Master Plan
Water Reclamation Facility
City of Sioux Falls, South Dakota

UEI Project No. 407.013

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

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EXECUTIVE SUMMARY

The City of Sioux Falls retained Ulteig Engineers to prepare a Master Plan for the City's Water Reclamation Facility (WRF). The purpose of the project was to establish a formal capacity rating for the existing plant and develop a schedule for improvements to increase capacity when needed and to replace the systems that have reached their useful life.

The City of Sioux Falls WRF consists of mechanical fine screens, grit removal, primary clarification, first stage trickling filters, first stage intermediate clarifiers, second stage trickling filters, second stage intermediate clarifiers, activated sludge, final clarification, effluent filtration, chlorination and effluent cascade aeration. The facility discharges treated effluent to the Big Sioux River. The solids portion of the facility consists of gravity thickeners, anaerobic digesters and facultative sludge basins. Stabilized biosolids are land applied. The rated capacity for the original facility is shown in the following table.

Original Sioux Falls WRF Capacity

Parameter	Value	
Average Daily Flow	13.43	mgd
Peak Instantaneous Flow	27.00	mgd
BOD Loading (avg)	48,443	lb/d
TSS Loading (avg)	34,059	lb/d
TKN Loading (avg)	5,419	lb/d

The Facility Re-Rate Report reviewed the facility design and performance information and recommended a rated capacity. The rated capacity was presented to the South Dakota Department of Environment and Natural Resources (SD DENR) for review. The City, Ulteig and the SD DENR collaboratively determined a true and formal capacity of the WRF. The Facility Capacity Re-Rate Report is included in the Appendix of this Document. The new capacity of the Sioux Falls WRF as determined in the Re-Rate Report is summarized in the following table.

Re-Rated Sioux Falls WRF Capacity

Parameter	Value
Average Daily Flow	21.0 mgd
Peak Hourly Flow	35.0 mgd
TBOD	51,240 lb/d
TSS	43,900 lb/d
TKN	9,440 lb/d

Based on the new capacity from the Re-rate Report, a Water Reclamation Facility Master Plan was prepared to guide the City in their capital improvements planning for future facility needs. The future needs were determined after a capacity analysis, future flow projections and a condition assessment of equipment.

The historical flow and loading trends to the Water Reclamation Facility were determined. A summary of the average and maximum month flow and loadings for the years 2000-2007 is shown in the following table.

Historical Influent Flows and Loadings at WRF

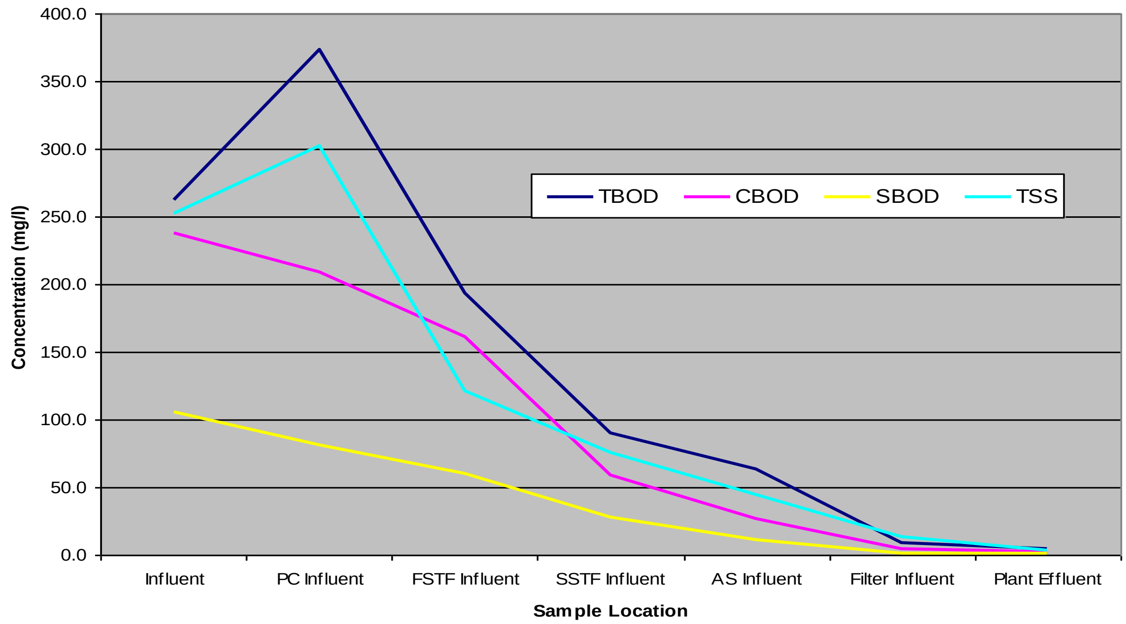
Parameter	2000	2001	2002	2003	2004	2005	2006	2007 Sep-Dec
Flow								
Average (mgd)	13.56	14.65	13.97	13.63	14.57	14.45	14.92	15.37
Max Month (mgd)	14.98	19.61	15.66	15.05	21.02	18.87	19.88	16.50
BOD								
Average (lb/d)	21,481	22,315	26,108	26,636	27,558	30,254	30,163	34,874
Max Month (lb/d)	23,647	29,163	29,308	29,681	29,433	37,843	33,738	38,208
TSS								
Average (lb/d)	24,318	24,188	24,499	24,442	25,913	27,239	27,335	31,711
Max Month (lb/d)	26,678	28,608	28,074	26,908	32,503	34,144	33,390	33,532
NH_3-N								
Average (lb/d)	2,379	1,877	1,882	2,234	2,488	2,831	2,790	3,054
Max Month (lb/d)	2,764	2,542	2,135	2,446	2,932	3,051	3,125	3,165
TKN								
Average (lb/d)	4,285	3,738	4,232	4,113	4,526	4,973	5,227	5,915
Max Month (lb/d)	4,874	4,112	5,338	4,619	5,463	5,818	5,687	6,662

Performance

The WRF data was analyzed to determine both the individual unit process performance and the overall facility performance. The WRF personnel take a number in intermediate samples throughout the facility.

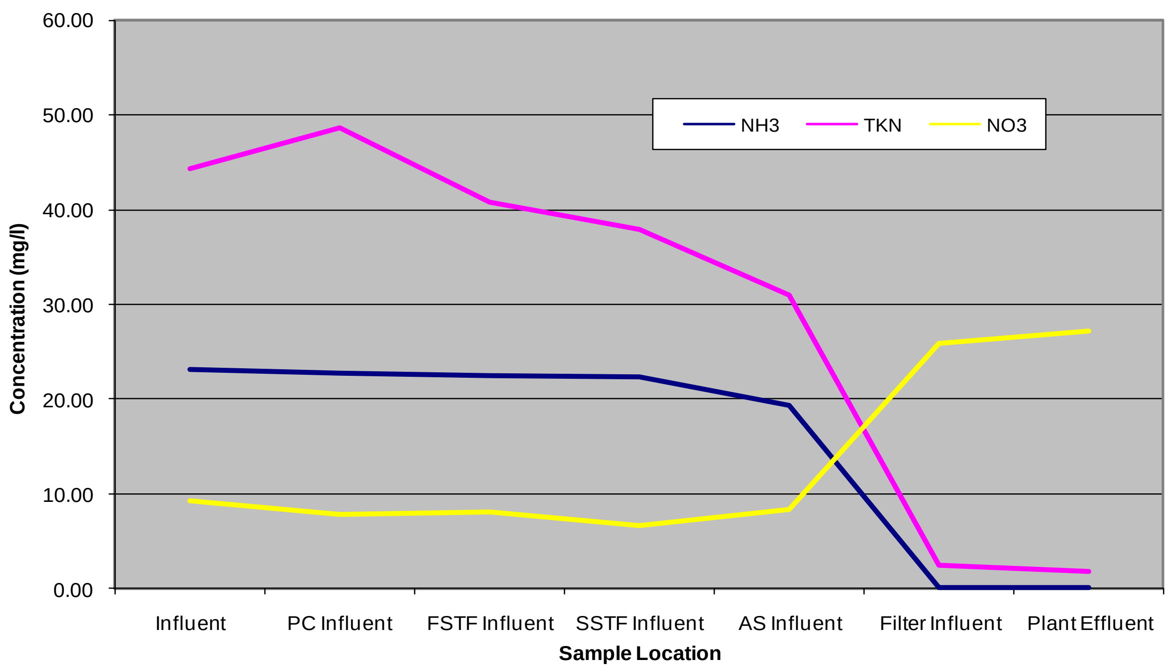
The graph on the following page shows the total BOD (TBOD), carbonaceous BOD (CBOD), soluble BOD (SBOD) and TSS reduction throughout the facility. The spike in TBOD and TSS at the primary clarifier influent is caused by the in plant waste addition. The facility unit processes substantially reduces the BOD and solids loading throughout the plant.

Unit Process Performance – BOD and TSS



The graph below shows the ammonia and TKN reduction through the facility unit processes. As shown in the graph, the ammonia and TKN reduction occurs primarily in the activated sludge process, which was the original design intent. Nitrate concentrations rise following the activated sludge process, which is the expected result of organic nitrogen and ammonia oxidation.

Unit Process Performance – NH3, TKN and NO3



The overall facility has had impressive removal efficiencies throughout the years. The 2006 and 2007 removal efficiencies of the entire facility are as follows:

Parameter	2006	2007
BOD	98.4%	98.5%
TSS	98.8%	98.4%
Ammonia	99.0%	99.7%
TKN	96.5%	96.0%

Equipment Evaluation

The Master Plan also included a condition assessment of the various process equipment at the Sioux Falls WRF. The major process equipment identified by City Personnel in the request for proposals was evaluated based on condition, performance, existing capacity, opportunity for optimization and redundancy. A summary of the equipment evaluation findings detailed throughout this report include:

- Brandon Road Pump Station and Forcemain – Pumps do not perform well at average flows. Forcemain lacks redundancy. (See Sections 6.1-6.2)
- Grit removal – Equipment is in adequate condition. Amount of grit in digesters may show grit removal at the front of the plant could be improved. (See Section 6.3)
- Primary Clarifiers – Equipment is in good condition. (See Section 6.4)
- Trickling Filter Distributor Arms – First and Second Stage Trickling Filter distributor arms have significant corrosion that limits the capacity of the processes. (See Section 6.5)
- Intermediate Clarifiers – Equipment is in good condition (See Section 6.6)
- Lime Feed System – The existing lime slaking and pumping system is in poor condition. Upgrades to the existing lime feed system are recommended. (See Section 6.7)
- Aeration System/Activated Sludge - The aeration system valves are in poor condition and need replacement. Switching to Fine Bubble Aeration appears to be cost effective in the long term. (See Section 6.8)
- RAS Pumps – Pumps appear to be operating too far on the right side of the pump curve according to existing gauge readings. A check of pump hydraulics is recommended. (See Section 6.9)
- Final Clarifier Mechanisms – The corrosive environment has caused significant damage to the final clarifier mechanisms. (See Section 6.10)
- Gravity Filter Replacement – Replacement of the existing Wheeler bottom gravity filters with a more applicable filter technology is recommended. (See Section 6.11)
- Disinfection – Ozonation, UV disinfection and on-site sodium hypochlorite generation are more expensive than utilizing current gaseous chlorine system. (See Section 6.12)

- Flow Measurement – At peak flows, the effluent weir is submerged, causing inaccurate flow measurement. (See Section 6.13)
- Sampling – New remote sampling buildings are recommended to achieve accurate process samples. (See Section 6.14)
- Emerging Contaminants – There are a number of technologies being piloted across the country for removal of endocrine disrupting chemicals (EDCs). There are currently no regulations for EDCs in plant effluent but there may be in the future. (See Section 6.15)
- Gravity Thickener Mechanism Rehabilitation – The Gravity Thickener Mechanisms have deteriorated over time and need immediate rehabilitation. Long term improvements include replacement. (See Section 6.16)
- Digester Mixing System – The existing mixing system is not operated and grit deposition in the digesters is an operational issue. A new mixing system to keep grit in suspension is recommended. (See Section 6.17)
- Sludge Pumping – The existing recirculation pumps and piping should be reconfigured to replace the existing transfer pumps.

Future Conditions

The City's projected population and industrial growth were evaluated to determine future flow and loadings to the WRF. The average daily flow, average dry day flow, peak month flow, peak hourly flow, peak instantaneous flow, TBOD loading, TSS loading and TKN loading for the year 2030 were determined. The following table shows the future average daily flow, TBOD loading, TSS loading, Ammonia loading and TKN loading for the Sioux Falls WRF.

WRF Projected Flow and Loading

Year	Population	ADF (mgd)	BOD (lb/d)	TSS (lb/d)	NH3-N (lb/d/cap)	TKN (lb/d/cap)
2007	144,000	15.26	31,680	31,680	2,880	5,760
2010	148,500	15.74	32,670	32,670	2,970	5,940
2015	156,000	16.54	34,320	34,320	3,120	6,240
2020	170,500	18.07	37,510	37,510	3,410	6,820
2025	185,000	19.61	40,700	40,700	3,700	7,400
2030	199,600	21.16(1)	43,912	43,912	3,992	7,984
Projection Value(2)		**106 gal/cap/d**	**0.22 lb/cap/d**	**0.22 lb/cap/d**	**0.02 lb/cap/d**	**0.04 lb/cap/d**

(1) The projected flow or load is outside of proposed re-rate capacity (21.0 mgd).
(2) Projection Value was determined in Table 3 from existing WWTF data.

