2,202 \text{ lbs/day}$

*Therefore the total additional solids removed as a result of the clarifier/UV project is predicted to be 534 lbs/day for average flow conditions.*

**Attachment D: Bethel Park Sewage Treatment  
Plant Sludge Study**

## TECHNICAL MEMORANDUM

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### Bethel Park Sewage Treatment Plant Sludge Study

**Prepared For:** Bethel Park Municipal Authority

**Prepared By:** Dan Goodwin, PE  
Clayton E. McCormack, PE

**Date:** December 13, 2012

#### Background

Wade Trim was authorized to perform an evaluation to determine the current and future sludge production for the Bethel Park Sewage Treatment Plant (STP), including estimating the solids generated by additional treatment necessary for additional nutrient removal that could be imposed on the plant by future Total Maximum Daily Load (TMDL) conditions. A digester upgrade project is being considered in the near term to maintain the operation of the existing system. A future modest increase in flow or the imposition of a future nutrient effluent limit may result in an increase in biosolids generation which needs to be anticipated during the planning of the digester upgrade work.

#### Treatment Process

The Bethel Park STP is a 4.92 million gallons per day (MGD) secondary wastewater treatment plant located in Bethel Park, PA. The plant currently has a reported annual average daily flow of approximately 4.1 MGD. The plant consists of the following liquid stream unit treatment processes: head works, primary clarifiers, secondary treatment with fixed film trickling filters, final clarifiers, sand filters (not in use), and chlorine contact tanks. The unit treatment process for the processing of the primary sludge and secondary sludge consists of gravity thickening, anaerobic digestion (primary and secondary digesters), and dewatering using a 1.5 meter belt filter press.

#### Future Effluent Limits

Based upon the currently regulatory environment, an effluent limit for total nitrogen of 4 to 5 mg/L as N and a total phosphorus limit of 0.2 to 0.4 mg/L as P could potentially be imposed within the next few permit cycles. As such, the plant will need to be retrofitted with additional treatment to achieve the removal to achieve these total nitrogen and phosphorus limits.

#### Approach

Wade Trim requested and was provided with plant operating information, including the following:

- Plant Influent Annual Average Daily Flow
- Influent Pollutant Concentration for CBOD, TSS, TKN, and Total P
- Primary Effluent Pollutant Concentration for CBOD, TSS, TKN, and Total P
- Primary Sludge Average Daily Flow
- Secondary Sludge Average Daily Flow

- Thickened Sludge Solids Concentration
- Thickened Sludge Underflow
- Anaerobic Digester Average Biogas Production
- Anaerobic Digester Typical Operating Temperature
- Anaerobic Digester Supernatant Flow
- Belt Filter Press Feed Pump Flow
- Dewatered Cake Total Solids Concentration

Using the plant data that was provided, a mass balance analysis of the treatment process was prepared. The mass balance was performed with the intent of calibrating the calculations to the total solids load at the thickener and to the volume of biogas generated at the primary anaerobic digester.

Good agreement was obtained between the calculated total solids load at the thickener as compared to the reported value. A calculated value of 4,340 lbs of solids per day was obtained from the analysis as compared to the reported value of 4,590 lbs of solids per day. This is a 5.3 percent difference which is considered acceptable given the planning level nature of the analysis and the limited amount of plant data. However, less agreement of the analysis was obtained for the calculated volume of biogas being produced at the anaerobic digester. The mass balance analysis calculated generating 25,200 cubic feet per day of gas (at 0°C and 1 atm) while the plant is reporting generating approximately 30,000 cubic feet per day of gas production. There is a 16 percent difference between the calculated and reported gas production volume. The calculation of less gas production than reported implies the digester is being loaded with more volatile suspended solids than calculated and the digester is performing better (having a higher level of volatile suspended solids removal) than expected. It will be necessary to obtain more plant data to refine the mass balance calculations to achieve better agreement between the calculated and reported biogas production volume.