The average daily dry weather flow, average daily flow, peak month flow, peak hourly wet weather flow, and peak instantaneous flows were projected and are shown in the following table.

WRF Projected Flows

Flow Parameter	2010	2015	2020	2025	2030
Average Daily Dry (mgd)	14.46	15.50	17.13	18.76	20.50
Average Daily (mgd)	15.74	16.54	18.07	19.61	21.16
Peak Month (mgd)	20.88	22.03	23.88	25.73	27.69
Peak Hourly (mgd)	38.22	40.90	45.32	49.75	54.40
Peak Instantaneous (mgd)	43.48	46.16	50.58	55.01	59.66

Alternatives to Meet Future Conditions

The existing WRF is well positioned at this time to handle ***average annual daily flows*** (ADF) now and into the future. However, during wet weather conditions the WRF is unable to maintain the recommended standards for weir loading, surface overflow rates and detention times in the various sedimentation tanks (primary, intermediate and final clarifiers). Also, field testing and modeling of various facilities at the WRF have revealed hydraulic limitations during high flows. Some of the facilities that show restrictive tendencies at higher flows include the first and second stage trickling filter distributor arms, process pumping station and the effluent filters. Even though the recommended standards cannot be met during peak flow events and hydraulic restrictions occur, it must be noted the WRF has not violated its NPDES Permit requirements. The Re-rate Document further explains the current peak flow capacity (35.0 mgd) determination.

To handle peak wet weather flows at the WRF today and into the future, various alternatives were evaluated. Two basic conceptual alternatives were evaluated to allow the WRF to handle projected peak flows:

- Construct equalization storage which would maintain peak flows to the WRF at the current capacity (35 mgd) or

- Expand unit processes throughout the plant to meet projected flows.

Each conceptual alternative included variations for location, phasing, and ancillary improvements necessary.

The ***equalization storage alternative*** was evaluated for a phased project completion. The existing 12 million gallon equalization basin would continue to be used in conjunction with a new 6 million gallon basin constructed as soon as possible. The existing EQ basin used with the new basin would be able to handle peak flows to approximately the year 2020. At that point, additional flow equalization would be necessary. It is projected that an additional 12 million gallons would be necessary by the year 2020 to reach the anticipated 2030 design flows.

There are two main options for location of a new equalization basin: at the WRF or at the existing equalization basin site. Based on the overwhelming benefit of prolonging the upgrades to the Brandon Road Pump Station, it is recommended that the Phase 1 equalization storage of 6 million gallons be constructed at the existing EQ basin site. Preliminary estimated costs for a 6 million gallons EQ basin are $8,700,000. Phase 2 equalization should be located at the WRF, which would take advantage of the screening and grit removal processes before storage. Preliminary estimated costs for a 12 million gallon EQ basin are $19,180,000.

By constructing equalization storage upstream of Brandon Road Pump Station, the life of the station can be extended to approximately 2015-2020, depending on city growth. At that time, upgrades to the pump station and the forcemain will be necessary. It is estimated the lift station upgrades would cost approximately $4,400,000. As previously mentioned, the Brandon Road Force Main lacks redundancy. Also, pipe velocities in the force main begin to exceed recommended standards at projected future flows. A second force main would decrease frictional loss and increase the capacity of the Brandon Road Pump Station. It is estimated a second forcemain would cost approximately $8,060,000.

The second conceptual alternative to meet future anticipated peak flows is the possibility of ***adding additional process components***. As previously mentioned, the facility has adequate average day flow capacity but limited peak flow capacity. The following unit process capacities are exceeded at the projected peak hourly 2030 design flow of 54.40 mgd.

- Brandon Road Pump Station (35.0 mgd)
- Primary Clarifiers (30.5 mgd)
- First Stage Trickling Filter Distributor Arms (27.0 mgd)
- First Stage Intermediate Clarifiers (25.9 mgd)
- Second Stage Trickling Filter Distributor Arms (30.0 mgd)
- Second Stage Intermediate Clarifiers (25.9 mgd)
- Process Pump Station (31.3 mgd)
- Final Clarifiers (31.8 mgd)
- Effluent Filters (33.3 mgd)
- Chlorine Contact Tank (41.0 mgd)

The estimated costs to expand the individual unit processes are $39,100,000. ***The recommended alternative for the Water Reclamation Facility to meet the City of Sioux Fall's wastewater treatment needs to the year 2030 is to construct equalization storage in two phases***. The first phase would be located at the existing equalization basin site consisting of approximately 6 million gallons of storage. The second phase would be located at the WRF and would be 12 million gallons.

Capital Improvements Plan

All major process equipment identified by City Personnel in the request for proposals and additional processes requested by facility personnel have been evaluated based on condition, performance, existing capacity, opportunity for optimization and redundancy. Based on the recommendations if this analysis, a capital improvements plan (CIP) was developed for City planning purposes. The CIP contains budgeted equipment costs for replacement due to age, condition of equipment and to meet the objectives for expansion of the plant due to future growth of Sioux Falls and the surrounding area. A summary of the recommendations due to capacity or condition issues detailed throughout this report include:

- Trickling Filter Distributor Arms – First and Second Stage Trickling Filter distributor arms have significant corrosion that limits the capacity of the processes. (See Section 6.5)
- Final Clarifier Mechanisms – The corrosive environment has caused significant damage to the final clarifier mechanisms. (See Section 6.10)
- Gravity Thickener Mechanism Rehabilitation – The Gravity Thickener Mechanisms have deteriorated over time and need immediate rehabilitation. Long term improvements include replacement. (See Section 6.16)
- Digester Mixing System – The existing mixing system is not operated and grit deposition in the digesters is an operational issue. A new mixing system to keep grit in suspension is recommended. (See Section 6.17)
- Gravity Filter Replacement – Replacement of the existing Wheeler bottom gravity filters with a more applicable filter technology. (See Section 6.11)
- Lime Feed System – The existing lime slaking and pumping system is in poor condition. Upgrades to the existing lime feed system are recommended. (See Section 6.7)
- Aeration System - The aeration system valves are in poor condition and need replacement. Switching to Fine Bubble Aeration appears to be cost effective in the long term. (See Section 6.8)
- Flow Measurement – At peak flows, the effluent flume is submerged, causing inaccurate flow measurement. (See Section 6.13)
- Brandon Road Pump Station and Forcemain – Capacity for future projected flows require upgrades in the long term. (See Sections 6.1-6.2)
- Sludge Pumping – Reconfigure sludge recirculation pumps for sludge transfer and remove existing sludge transfer pumps.
- Equalization Basins – Construct new equalization basins as a phased approach to accommodate future peak flows. Phase 1 equalization of 6 million gallons at the existing EQ basin site; Phase 2 equalization of 12 million gallons at the WRF.

These recommendations for improvements were separated into short range and long range needs. The table on the following page shows the recommended improvements with preliminary cost estimates for planning purposes.

Short Range and Long Range Improvements

Short Range Improvements: 2009 - 2013	
1. Trickling Filter Distributor Arms (4 FSTF @ 135')	$ 1,270,000
2. Trickling Filter Distributor Arms (4 SSTF @ 145')	$ 1,330,000
3. Final Clarifier Mechanisms (4)	$ 2,060,000
4. Gravity Thickener Mechanism Rehabilitation (2)	$ 100,000
5. Digester Mixing System	$ 480,000
6. Gravity Filter Replacement	$ 2,960,000
7. Hydrated Lime System	$ 830,000
8. Aeration System Valve Replacement	$ 140,000
9. 2010-2020 Flow Equalization Storage Basin	$ 8,700,000
Subtotal	**$17,900,000**
Long Range Improvements: 2014 - 2030	
10. Fine Bubble Aeration for 3 Basins and Blowers	$ 1,610,000
11. Flow Measurement	$ 50,000
12. Gravity Thickener Mechanisms Replacement	$ 390,000
13. Emerging Contaminants	$ 18,870,000
14. Brandon Road Pump Station Improvements	$ 4,400,000
15. Brandon Road Force Main	$ 8,060,000
16. Digester Pumping System Modifications	$ 60,000
17. 2020-2030 Flow Equalization Storage Basin	$ 19,180,000
Subtotal	**$52,620,000**

1.0 INTRODUCTION

1.1 Background

The City of Sioux Falls Water Reclamation Facility (WRF) was constructed in three phases from 1980 through 1986. The first phase involved the construction of the tertiary treatment processes to bring the City into compliance with new water quality standards set by the EPA for the Big Sioux River. The second phase included the construction of the solids handling facilities along with the administration and maintenance buildings. The third phase provided the primary and secondary treatment processes.

The City has been meeting effluent limits since the facility was fully operational in 1986; however, current wastewater flows exceed the original design capacity while organic and nitrogen loadings are below design capacity.

The City retained Ulteig Engineers to study the facility design and performance information and recommend a rated capacity. The rated capacity was presented to the South Dakota Department of Environment and Natural Resources (SD DENR) for review. The City, Ulteig and the SD DENR collaboratively determined a true and formal capacity of the WRF. The Facility Capacity Re-Rate Report is included in the Appendix.

Based on the new capacity from the Re-rate Report, a Water Reclamation Facility Master Plan has been prepared to guide the City in their planning efforts for future facility needs. The future needs were determined after a capacity analysis, future flow projections and a condition assessment of equipment.

1.2 Purpose and Scope

The goal of the Water Reclamation Facility Master Plan is to establish a formal plant capacity rating (Re-Rate Report) and develop a schedule for improvements at the plant to increase capacity when needed and replace systems that have reached their useful life.

The historical flow and loading trends along with the City's projected population and industrial growth were evaluated to determine future flow and loadings to the WRF. The average daily flow, average dry day flow, peak month flow, peak hourly flow, peak instantaneous flow, TBOD loading, TSS loading and TKN loading for the year 2030 were determined. Options for improvements necessary to meet the future conditions were evaluated.

The Master Plan also includes a condition assessment of the various process equipment at the Sioux Falls WRF. The major process equipment identified by City Personnel in the request for proposals was evaluated based on condition, performance, existing capacity, opportunity for optimization and redundancy.

A capital improvements plan (CIP) was developed for City planning purposes. The CIP contains budgeted equipment costs for replacement due to age, condition of equipment and to meet the objectives for expansion of the plant due to future growth of Sioux Falls and the surrounding area.

1.3 Abbreviations and Terms

BOD	5 Day Biochemical Oxygen Demand
TBOD	Total Biochemical Oxygen Demand (CBOD + NBOD)
CBOD	Carbonaceous Biochemical Oxygen Demand
NBOD	Nitrogenous Biochemical Oxygen Demand
SBOD	Soluble Biochemical Oxygen Demand
COD	Chemical Oxygen Demand
DO	Dissolved Oxygen
NH_3-N	Ammonia Nitrogen
NO_3-N	Nitrate Nitrogen
TSS	Total Suspended Solids
VSS	Volatile Suspended Solids
H_2S	Hydrogen Sulfide
DHS	Dissolved Hydrogen Sulfide
$FeCl_2$	Ferrous Chloride
H_2O_2	Hydrogen Peroxide
CO	Carbon Monoxide
O_2	Oxygen
LEL	Combustible Gas
WWTF	Wastewater Treatment Facility
EPA	Environmental Protection Agency
NPDES	National Pollution Discharge Elimination System
mg/L	Milligrams per Liter
ppm	Part per Million
gal	Gallon
gph	Gallons per Hour
gpd	Gallons per Day
mgd	Million Gallons per Day
ft	Feet
sf	Square Feet
cf	Cubic Feet
lb/d	Pounds per Day
WRF	Water Reclamation Facility
kW	Kilowatt
BTU	British Thermal Unit
I/I	Infiltration and/or Inflow
PVC	Polyvinyl Chloride
RCP	Reinforced Concrete Pipe
DIP	Ductile Iron Pipe

A number of flow conditions are considered in the design and evaluation of wastewater treatment facilities. The following flows are used in this study of the Sioux Falls WRF.

- *Average Daily Flow* – The average daily flow is the average of the daily volumes received for a continuous 12 month period expressed as a volume per unit time (million gallons per day or mgd).

- *Maximum Day Flow* – The maximum day flow is the largest volume of flow received during a continuous 24 hour period expressed as a volume per unit time (mgd).

- *Maximum Month Flow* – The maximum month flow is the largest volume of flow received during a month long period expressed as a volume per unit time (mgd).

- *Peak Hourly Flow* – The peak hourly flow is the largest volume of flow to be received during a one hour period expressed as a volume per unit time (mgd).

- *Peak Instantaneous Flow* – The peak instantaneous flow is the instantaneous maximum flow rate received as a volume per unit time (mgd).

2.0 EXISTING WASTEWATER TREATMENT AND CONVEYANCE FACILITIES

The City of Sioux Falls has a flow equalization basin that dampens diurnal flow to the treatment facility. The equalization basin is located at the old wastewater treatment facility site and has approximately 12 million gallons of storage. The basin is upstream of the Brandon Road Pump Station.