Current Sludge Production

Solids handling facilities are generally evaluated on the basis of the period of time when the maximum solids generation is expected. At the Bethel Park STP, the period of maximum solids production occurs during the seasonal “unloading” of the trickling filters. When the seasons change in the spring and fall, the change in ambient temperature causes a rapid loss of the attached biomass that can overwhelm the solids handling treatment processes at the plant. The loss of biomass can be sustained for a period of two to four weeks depending on the weather conditions. Additionally, periods of unloading may occur throughout the year. Therefore, the balance of the analysis will be based upon worst case conditions characterized as the time period of trickling filter unloading.

Based upon the current influent flow of 4.1 million gallons per day, the current influent pollutant loadings of 3,800 lbs/day of CBOD / 3,600 lbs/day of TSS, and seasonal trickling filter “unloading” conditions, then the following is the calculated current generation of biosolids from the treatment process under worst case type conditions.

	<b>Current</b>
Annual Average Flow, MGD	4.1
Primary Sludge, lbs/day	2,400
Secondary Sludge w/Unloading, lbs/day	3,900
Alum Sludge, lbs/day	0
Denit Sludge, lbs/day	0

Total Sludge, lbs/day	6,300
Digestion, lbs/day	-1,950
Total for Disposal, lbs/day	4,350

Current Gravity Thickener/Digester Performance

The mass balance analysis was also used to determine the performance of the thickener and digester relative to typical ranges for specific design parameters. The following table shows this comparison:

**Current Gravity Thickener Performance**

Parameter	Value	Typical Range
Solids Loading, lbs/sqft day	6.1	12 to 20
Overflow Rate, gpd/sqft	155	150 to 300
Side Water Depth, ft	9.75	10 to 13

**Current Anaerobic Primary Digester Performance**

Parameter	Value	Typical Range
Solids Loading, lbs VSS/1000 cu ft day	55	< 40 for Poor Mixing, < 80 for Good Mixing
Solids Retention Time, day	12	15 to 20

Note: Typical range provide is for mesophilic, high-rate, complete-mix anaerobic digester.

The Gravity Thickener appears to be operating within typical solids loading and overflow rate rates for current conditions. However, the side water depth of the unit is less than recommended. This results in the unit providing significantly less volume of storage than is typical, and potential washout of solids if the thickener is carrying a high sludge blanket.

The Anaerobic Digester appears to be operating within typical parameters when compared to solids loading and a slightly shorter solids retention time than is recommended. It should be noted that plant staff reports the digester contained an unknown amount of accumulated grit and scum that, if present in significant quantity, would result in an even shorter solids retention time than shown above.

Future Sludge Production

The principal purpose of the analysis was to determine the expected future sludge generation at an increased influent flow of 5.5 million gallons per day as well as consider the solids generated by the additional level of treatment required to meet future effluent permit limits for nutrients.

Future effluent limits for total nitrogen will likely require the implementation of denitrification unit treatment process so as to convert nitrates to nitrogen gas and therefore remove total nitrogen. Because the current secondary treatment process is a fixed film type attached growth trickling filter, the logical treatment approach to performing denitrification is to follow the existing secondary system with an attached growth, downflow, deep-bed type denitrification filter with supplemental carbon addition. Examples of this type of treatment technology include the Severn Trent TETRA® DeepBed™ Denite® System or the Leopold® elimi-NITE® Denitrification System. Additional solids from this treatment system are generated when the filters are backwashed and the biomass attached to the media is removed during the backwash process.

Future effluent limits for total phosphorus will likely require the implementation of a chemical addition of an aluminum salt (alum) to provide chemical phosphorus removal. The chemical would be added prior to the final clarifiers to allow the chemical precipitate to form following secondary treatment and co-settle with the biological solids that slough off from the trickling filter media.

The mass balance analysis has been completed to project the additional sludge generation if these nutrient removal technologies were to be implemented in the future.