The Brandon Road Pump Station conveys the majority of the domestic and industrial wastewater from the City of Sioux Falls to the Water Reclamation Facility (WRF). At the WRF, rotary fine screens installed in 2007 pre-treat the wastewater to remove large materials that could damage or plug equipment. The screenings removed are washed and pressed before disposal. After screening, the wastewater enters an aerated grit chamber where sand and gravel are removed to minimize wear on equipment. Grit removed from the wastewater is washed and dewatered before disposal in the landfill.

The facility has four Primary Clarifiers to remove settable solids and scum from the wastewater. The Primary Clarifiers are 90 feet in diameter and have 8-feet of sidewater depth. The Clarifiers are center-feed with peripheral weirs. Settled solids (sludge) and scum are collected by a rotating arm and are pumped to the solids handling units.

Secondary treatment is accomplished by two stages of Trickling Filters. The four First Stage Trickling Filters are 135 feet in diameter and are 7 feet deep. The four Second Stage Trickling Filters are 145 feet in diameter and are 7 feet deep. The Trickling Filters contain Sioux Quartzite Rock media, distributor arms and an underdrain system. The microorganisms on the media of the Trickling Filters remove pollutants in the waste stream. Each stage of Trickling Filters is followed by two 105 foot diameter Intermediate Clarifiers with side water depths of 10 feet. The Intermediate Clarifiers remove biomass that sloughs off of the Trickling Filter media by gravity settling.

Tertiary treatment for ammonia removal is accomplished in the Activated Sludge System. Microorganisms or “activated sludge” in the basin is mixed with the wastewater from the secondary treatment process. Coarse bubble diffusers supply air to provide oxygen to the microorganisms and mixing. In the aerobic environment, the microorganisms nitrify ammonia to allow the facility to meet permit requirements. The effluent from the Aeration Basins flows into four 100-foot diameter Final Clarifiers with side water depths of 14 feet. The Final Clarifiers settle solids from the treated wastewater. The sludge from the Final Clarifiers is returned to the activated sludge process or wasted.

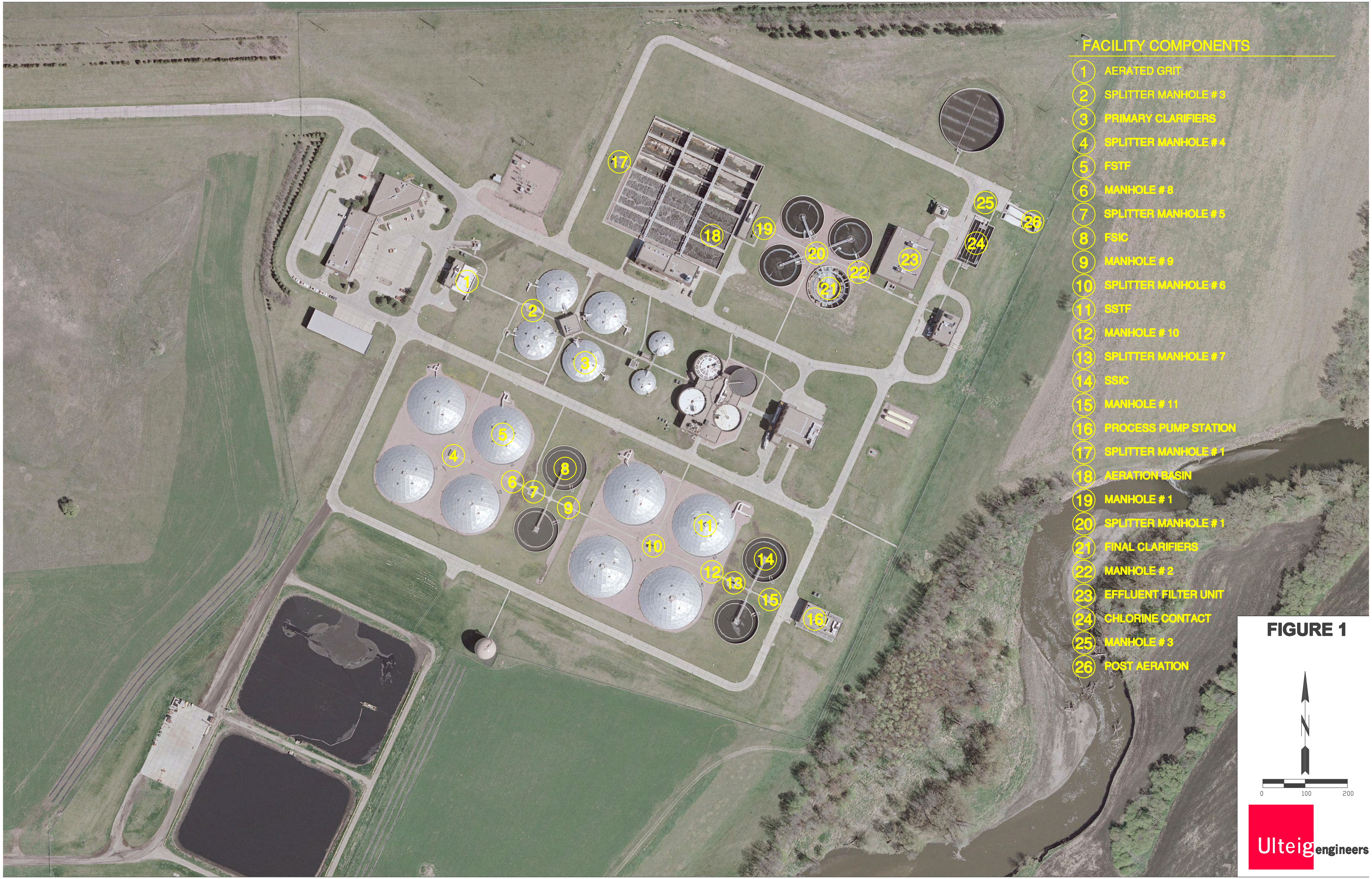
Effluent from the Final Clarifier flows to the Effluent Filter Unit for final polishing. Eight dual-media gravity filters 34 feet by 17 feet by 8 feet deep further remove any pollutants remaining in the water to ensure compliance with permit requirements. Filtered water flows to the Chlorine Contact Basin where chlorine is added for disinfection. Residual chlorine in the wastewater is removed by sulfur dioxide. A Cascade Aeration Unit increases the dissolved oxygen in the water before final discharge to the Big Sioux River.

The design influent conditions of the facility are shown in Table 1 below.

Table 1 Facility Influent Design Conditions

Parameter	Value	
Average Daily Flow	13.43	mgd
Peak Instantaneous Flow	27.00	mgd
BOD Loading (avg)	48,443	lb/d
TSS Loading (avg)	34,059	lb/d
TKN Loading (avg)	5,419	lb/d

Figure 1 on the following page shows the facility components on an aerial photo.


FACILITY COMPONENTS
1 AERATED GRIT
2 SPLITTER MANHOLE # 3
3 PRIMARY CLARIFIERS
4 SPLITTER MANHOLE # 4
5 FSTF
6 MANHOLE # 8
7 SPLITTER MANHOLE # 5
8 FSIC
9 MANHOLE # 9
10 SPLITTER MANHOLE # 6
11 SSTF
12 MANHOLE # 10
13 SPLITTER MANHOLE # 7
14 SSIC
15 MANHOLE # 11
16 PROCESS PUMP STATION
17 SPLITTER MANHOLE # 1
18 AERATION BASIN
19 MANHOLE # 1
20 SPLITTER MANHOLE # 1
21 FINAL CLARIFIERS
22 MANHOLE # 2
23 EFFLUENT FILTER UNIT
24 CHLORINE CONTACT
25 MANHOLE # 3
26 POST AERATION
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Ulteig engineers

FIGURE 1

3.0 HISTORICAL POPULATION, FLOW AND LOAD

3.1 Population

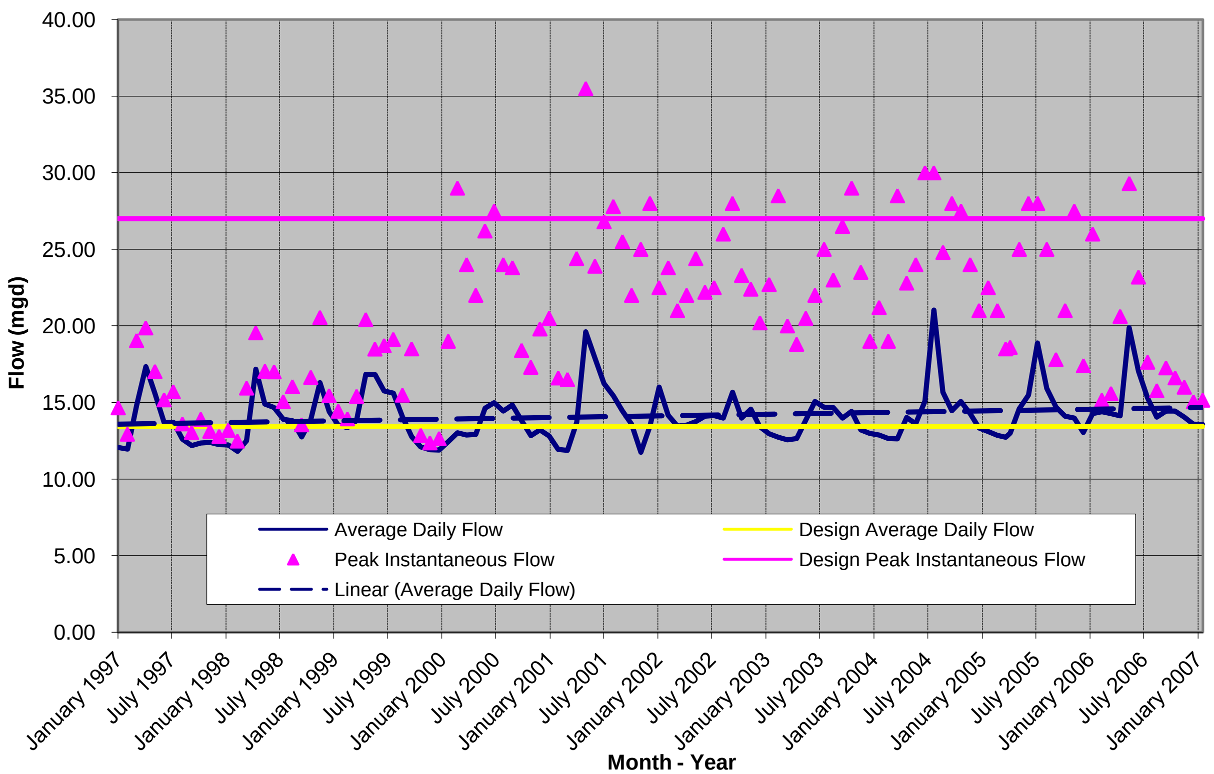
The flow and loading to the Sioux Falls WRF has been steadily increasing from 2000 to 2006. The following are the population figures from 2000-2006.

Year	Actual
2000	124,158
2001	127,530
2002	130,900
2003	134,260
2004	137,600
2005	141,000
2006	142,500

3.2 Flow and Loading

The daily flows for the WRF were compiled from 1997 through 2006. Influent flow from 1997 to 2006 is shown in Figure 2.

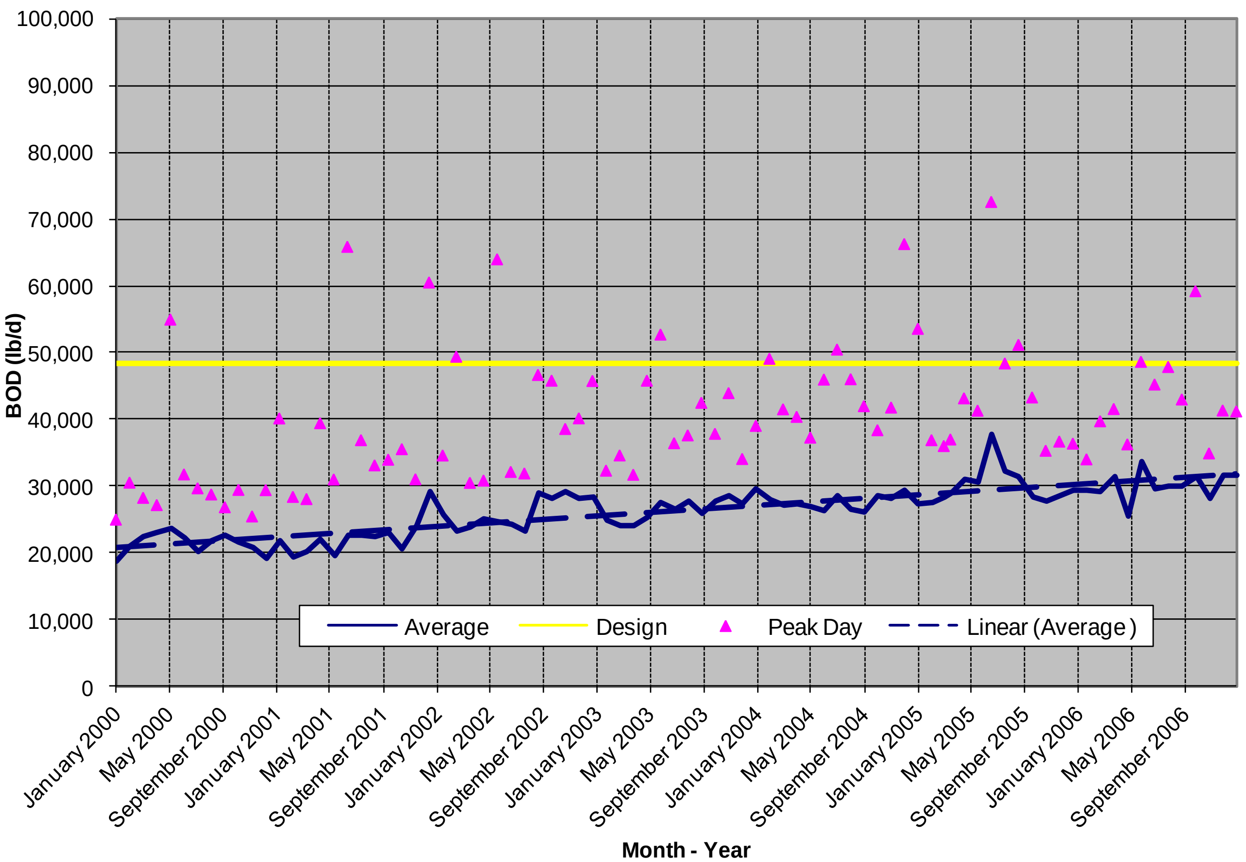
Figure 2 Influent Flow



The Average Daily Flow at the WRF has been consistently higher than the Design Average Daily Flow of 13.43 mgd. The flow trend has been increasing slightly over the past ten years, at approximately 157,000 gallons per day per year. Peak flows have been higher than the design Peak Hourly Flow of 27.0 mgd over the past 10 years. The peak instantaneous flow recorded for the studied time frame was 58.50 mgd on June 16, 2004. This value and the June 17, 2004 peak flow of 46.20 mgd were removed from Figure 2 for clarity but are noted as the peak instantaneous flow values of the facility.