	Current	Future	Future w/Nutrient Removal
<b>Annual Average Flow, MGD</b>	<b>4.1</b>	<b>5.5</b>	<b>5.5</b>
Primary Sludge, lbs/day	2,400	3,200	3,400
Secondary Sludge w/Unloading, lbs/day	3,900	5,100	5,100
Alum Sludge, lbs/day	0	0	1,200
Denit Sludge, lbs/day	0	0	1,000
<b>Total Sludge, lbs/day</b>	<b>6,300</b>	<b>8,300</b>	<b>10,700</b>
Digestion, lbs/day	-1,950	-2,100	-2,400
<b>Total for Disposal, lbs/day</b>	<b>4,350</b>	<b>6,200</b>	<b>8,200</b>

As can be seen above, increasing the plant influent flow from 4.1 to 5.5 million gallons per day results in an increased sludge generation for disposal of 1850 lbs/day, which is an increase of 42%. When the addition of the sludge from additional nutrient removal is considered, the sludge generation increases to 8,200 lbs/day, which is approximately double the current sludge production.

Future Gravity Thickener/Digester Performance

Given the expected future solids generation, the following table shows the performance of the thickener and digester relative to typical range for specific design parameters.

**Future Gravity Thickener Performance**

Parameter	Value	Typical Range
Solids Loading, lbs/sqft day	10.5	12 to 20
Overflow Rate, gpd/sqft	270	150 to 300
Side Water Depth, ft	9.75	10 to 13

**Future Anaerobic Primary Digester Performance**

Parameter	Value	Typical Range
Solids Loading, lbs VSS/1000 cu ft day	84	< 40 for Poor Mixing, < 80 for Good Mixing
Solids Retention Time, day	7.3	15 to 20

Note: Typical range provide is for mesophilic, high-rate, complete-mix anaerobic digester.

The Gravity Thickener continues to be operating within typical solids loading and overflow rates for future conditions. However, the side water depth of the unit is less than recommended. This results in the unit providing significantly less volume of storage than is typical.

The Anaerobic Digester appears to be operating within typical parameters when compared to solids loading but the solids retention time is well below the typical range. Operating the digester with less solids retention time than recommended will result in incomplete digestion, objectionable odors during dewatering, and less than optimal biogas production.

### Recommendations

As a result of this work, Wade Trim makes the following finding and recommendations:

- 1) The Bethel Park Sewage Treatment Plant currently generates approximately 4,300 lbs per day of biosolids during period of high solids production, which typically occurs during the seasonal “unloading” of the trickling filters. This unloading period can last for up to a month.
- 2) The existing solids processing facilities are adequate sized to treat the currently solids production. However, the gravity thickener has a side water depth less than the recommended minimum side water depth. If the sludge blanket in the thickener is high, this can cause a washout of solids.
- 3) To meet a future total nitrogen and total phosphorus permit limit, Wade Trim has selected to calculate the additional solids production assuming the use of attached growth, downflow, deep-bed type denitrification filter and the addition of alum for chemical phosphorus removal. This system was chosen because it should work well with the plant’s existing trickling filter biological treatment.
- 4) The production of solids can be expected to approximately double in the future to 8,200 lbs/day if the influent flow increases to 5.5 MGD and the plant has to provide additional treatment due to future permit limits for total nitrogen and total phosphorus.
- 5) The anaerobic digestion volume needs to increase an additional 350,000 gallons to provide the appropriate solids retention time for the amount of sludge expected if the plant flow increases to 5.5 MGD and additional treatment becomes necessary due to future permit limits for total nitrogen and total phosphorus.

### References

Metcalf & Eddy, Inc, 2003, *Wastewater Engineering: Treatment and Reuse*, 4<sup>th</sup> Edition.

Water Environmental Federation, 1998, *Biological and Chemical Systems for Nutrient Removal*.

Water Environmental Federation, 2010, *Design of Municipal Wastewater Treatment Plants* (WEF Manual of Practice No. 8), 5<sup>th</sup> Edition.

United States EPA Office of Water Program Operations, 1976, *Anaerobic Sludge Digestion Operations Manual*, EPA 430/9-76-001.

CEM:

MBP2009 01H

Path & File Identification –Mandatory

**Attachment E: Union Township, Estimation of  
Solids Removed**

### Union Township Project, Estimation of Solids Removed

Assumptions = 24 homes removed from system

Flow from each household (daily average), per Section 73.17 of the PA Code for a single family residence  
= 400 gallons/day

Typical solids in raw wastewater from residences = 200 mg/L

#### ***Solids removal calculation***

Total flow =  $24 * 400 = 9600$  gal/day or 0.0096 MGD

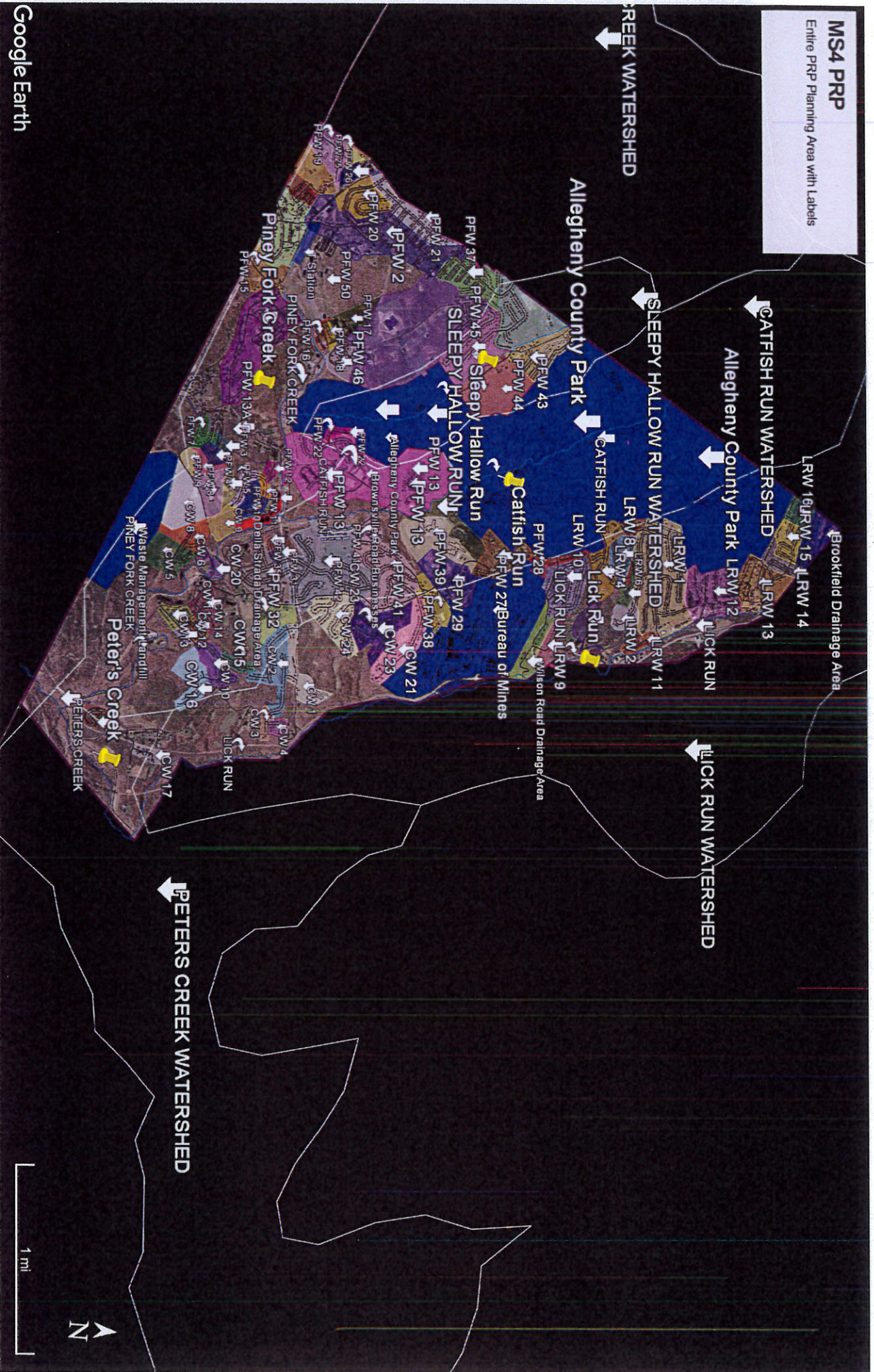
*Solids removed (lbs/day) =  $200 \text{ mg/L} * 0.0096 \text{ MGD} * 8.34 = 16 \text{ lbs/day}$*