The influent BOD loadings to the WRF were complied for the years of 2000 through 2006. The following figure shows the influent loadings at the WRF for the past 6 years.

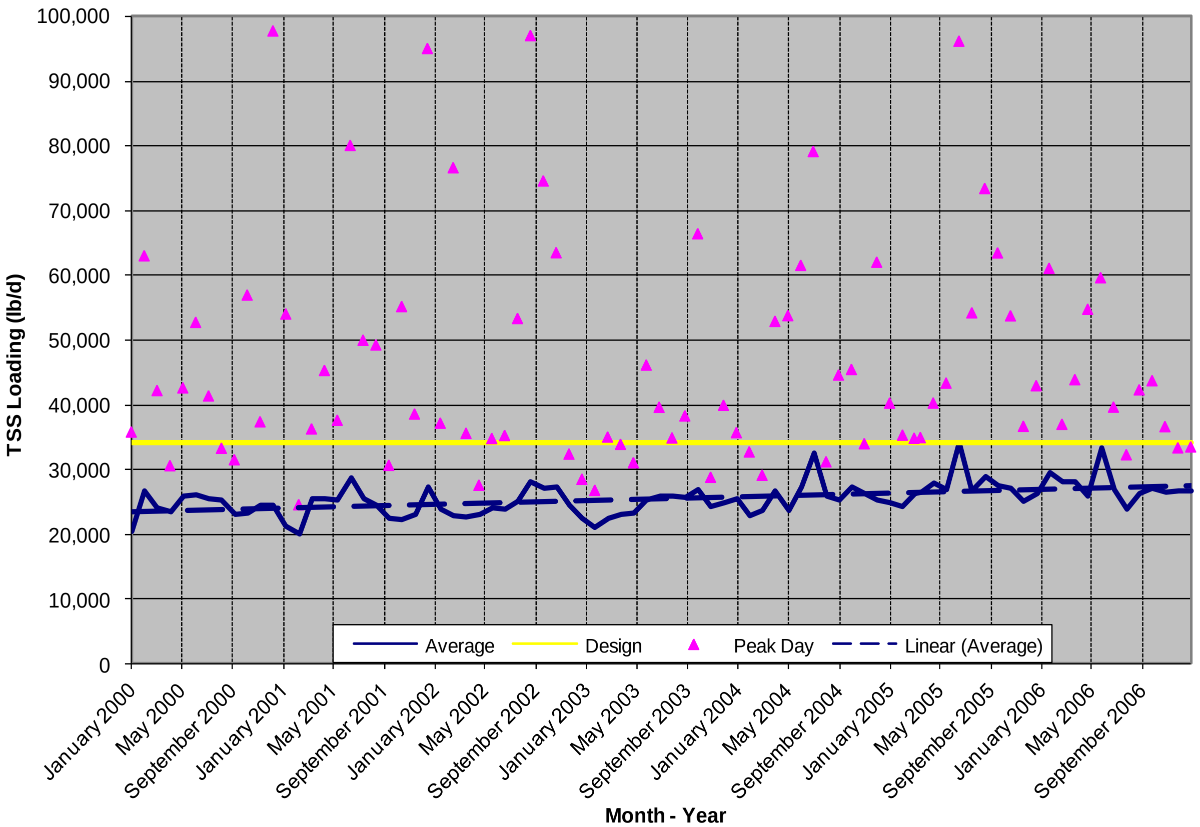
Figure 3 Influent BOD Loading



The average BOD loading to the WRF has been consistently below the Design Average Loading of 48,442 lb/d. The average 2006 BOD loading was 30,163 pounds per day. The loading trend has been steadily increasing at approximately 1,240 pounds per day per year based on the sampling data collected prior to 2007.

The influent TSS loadings to the WRF were complied for the years of 2000 through 2006. The following figure shows the influent loadings at the WRF for the past 6 years.

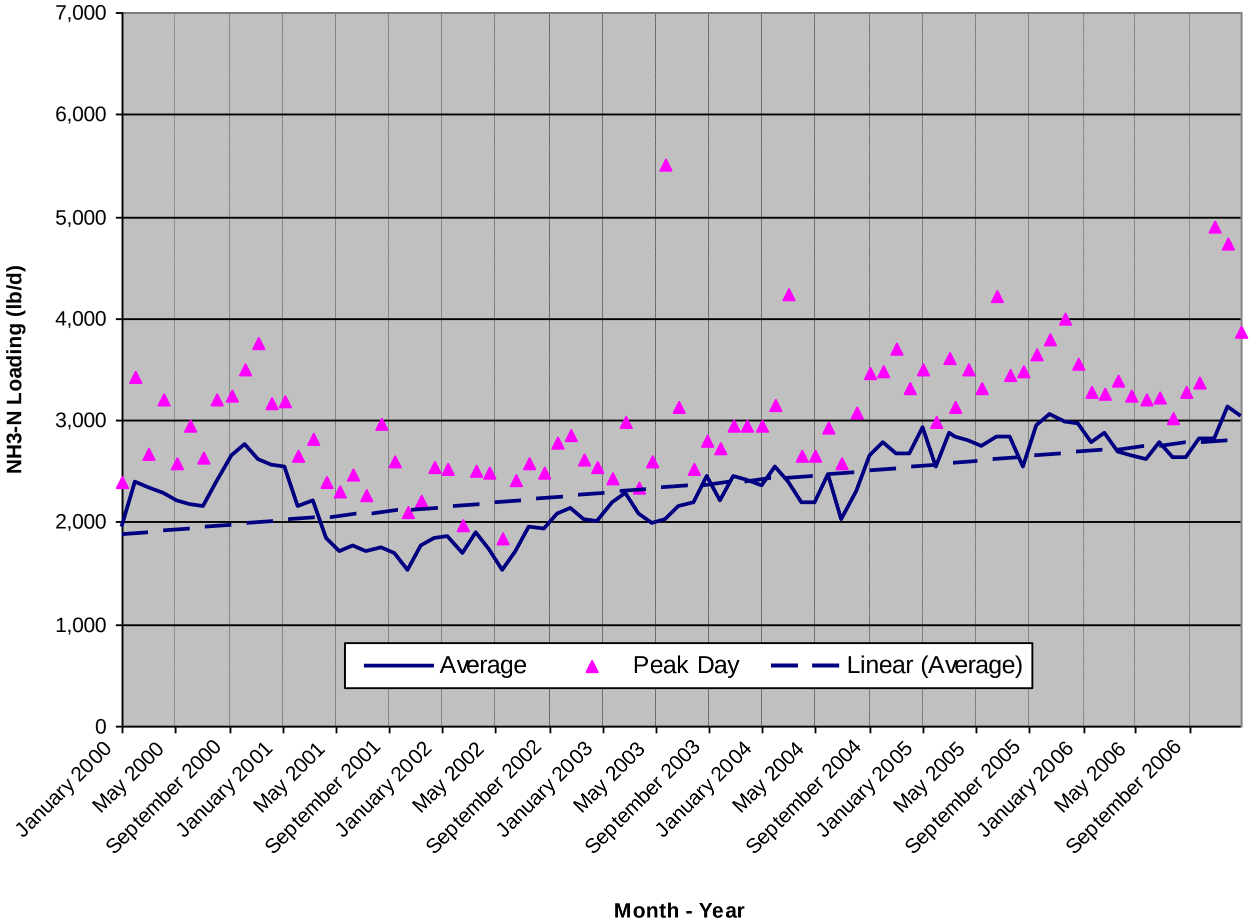
Figure 4 Influent TSS Loading



The Average Daily TSS loading to WRF has been consistently below the Design Average Daily Loading of 34,059 lb/d. The TSS loading trend has been increasing at approximately 431 pounds per day per year based on the sampling data collected prior to 2007.

The influent ammonia loadings to the WRF were complied for the years of 2000 through 2006. The following figure shows the influent loadings at the WRF for the past 6 years.

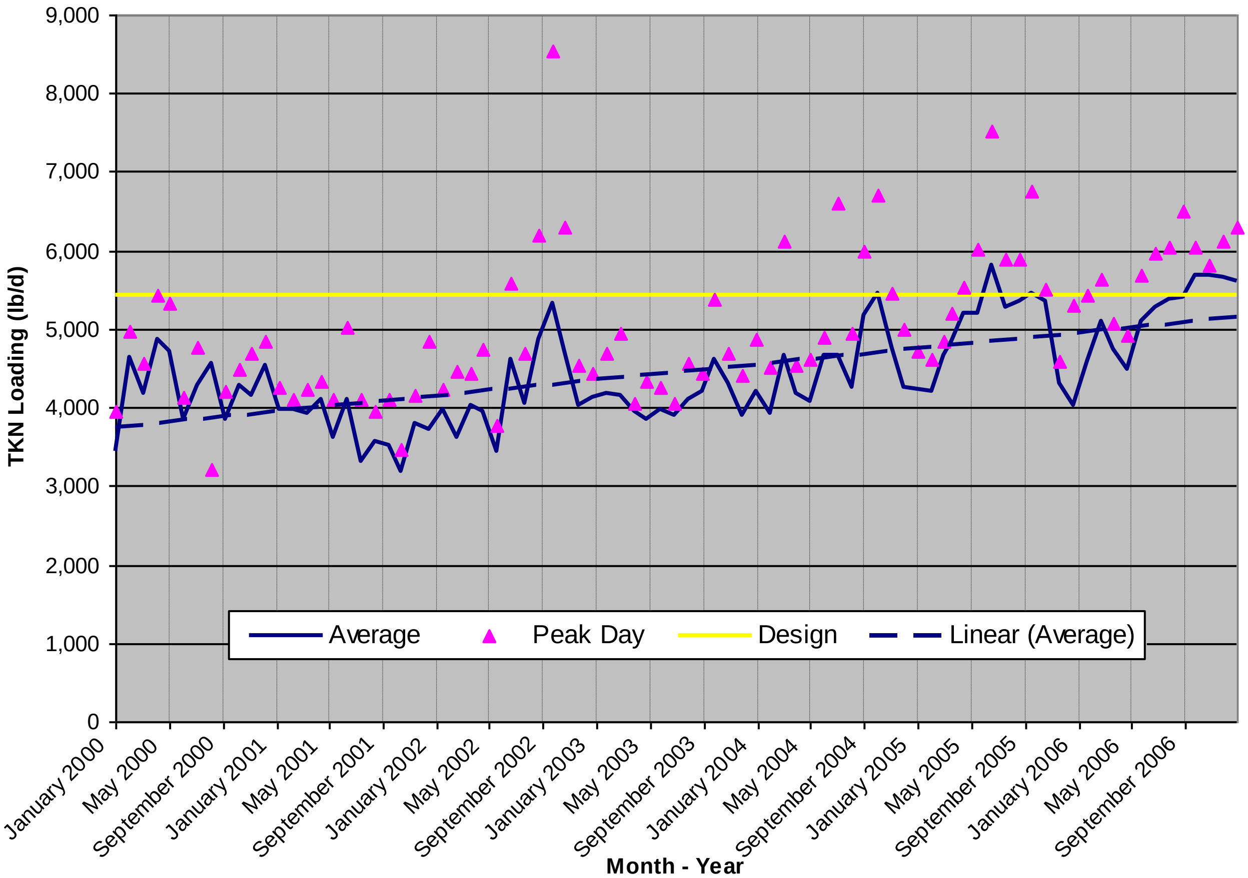
Figure 5 Influent NH_3-N Loading



The average daily ammonia loading to the facility has been steadily increasing at approximately 59 pounds per day per year over the past six years based on the sampling data collected prior to 2007.