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**Attachment F: PRP Map, Planning Area and  
Existing BMPs**

# MS4 PRP

Entire PRP Planning Area with Labels

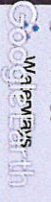


# MS4 PRP

Southwest Corner of South Park (Zone 1)

**Legend**

- Impervious Areas
- Parceled Areas
- Saddlebrook Drainage Area
- Middle School Drainage Area
- PFW 1
- PFW 2
- PFW 13
- PFW 13A
- PFW 14
- PFW 15
- PFW 16
- PFW 17
- PFW 18
- PFW 19
- PFW 20
- PFW 21
- PFW 22
- PFW 25
- PFW 26
- PFW 37
- PFW 44
- PFW 45
- PFW 46
- PFW 48
- PFW 49
- PFW 50





**MS4 PRP**  
Southeast Corner of South Park (Zone 2)

Legend	
	Impervious Areas
	Paved Areas
	Piney Fork Treatment
	Waterways
	CW 1
	CW 2
	CW 3
	CW 4
	CW 5
	CW 6
	CW 7
	CW 8
	CW 9
	CW 10
	CW 11
	CW 12
	CW 13
	CW 14
	CW 15
	CW 16
	CW 17
	CW 18
	CW 19
	CW 20
	CW 21
	CW 22
	CW 23
	CW 24
	Della Strada Drainage Area
	PFW 3
	PFW 4
	PFW 5
	PFW 6
	PFW 7
	PFW 8
	PFW 9
	PFW 10
	PFW 11
	PFW 12
	PFW 13
	PFW 31
	PFW 32
	PFW 35
	PFW 41
	PFW 42





**MS4 PRP**  
 Northeast Corner of South Park (Zone 3)



Legend	
	Impervious Areas
	Paved Areas
	PRP Planning Area
	Waterways
	CW 21
	CW 23
	High School Stadium Drainage Area
	L.R.W. 10 Drainage Area
	L.R.W. 9 Drainage Area
	LRW 3
	LRW 4
	PFW 27
	PFW 29
	PFW 38
	PFW 39
	PFW 41
	PFW 43
	PFW 44
	PFW 45
	Wilson Road Drainage Area

3000 ft

Google Earth

**MS4 PRP**  
NorthWest Corner of South Park (Zone 4)



**Legend**

Impervious Areas	Impervious Areas
Parcels Areas	Parcels Areas
PRP Planning Area	PRP Planning Area
Waterways	Waterways
Brookfield Drainage Area	Brookfield Drainage Area
LRRW 1	LRRW 1
LRRW 2	LRRW 2
LRRW 3	LRRW 3
LRRW 4	LRRW 4
LRRW 5	LRRW 5
LRRW 7	LRRW 7
LRRW 8	LRRW 8
LRRW 11	LRRW 11
LRRW 12	LRRW 12
LRRW 13	LRRW 13
LRRW 14	LRRW 14
LRRW 15	LRRW 15
LRRW 16	LRRW 16

Google Earth

3000 ft