The influent TKN loading to the WRF were compiled for the past six years. The following figure shows the influent TKN loading from 2000 through 2006.

Figure 6 Influent TKN Loading



The average daily TKN loading to the WRF has been under the average design loading of 5,419 most of the time. However, there have been some occurrences of the daily loading exceeding the design loading. The TKN loading to the facility has been steadily increasing at approximately 134 pounds per day per year over the past 6 years.

The flow, BOD loading, TSS loading, ammonia loading and TKN loading were compared to the design values for the years 2000 through 2006 and are shown in Table 2. Also, the new sampling data from September 2007 through December 2007 is shown.

Table 2 Influent and Design Conditions

Parameter	2000	2001	2002	2003	2004	2005	2006	2007 Sep-Dec
Flow								
Average (mgd)	13.56	14.65	13.97	13.63	14.57	14.45	14.92	15.37
Percent of Design	101%	109%	104%	102%	108%	108%	111%	114%
Max Month (mgd)	14.98	19.61	15.66	15.05	21.02	18.87	19.88	16.50
Max Month/Ave Day	1.10	1.34	1.12	1.10	1.44	1.31	1.33	
Peak Instantaneous (mgd)	29.00	35.50	28.00	29.00	58.50	28.00	29.30	
BOD								
Average (lb/d)	21,481	22,315	26,108	26,636	27,558	30,254	30,163	34,874
Percent of Design	44%	46%	54%	55%	57%	62%	62%	72%
Max Month (lb/d)	23,647	29,163	29,308	29,681	29,433	37,843	33,738	38,208
Max Day (lb/d)	54,959	65,869	63,996	52,711	66,282	72,592	59,215	66,112
TSS								
Average (lb/d)	24,318	24,188	24,499	24,442	25,913	27,239	27,335	31,711
Percent of Design	71%	71%	72%	72%	76%	80%	80%	93%
Max Month (lb/d)	26,678	28,608	28,074	26,908	32,503	34,144	33,390	33,532
Max Day (lb/d)	97,641	134,353	96,928	79,062	79,062	96,065	61,051	44,343
NH_3-N								
Average (lb/d)	2,379	1,877	1,882	2,234	2,488	2,831	2,790	3,054
Percent of Design	-	-	-	-	-	-	-	-
Max Month (lb/d)	2,764	2,542	2,135	2,446	2,932	3,051	3,125	3,165
Max Day (lb/d)	3,766	3,184	2,857	5,513	4,240	4,224	4,898	4,205
TKN								
Average (lb/d)	4,285	3,738	4,232	4,113	4,526	4,973	5,227	5,915
Percent of Design	79%	69%	78%	76%	84%	92%	96%	109%
Max Month (lb/d)	4,874	4,112	5,338	4,619	5,463	5,818	5,687	6,662
Max Day (lb/d)	5,421	5,015	8,548	5,383	6,717	7,518	6,512	8,085

The facility has been above the average daily design flow since 2000. The BOD, TSS and TKN loading has been below the design loading historically. However, the TKN loading was above the design loading, and the TSS loading was near the original design value based on the data collected from September through December 2007.

3.3 Per Capita Flow and Load

The existing influent flow and loading on a per capita basis were determined from the influent flow conditions and the corresponding population figures presented in Sections 3.1 and 3.2. These per capita flow and loading contributions were compared to typical values. The per capita contributions for average flow, BOD loading, TSS loading, NH3-N loading, and TKN loading are shown in the following table.

Table 3 Per Capita Flow and Loads

Year	Population	Flow (gpd/cap)	BOD (lb/d/cap)	TSS (lb/d/cap)	NH3-N (lb/d/cap)	TKN (lb/d/cap)
2000	124,158	109	0.17	0.20	-	0.034
2001	127,530	115	0.17	0.19	0.015	0.029
2002	130,900	107	0.20	0.19	0.014	0.032
2003	134,260	102	0.20	0.18	0.017	0.031
2004	137,600	106	0.20	0.19	0.018	0.033
2005	141,000	102	0.21	0.19	0.020	0.035
2006	142,500	105	0.21	0.19	0.020	0.037
Average	**-**	**106**	**0.19**	**0.19**	**0.015**	**0.033**
10 State Standards	-	100	0.17-0.22	0.20-0.25	-	-
Typical Range(1)	-	75-130	0.11 - 0.26	0.13 – 0.33	0.019	0.032
Projection Value(2)	**-**	**106**	**0.22**	**0.22**	**0.020**	**0.040**

(1) Metcalf and Eddy Wastewater Engineering, 2003
(2) Projection Value will be used for future treatment facility design.

The flow per capita since 2000 was 106 gallons per person per day, which is consistent with typical ranges. The BOD loading per capita was 0.19 pounds per person per day, which is in the middle of typical ranges. However, there is an increasing trend in the BOD loading: the 2006 loading was 0.21 pounds/capita/day. Therefore a conservative estimate of 0.22 lb/cap/day will be used for projecting future BOD loading. The TSS loading was 0.19 pounds per person per day, which is lower than the standard range for design. A higher per capita contribution of 0.22 lb/person/day will be used for future design to be consistent with typical ranges.

3.4 Industrial Contributions

The City of Sioux Falls had 23 industries permitted to discharge to the municipal collection system in 2006. Table 4 shows the average daily flow and average daily loading contribution from each industry in 2006.

Table 4 2006 Industrial Loadings by Source

Permitted Industry	Flow (gpd)	BOD (lb/d)	TSS (lb/d)	TKN (lb/d)
Alamo Group (SMC)	2,427	1.3	2.2	0.4
Avera McKennan Hospital	128,952	440	337	45
Dakota Plating	2,082	3.3	2.1	2.3
Dean Foods	49,320	517	158	24
Gage Brothers Concrete	11,368	29	50	12
Howes Oil Co. Gas Stop	649	0	0	0
Hansen Mfg.	2,822	3.4	3.9	0.9
Hutchinson Technology	70,538	26	50	12
Land O' Lakes	37	42	13	1.8
Crimson Fire	3,284	5.7	3.6	1.0
Luverne Truck Equipment	3,475	23	14	1.9
John Morrell Company	264,951	2,544	1,271	246
North End Truck Wash	54,414	996	1,541	159
Qwest Communications	854	0.02	0.04	0.01
Sara Lee	5,065	182	52	3.0
Sioux Falls Regional Airport	704	22	0.01	0
Sioux Falls Regional Landfill	4,240	76	3.2	15
Sioux Falls Stockyards	76,249	48	74	13
Sioux Valley Hospital	162,803	447	348	40
SD Penitentiary	210,178	536	357	61
Smurfit-Stone Container	3,745	30	42	3.6
Veterans Administration Hospital	63,583	114	145	21
Woods Equipment Company	3,339	1.5	3.3	0.8
Total	**1,125,179**	**6,087**	**4,469**	**666**

Table 4 shows the industrial flow was approximately 1.1 mgd in 2006. The BOD loading was 6,087 lb/d, TSS loading was 4,469 lb/d and the TKN loading was 666 lb/d.

4.0 FACILITY PERFORMANCE

The WRF facility performance was analyzed to determine the unit process deficiencies and any "weak links" in the system.

4.1 Facility Samples

The WRF routinely samples between processes to gauge performance of the individual treatment units. The following sample points throughout the facility were used in the process analysis:

01	City Influent
01B	Primary Clarifier Influent
02	First Stage Trickling Filter Influent
03	Second Stage Trickling Filter Influent
10A	Activated Sludge Influent
12	Effluent Filter Influent
16	WRF Effluent

Figure 7 on the following page shows the sample points throughout the facility. During analysis of data for the Master Plan, it was discovered some sample points may not be representative of the actual conditions. The samplers for points 01, 02, 03, and 10A were located in the process pump station. The sample line lengths were long (especially points 01 and 02), and it was speculated that treatment occurred in the sample lines. Additional samplers were placed at the actual sample sites to compare with the existing, remotely located samplers. The original remote sample points and new sample points were tested concurrently for comparison purposes. After analysis, a number of the old sample sites were indeed determined to not be representative of actual conditions. The new sample sites were used for the analysis of the Re-Rate Document and Master Plan; therefore, the facility performance graphs shown in Section 4.0 are from September 8, 2007 through December 31, 2007. For more information regarding the sample comparison analysis, see Section 4.1 of the Re-Rate Document located in the Appendix.

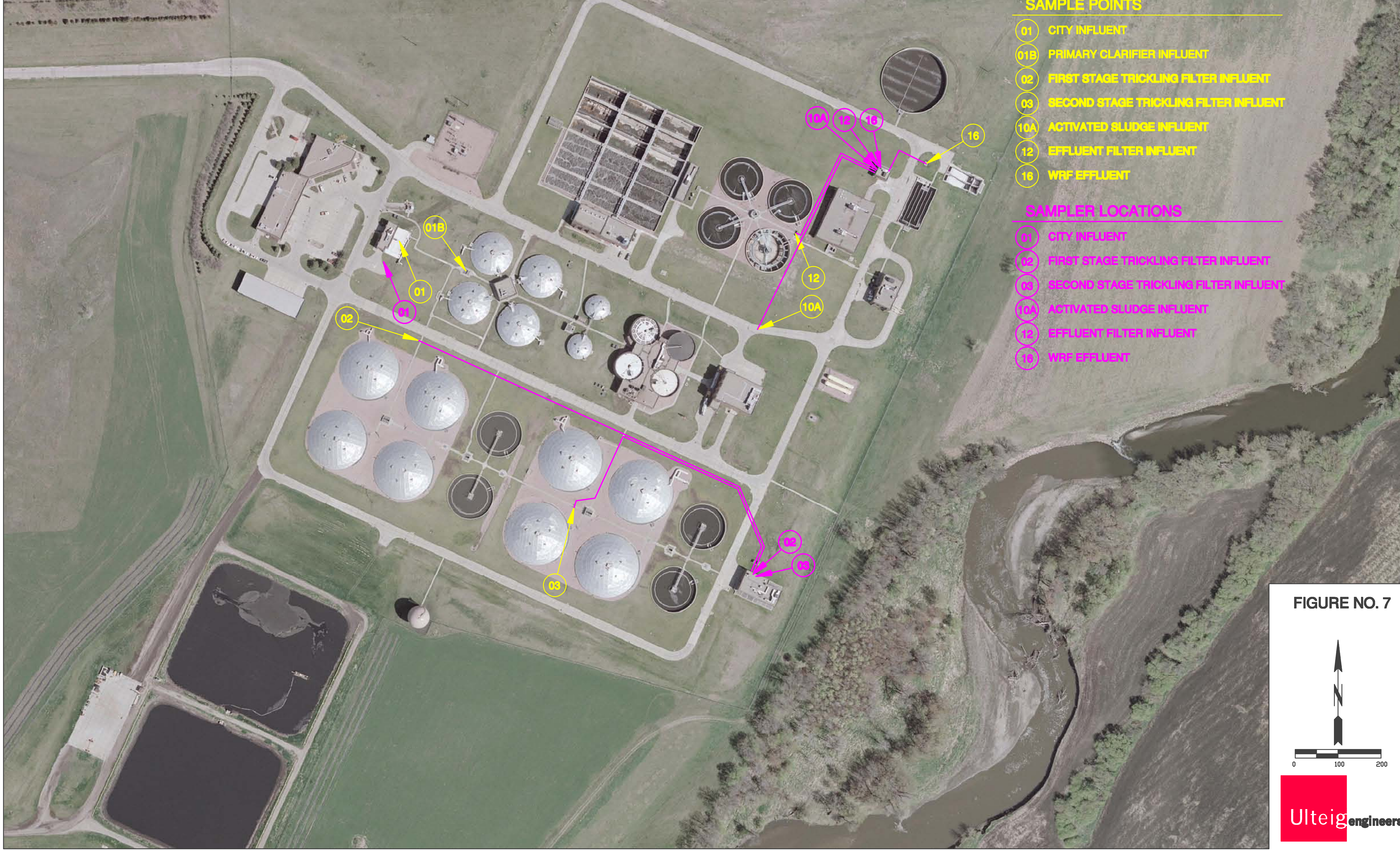

SAMPLE POINTS
01 CITY INFLUENT
01B PRIMARY CLARIFIER INFLUENT
02 FIRST STAGE TRICKLING FILTER INFLUENT
03 SECOND STAGE TRICKLING FILTER INFLUENT
10A ACTIVATED SLUDGE INFLUENT
12 EFFLUENT FILTER INFLUENT
16 WRF EFFLUENT
SAMPLER LOCATIONS
01 CITY INFLUENT
02 FIRST STAGE TRICKLING FILTER INFLUENT
03 SECOND STAGE TRICKLING FILTER INFLUENT
10A ACTIVATED SLUDGE INFLUENT
12 EFFLUENT FILTER INFLUENT
16 WRF EFFLUENT
N
0
100
200
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FIGURE NO. 7

4.2 Unit Process Performance

Figure 8 shows the average total BOD, carbonaceous BOD, soluble BOD and TSS through the facility. Figure 8 includes only samples taken at new locations from September 8, 2007 through December 31, 2007.

Figure 8 BOD, CBOD, SBOD and TSS through WRF

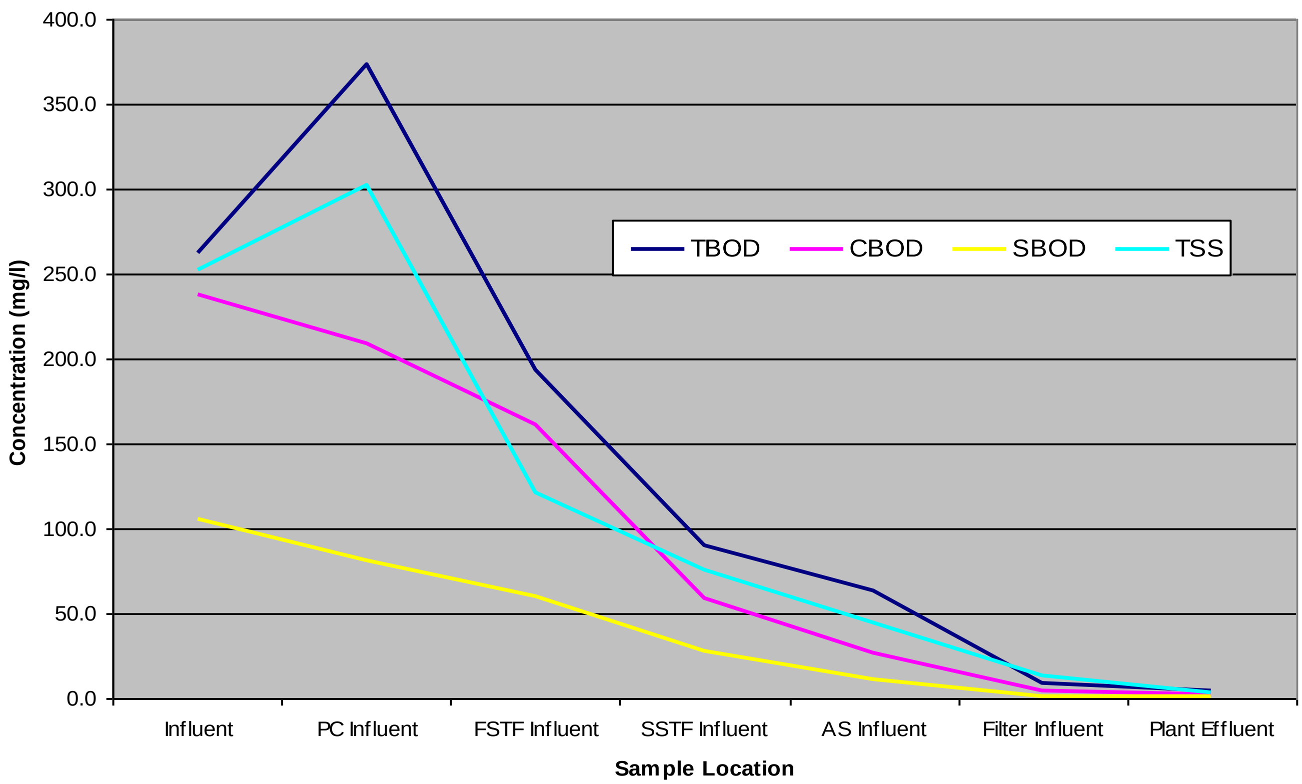
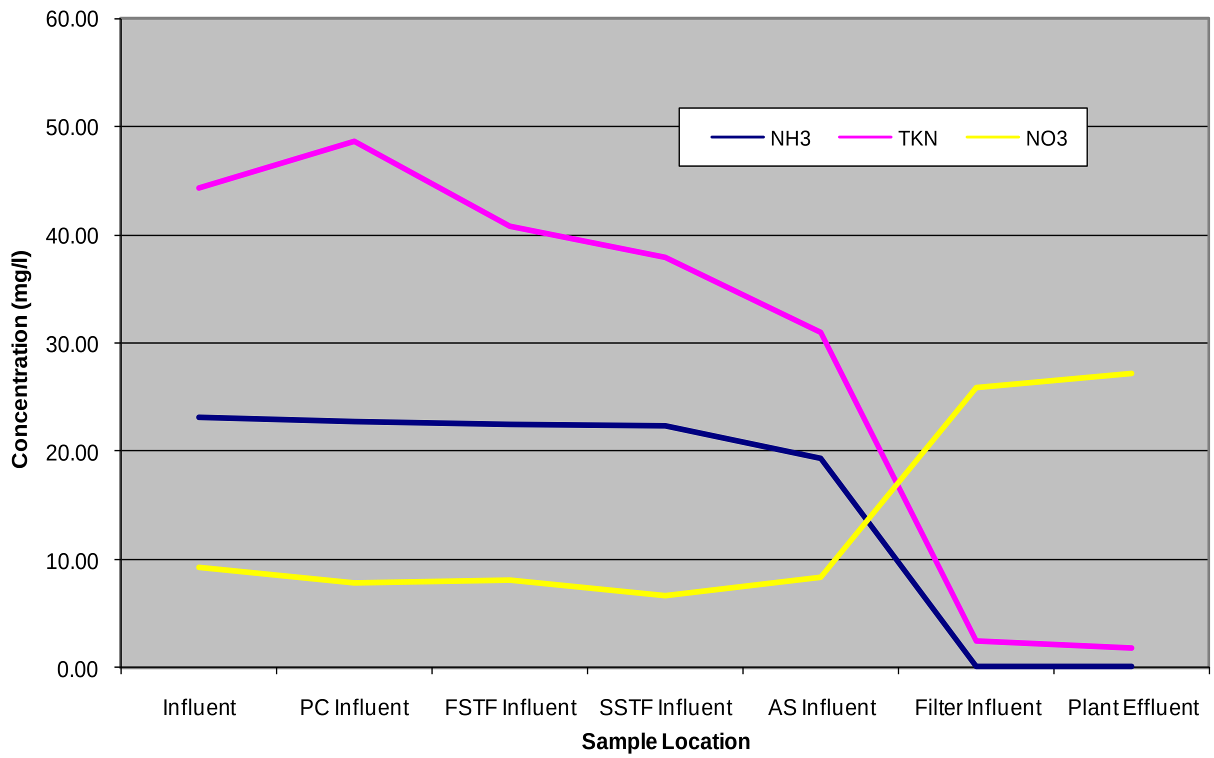


Figure 8 shows the influent organic strength increasing after the in-plant waste is added. CBOD and SBOD do not increase following in-plant waste addition because these parameters are less dependent upon solids. The return waste is high in solids content, and therefore only TBOD and TSS are affected. The primary clarifiers and first stage trickling filter processes reduce the organic strength substantially. Figure 13 shows the WRF processes are efficient at reducing organic strength.

Figure 9 shows the ammonia and TKN reduction through the facility unit processes. The data included was collected from September 8, 2007 through December 31, 2007.

Figure 9 Ammonia and TKN Reduction through WRF



As evident by the data in Figure 9, ammonia and TKN reduction occurs in the activated sludge process. Because the activated sludge basin was designed as the nitrification step at the WRF, the majority of TKN and ammonia are expected to be reduced in this process, as evident by Figure 14. The activated sludge basin appears to be nitrifying effectively as the TKN and ammonia are reduced below 3 and 1 mg/L in the WRF effluent, respectively. Nitrate concentrations rise following activated sludge, as would be expected following organic nitrogen and ammonia oxidation. The activated sludge system is generally operated at one-half the total available volume. There are no performance concerns related to the activated sludge basin at this time.

The overall facility has had excellent removal efficiencies throughout the years. The 2006 and September through December 2007 removal efficiencies are as follows:

Parameter	2006	2007
BOD	98.4%	98.5%
TSS	98.8%	98.4%
Ammonia	99.0%	99.7%
TKN	96.5%	96.0%

5.0 RECOMMENDED FORMAL CAPACITY OF WRF

The facility performance data, regulatory standards and a hydraulic model were used to determine the individual unit process capacities. The capacity available for each unit process is shown in Table 5 below. The yellow highlights correspond to average daily capacities; the green highlights correspond to peak hourly flow capacities; the orange highlights correspond to equipment limitations and the white rows correspond to hydraulic model and field testing. For more information regarding the capacities listed below see the Facility Capacity Re-rate Report located in Appendix A.

Table 5 Unit Process Constraints

Unit Process	Capacity/Constraint	Flow	
FS Intermediate Clarifiers	Average	17.31	mgd
SS Intermediate Clarifiers	Average	17.31	mgd
Effluent Filters	Bypass begins to Cl_2 Contact	22.00	mgd
Primary Clarifiers	Average	25.43	mgd
FS Intermediate Clarifiers	Peak Hourly	25.90	mgd
SS Intermediate Clarifiers	Peak Hourly	25.90	mgd
Second Stage IC	Weirs submerged	26.50	mgd
MH 11	Surcharging begins	26.50	mgd
SP MH 4	Surcharging begins	27.00	mgd
FS Trickling Filter	Distributor limitations begin	27.00	mgd
SS Trickling Filter	Distributor limitations begin	28.00	mgd
Chlorine Contact	Effluent flume surcharges	28.00	mgd
Process Pump Station	Pumping Capacity	30.00	mgd
Primary Clarifiers	Weirs submerged	30.00	mgd
Primary Clarifiers	Peak Hourly	30.52	mgd
SP MH 7	Weirs submerged	31.00	mgd
SP MH 3	Weirs submerged	31.00	mgd
Final Clarifiers	Peak Hourly	31.40	mgd
Effluent Filters	Peak Hourly	33.29	mgd
Brandon Road Pump Station	Pumping Capacity	35.00	mgd
SP MH 4	Structure freeboard limited	35.00	mgd
SP MH 1	Weirs submerged	36.00	mgd
Primary Clarifiers	Structure freeboard limited	37.00	mgd
Aerated Grit Removal	Effluent weir submerged	38.00	mgd
Chlorine Contact	Structure freeboard limited	41.00	mgd
Effluent Filters	Effluent weir submerged	41.00	mgd
First Stage IC	Weirs submerged	46.00	mgd
SP MH 5	Weirs submerged	48.00	mgd

5.1 Flow and Loading Capacity

The recommended average daily flow capacity for the Sioux Falls WRF is 21.0 mgd. This recommendation is based on:

- Hydraulic model simulations indicating effluent filter by-pass flow occurs at approximately 22.0 mgd.
- Field observations at varying flows indicating effluent filter by-pass conditions at high flows.
- No hydraulic constraints were simulated or observed during field testing below 22.0 mgd.

The recommended peak hourly flow capacity for the Sioux Falls WRF is 35.0 mgd. This recommendation is based on:

- Hydraulic model simulations indicating freeboard limitations at 35.0 mgd, however:
- Process pump station limits flow to the tertiary and final treatment process to 26.0 - 30.0 mgd. Process pump station is currently being expanded to accommodate increased flows and will not limit the facility capacity.
- Clarifier surface overflow rates appear to limit capacity according to recommended standards. However, performance data presented in the Re-Rate Document proves clarifier weir submergence still allows facility to easily meet limits to at least 35.0 mgd.

A number of organic models were evaluated to select an appropriate predictor of TBOD removal in the first stage trickling filters. The models evaluated include Velz, Modified Velz, *Modified Velz, NRC, Eckenfelder, and Regression of plant data. While the Velz model offered a good fit to the data, the regressed equation provided a better fit and presented a more conservative predictor for removal efficiency, when compared to the Velz equation. Therefore, the regressed equation is the recommended model for simulating BOD removal across the FSTF process at the Sioux Falls WRF.

Based on the performance data gathered from September through December of 2007 and the recommended Organic Model, the allowable headworks loadings for TBOD, TSS and TKN were determined in the Facility Capacity Re-rate Document included in the Appendix. ***The recommended loadings for the WRF are as follows: TBOD – 51,240 lb/d, TSS – 43,900 lb/d, and TKN – 9,440 lb/d.***

Table 6 shows a summary of the flow and loading capacities of the WRF as determined in the Facility Capacity Re-rate Document in the Appendix.

Table 6 Recommended Allowable Headworks TBOD, TSS and TKN Loadings

Parameter	Value
Average Daily Flow	21.0 mgd
Peak Hourly Flow	35.0 mgd
TBOD	51,240 lb/d
TSS	43,900 lb/d
TKN	9,440 lb/d

6.0 EQUIPMENT ASSESSMENT

An equipment assessment was performed based on condition, performance, existing capacity, opportunity for optimization and redundancy. The equipment assessment is presented in the order of the flow through the facility.

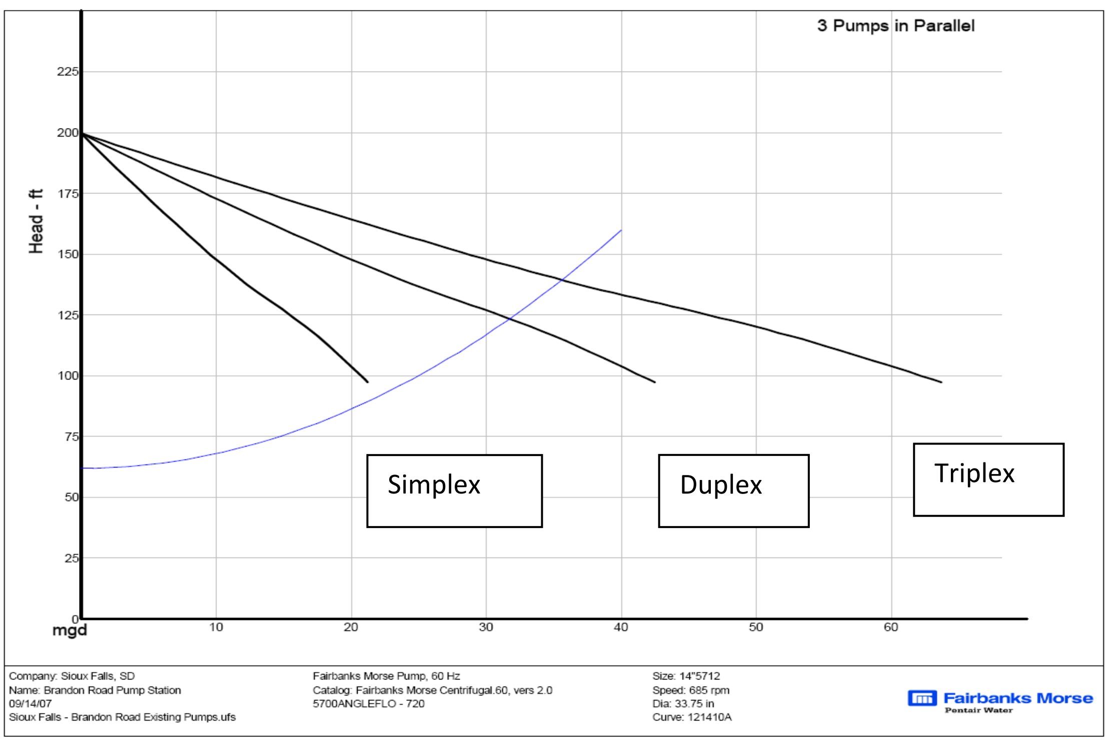
6.1 Brandon Road Pump Station

The Brandon Road Pump Station consists of the pumps, the Eddy Current Drives and the FMS Mechanical Screens.

6.1.1 Brandon Road Pumps

The Brandon Road pumps were sized for the original peak design flow and do not fit normal low flow conditions very well. The wide range between 'peak' flow and 'normal-low' flows can be difficult to match with a pump. The Equalization Basin that was added may reduce the required peak capacity and allow for pumps that are a better fit during normal operation. Figure 10 shows the approximate Brandon Road pump station hydraulics.

Figure 10 Brandon Road Hydraulics



The Brandon Road pumps cavitate when they operate alone at normal low flows. This is suction cavitation which occurs when there is less discharge pressure than the pump was designed for. On a printed pump curve, this type of cavitation occurs when a pump is operating past the right end of the curve (see Figure 10 – Simplex). Suction cavitation by itself isn't overly destructive in most cases, however, it indicates larger problems.

The operating conditions that cause suction cavitation also cause unbalanced radial loading within the pump, loading that the pump was never designed to handle. This is analogous to the wheel of a car that is out of alignment. The unbalanced loads shorten the life of shafts, bearings, and seals.

When the Brandon Road pumps are replaced in the CIP program the required 'Peak Capacity' should be re-examined. Installing pumps that better match the normal operating conditions would reduce maintenance and could also save $5000 - $10,000 a year in electrical costs.

Note that these comments are based on visual observations and calculated hydraulics from the original as-built documents. Hydraulics change over time so a thorough check of the Brandon Road hydraulics, with accurate gauges, should be performed prior to replacement.

6.1.2 Eddy Current Drives

The eddy current variable speed drives are an old technology that has been eclipsed by electronic VFDs. The main disadvantages of the eddy current drives are their relatively high initial cost and that they are prone to problems in environments where heat, dust, or condensation is an issue.

The best case electrical efficiency of new VFDs would only be marginally better than the existing eddy current drives. This equates to a potential best case savings of less than $10,000 per year total if all were replaced. This has proven to be a good environment for the eddy current drives so there is little incentive to replace them as long as parts are available.

6.1.3 Mechanical Screens

Two (2) FMS mechanical screens are installed at the Brandon Road Pump Station (see Figure 11). The mechanical screens have two purposes. The first is to protect the pumps from large debris. The second is to remove as many fine screenings as practical to keep them from entering the plant process train. When this station was built, it was the industry standard to install mechanical screens with 1" to 2" openings at all large pump stations.

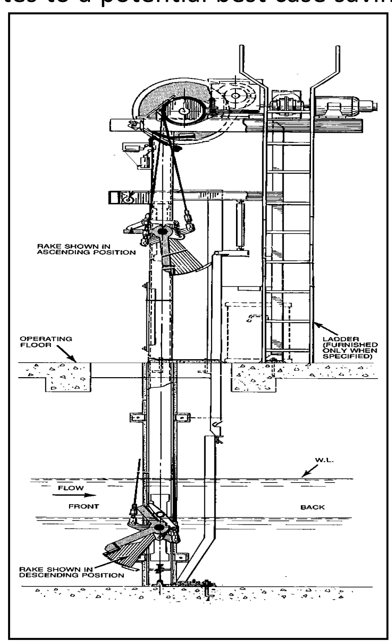


Figure 11 FMS Mechanical Screens

Over the years the capture rates of fine screens located at plant headworks has greatly improved. This has lessened the importance of capturing a portion of the fine screenings at many pump stations. Also the quantity of large debris has dropped in many collection systems as improvements have been made. In some applications the decision has been made to remove as many screenings as possible at the pump stations. In other applications it has been decided to send any debris that won't cause damage through the pumps and on to the plant headworks for removal. Prior to replacing these screens in the CIP program, the following items should be discussed so that any new screen is a good fit into the complete system:

- Performance of the Huber fine screens installed at the headworks in 2007
- Normal operating and maintenance procedures
- Type and quantity of debris in the Collection System
- Potential for large solids in the forcemain or wet well
- Pump Free Passage (the current literature shows a 6" spherical free passage but this should be verified before making a screening decision).

6.2 Brandon Road Force Main

The Brandon Road Force Main connecting the pump station to the plant headworks was constructed in 1982. It is a ductile iron force main with concrete/steel pipe sections at both ends. This force main is a critical item that lacks redundancy. Also, at future flow projections, pipe velocities exceed recommended standards. Portions of the force main were checked by plant operating personnel during the recent headworks upgrade and Brandon Road header replacement. These portions of the force main were in good condition. No condition assessment of the Brandon Road Force Main was done as part of this project. A redundant forcemain is estimated to cost approximately $8,060,000.

6.3 Grit Aeration System

Aerated grit chambers are located after the headworks screens to remove sand, small gravel, broken glass, cinders, metal fragments, and other small inorganic particles from the waste stream. Flow velocities are reduced in the grit chamber to allow heavier particles to settle out. The rising air bubbles and rolling flow pattern from the air eductor tube keep the heavier organic particles in suspension rather than settling out with the grit.

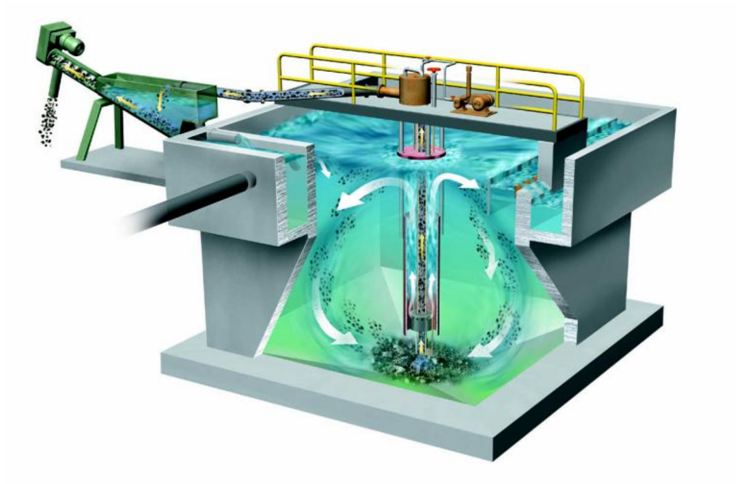
6.3.1 Equipment Description

The aerated grit chambers were provided by Walker Process (Aurora, IL; Contract #UW34431) and include the following.

- ¼" Steel Plate Lift (eductor) Tube with 304 Stainless Steel Pipe Supports
- 304 Stainless Steel Airline
- ¼" Plate Steel Baffles
- Galvanized Diffuser, 3/8" holes, qty. 36 per diffuser; 160-320 SCFM per unit

- Aluminum Bridge Structure (6063-T6)
- 3/8" Aluminum Checkered Floor Plate (6063-T6)

Figure 12 Walker Process Grit Chamber



The positive displacement blowers are Roots (RAI frame 56) operating at 1300 rpm with a capacity of 206 at 7 psi (design) and 224 SCFM at 4 psi. Blowers are driven by 15 hp, 1750 rpm motors.

6.3.2 Equipment Condition

The aerated grit chambers were not inspected as part of the field inspections. However, the next time these units are taken down for inspection and maintenance the bridges should be checked. The bridge beams are the same grade of aluminum as the trickling filter distributors which have been severely attacked. The other items that may have degraded or worn over time are:

- Plate steel eductor and center baffles
- Galvanized steel pipe clamps where the support pipes meet the eductor
- Galvanized DIP air diffuser
- Effluent baffles and baffle supports

The unique feature of rolling grit chambers is that settling is not dependent on the retention times, above certain minimums, but rather by the rolling action induced by the aeration system. This allows for a high degree of turn-down without a loss of performance.

The optimum air flow rate to the grit chamber maximizes the amount of grit removed with a minimal amount of putrescibles. Determination of the optimum air flow rate can only be made through a process of trial and error. The manufacturers approximate setting for this specific size grit chamber is for the air valve to be ½ open.

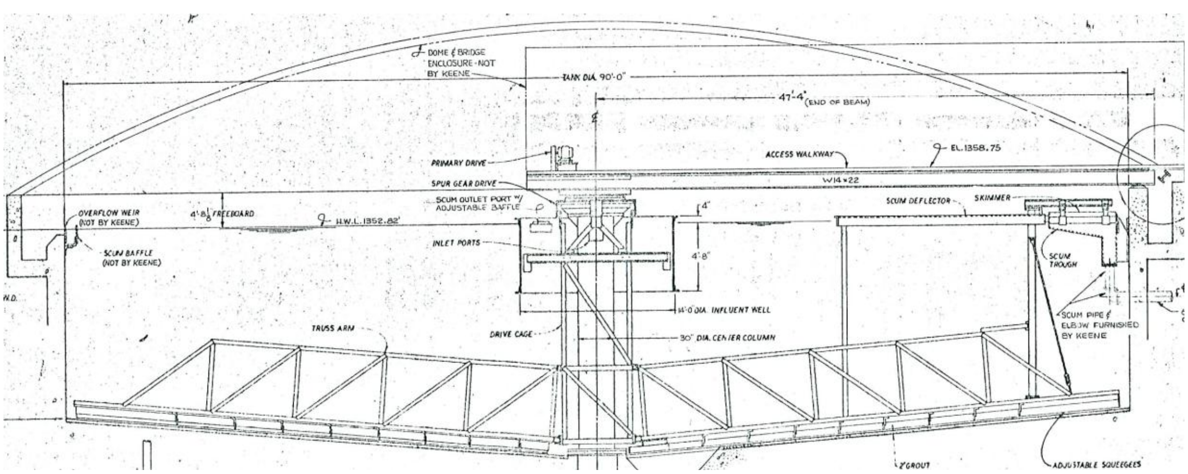
The grit from the receiving dumpster should be examined for putrescible content. Too high a putrescible content signifies that too little air is being applied to the grit chamber. Grit that is very clean or is comprised of only coarse material signifies too much air is being applied to the grit chamber, causing carryover into the primary clarifier. A large quantity of grit is present in the digesters; therefore, improvements to the grit removal system may reduce the digester problem.

The grit washer should also be checked to make sure that the quantity of flow being discharged to the grit washer does not interfere with settling within the unit. Excessive flows will re-circulate grit back to the aerated grit chamber. The flow to the grit washer is controlled by changing the speed of the Wemco Model C pumps.

6.4 Primary Clarifiers

The facility has four Primary Clarifiers to remove settleable solids and scum from the wastewater. The Primary Clarifiers are 90 feet in diameter and have 8-feet of sidewater depth. The Clarifiers are center-feed with peripheral weirs. Settled solids (sludge) are collected by rotating scraper arms on the bottom of the tank and directed to the sludge withdrawal hopper. Skimmers deflect scum into a scum trough. From the sludge hopper and scum trough, the sludge and scum are pumped to the solids handling units.

Figure 13 Primary Clarifier



6.4.1 Equipment Description

The Primary Clarifier mechanisms were manufactured by Keene Corp. (Aurora, IL., ID #S.O. 90128-1) and include:

- Eurodrive helical main drive
- 1 hp Baldor motor, explosion-proof to Class 1, Division1
- Rotation Speed of two (2) complete arm revolutions per hour
- ¼" ASTM A36 steel center column, drive cage, and truss arms, field coated
- 10 gauge brass squeegees
- Fiberglass Reinforced Plastic (FRP) Weirs
- Over-torque protection provided by motor overloads, drive cut-out and alarm limit switches, and a 50,000 ft-lb shear pin coupling

6.4.2 Equipment Condition

The wastewater plant has three of the four clarifiers in service during normal operation. Each year a different clarifier is taken out of service and scheduled maintenance is performed.

Primary Clarifier #4 was field inspected and is in good working condition. All metalwork had recently been sandblasted and painted. The structural steel components, including the center column, influent well, drive cage, arms, skimmer, and scum trough, appear to be in good condition with minimal decrease in metal thickness.

The concrete basin is in good condition with minimal spalling. There are some surface voids left from the original concrete pour but this doesn't affect clarifier performance. The FRP weirs are in very good condition.

The access walkway, center platform, and housing are in good condition. The clarifier domes appear to be in good condition with no significant corrosion.

Inspecting the center column interior, the helical drive, sprockets, gears, chain drive, and bearings for corrosion and wear was beyond the scope of this inspection and thus, no opinion was given as to the condition.

6.5 Trickling Filter Rotary Distributors

Secondary treatment is accomplished by two stages of Trickling Filters. The four First Stage Trickling Filters are 135 feet in diameter and are 7 feet deep. The four Second Stage Trickling Filters are 145 feet in diameter and are also 7 feet deep. The Trickling Filters contain Sioux Quartzite Rock media, distributor arms and an underdrain system. The microorganisms on the media of the Trickling Filters remove pollutants in the waste stream.

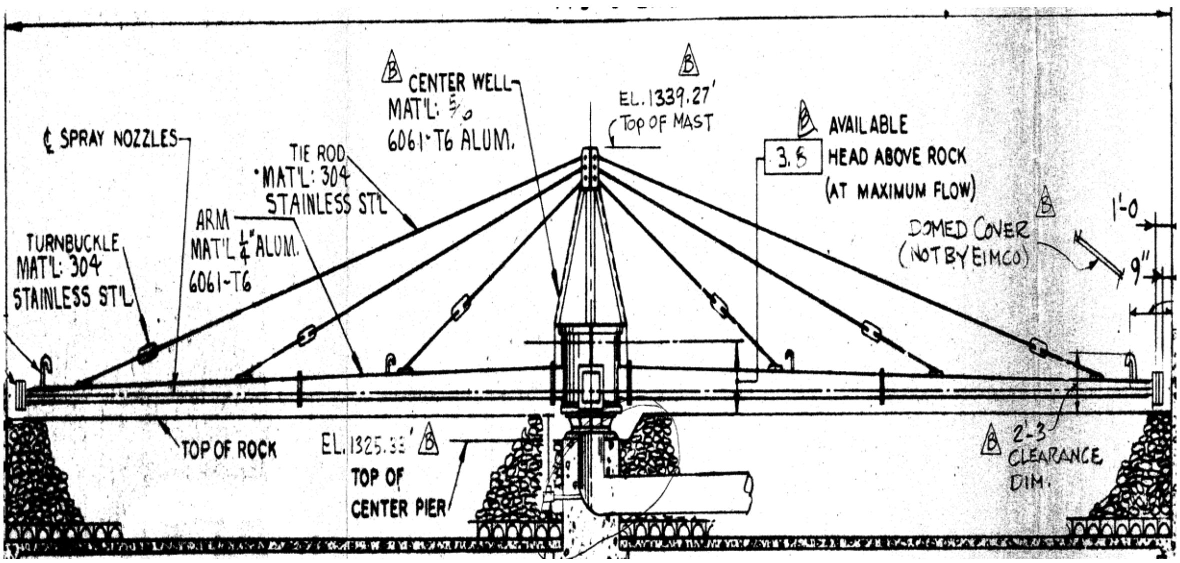
6.5.1 Equipment Description

The mechanical portion of the First and Second Stage Trickling Filters were manufactured by EIMCO (Salt Lake City, UT; Project #1846-M) and include:

- Four (4) Distribution Arms per filter, 6061-T6 Aluminum ¼" thick.

- 304 Stainless steel tie rods and turnbuckles
- 5/8” Aluminum 6061-T6 Center Well and Support Mast
- Aluminum Vent Pipes and Dump Gates
- Aluminum Nozzles
- Cast Iron base assembly

Figure 14 EIMCO Trickling Filter



6.5.2 Equipment Condition

The four (4) First Stage and four (4) Second Stage trickling filters are in service during normal plant operation. First Stage Trickling Filter #1 and Second Stage Trickling Filter #1 were taken out of service for inspection.

The aluminum trickling filter components, including the distributor arms, center columns, and support masts, have experienced severe corrosion and are in very poor condition. There is significant pitting on the outside of the arms and a buildup of oxidized aluminum (see Figure 15).

Figure 15 Trickling Filter Arm Corrosion

The corrosion has not been limited to the exterior surfaces. A significant amount of interior corrosion is visible through the nozzles (see Figure 16).

Figure 16 Trickling Filter Nozzle Detail

This interior arm corrosion adds friction to the flow, which creates a significant amount of head loss at high flows. The increased headloss in the distributor arms causes water to backup in the center column and upstream in Splitter Manhole #4. This was proven during hydraulic field testing conducted October 11, 2007.

The severe arm corrosion and reduction in metal thickness raise concerns about the structural integrity of the arms. Similar corrosion has taken place on the center column and support mast.

It is important to note that the Headworks and Primary Clarifiers are vented to trickling filters FSTF2, FSTF4, SSTF2, and SSTF4. These units were not field inspected because of safety concerns related to hydrogen sulfide in the received venting. Due to the hydrogen sulfide, it is likely that these trickling filters are in worse condition than the ones inspected. Some of the aluminum nozzles were worn thin and some had already been replaced. There appeared to be some crystallized buildup in the nozzle openings (see Figure 12). The concrete basins appeared to be in good condition. The outside of the domed covers appeared to be in good condition. There was some corrosion on the inside of the domes however a thorough inspection of the domed covers was not performed.

6.6 Intermediate Clarifiers

Each stage of Trickling Filters is followed by two Intermediate Clarifiers to remove biomass that sloughs off the Trickling Filter media. These Clarifiers are 105 foot diameter with side water depths of 10 feet. The Clarifiers are center-feed with peripheral weirs. Settled biomass is collected by squeeges on the bottom of the rotating arms and directed to the sludge withdrawal hopper. From the sludge withdrawal hopper the sludge is gravity fed to the Humus Wet Well in the Process Pump station.

6.6.1 Equipment Description

The Intermediate Clarifier mechanisms were manufactured by Keene Corp. (Aurora, IL., First Stage ID #S.O. 90129-1; Second Stage ID #S.O. 90129-1) and include:

- Eurodrive helical main drive
- 1 hp General Electric motor, premium efficiency, TEFC
- Rotation Speed of two (2) complete arm revolutions per hour
- ¼” ASTM A36 steel center column, drive cage, and truss arms, field coated
- 10 gauge brass squeegees
- Fiberglass Reinforced Plastic (FRP) Weirs
- Over-torque protection provided by motor overloads, drive cut-out and alarm limit switches, and a 50,000 ft-lb shear pin coupling

6.6.2 Equipment Condition

All intermediate clarifiers are in service during normal plant operation. The First Stage Intermediate Clarifier #1 was taken out of operation and field inspected. The center column, drive cage, and truss arms are in good condition with minimal corrosion. The influent well appears to be in fair condition but there are several holes and two (2) seam leaks on the south southwest side as shown in Figure 17. Several holes appear to be symmetrical so they may be intentional drainage although there are no drain holes documented on the equipment shop drawing submittals.

Figure 17 First Stage Intermediate Clarifier #1 Influent Well

The concrete basin is in good condition with minimal spalling. The concrete trough has several leaks near form seams (see Figure 18). The weirs are in good condition.

Figure 18 First Stage Intermediate Clarifier #1 Effluent Trough

The access walkway, center platform, and railings are in very good condition. There was no apparent oil leakage from the main drive but inspecting the internal mechanical components for wear was beyond the scope of this inspection and thus, no opinion was given as to the condition.

6.7 Lime Feed System

Lime was used only sparingly until 5 years ago due to the alkalinity of the groundwater infiltrating the collection system. Now that groundwater infiltration has been reduced, lime is being fed seasonally at a rate varying from 900 – 2,400 lbs/day.

6.7.1 Equipment Description

The existing lime feed system utilizes quicklime, or 'pebble' lime. This type needs to be slaked prior to use by chemically reacting the quicklime with water at controlled quantities and temperatures. Precision in the slaking process is critical, an inefficient slaking process results in excessive wasted lime in the form of grit. Grit is hard on equipment and is labor intensive to remove.

Lime may be purchased from the lime suppliers already slaked in the form of hydrated lime which is a dry powder. Quicklime is somewhat less expensive but hydrated lime requires less equipment and O&M. The cost difference is dependent on the number of local suppliers and their proximity to the plant. The cost difference for the Sioux Falls plant in 2008 would be $10 - $11 per ton. Hydrated lime is 25% water by weight so hydrated lime costs would be 36% higher than current quicklime costs.

6.7.2 Equipment Condition

The original system was designed with positive displacement diaphragm pumps. Diaphragm pumps, and the ball check valves used with them, proved to be a poor fit for the quicklime system with frequent clogging and high wear. A positive displacement rotary lobe pump, that was on-hand, has been placed into service for the lime feed. This has reduced the clogging somewhat but not the wear. Most new plants are designed so that the lime is fed by gravity into the basins but the existing piping and equipment elevations make this difficult. Two pump types that may work better in this application are peristaltic 'hose' pumps and progressive cavity pumps.

6.7.3 Alternatives

The current and future predicted usage were calculated as follows:

2006 Lime Feed Peak Day (Dec. 6th):	2,213 lbs
Dec. 6th Daily Flow (dry weather):	14.41 MGD
Lime dosing rate:	18.41 mg/L
Predicted 2030 Dry Weather Flow:	20.5 MGD
Lime dosing rate:	18.41 mg/L
Predicted 2030 Lime Feed Peak Day:	3,148 lbs
With 15% safety factor:	3,480 lbs.

The options for improving the lime feed system include:

<u>Option 1 – New slakers and pumps with quicklime in the existing location</u>

The existing lime feed equipment would be completely replaced, except for the silos. The existing lime slakers provide poor control of the slaking process causing excessive lime to be wasted as useless grit. The grit that escapes the grit removal system causes significant wear to the lime feed pumps. The best available slaker technology, which optimizes the slaking process was used in this alternative. This minimizes wasted material and wear on ancillary equipment due to grit. There are two general types of lime feeders, volumetric and weight based. The quicklime alternatives are based on weight type feeders for precise chemical feed into the slakers, optimizing the slaking process. New hose pumps are included to pump the lime to the existing application points.

Capital Cost:	**$1,099,000**
20yr, 3.5% Finance Cost:	$ 77,300
Yearly Quicklime Cost (2006 usage)	$ 20,700
Annual Cost:	**$ 98,000**

Option 2 – Hydrated Lime at the existing location

The existing lime feed equipment, including the silo, would be completely removed and replaced with equipment suitable for hydrated lime. Volumetric feeders would deliver the hydrated lime from the silo to the mix tanks. Since the lime is already slaked the volumetric feeders provide adequate accuracy. New hose pumps are included to pump the lime slurry to the existing application points.

Capital Cost:	**$769,000**
20yr, 3.5% Finance Cost:	$ 54,100
Yearly Quicklime Cost (2006 usage)	$ 28,100
Annual Cost:	**$ 88,200**

Alternative 3 – Hydrated Lime at a new location to allow gravity feed

A new hydrated lime feed system with silo would be constructed at a location that allows the lime to flow by gravity into the process stream. Likely locations are at the aeration basin splitter box or near the process pump station. The silo, volumetric feeders, tanks, mixers, and controls would come as an integrated package from the manufacturer. A small lime feed building would be constructed to provide temperature and moisture control around the silo cone and the lime feeders as well as some workspace around the mix tanks. Lime slurry would flow by gravity from the mix tanks to a feed trough. This option minimizes O&M by eliminating slakers, feed pumps, and feed piping and therefore is the recommended alternative for lime feed improvements.

Capital Cost:	**$830,000**
20yr, 3.5% Finance Cost:	$ 58,400
Yearly Quicklime Cost (2006 usage)	$ 28,100
Annual Cost:	**$ 86,500**

6.7.4 Recommendations

Alternative #3 is recommended because the lime feed system is critical to meeting effluent requirements and this option eliminates the major sources of failure.

Allowing the lime to flow by gravity eliminates failures due to clogging and wear in lime feed pumps and feed piping. Purchasing hydrated lime rather than buying new lime slakers makes sense economically for the volume used at the plant. This also eliminates potential failures associated with the slaking process.

Alternative 3 eliminates the electrical cost of pumping lime but this is offset by heating costs for a new structure. The annual costs are for capital costs and lime only. They do not include depreciation costs associated with the slakers in option #1, or the lime feed pumps in options #1 and #2. Nor do the annual costs account for plant maintenance time associated with additional equipment and pumping lime.

6.8 Activated Sludge Basin

The activated sludge system at the Sioux Falls WRF is designed for ammonia oxidation from the waste stream before it enters the Big Sioux River. The activated sludge system includes the aeration basin, blowers, diffuser system and control building.

6.8.1 Equipment Description

There are six (6) aeration basins for nitrification. Typically only three of the six are in operation. Each basin is 280 ft x 43 ft with a side water depth of 14 ft. The total volume for the aeration basins is 8.2 million gallons.

Four (4) 800-horsepower blowers are used to provide oxygen to the aeration basins through a network of diffusers and aeration piping. Since the plant went on line the City has used only one (1) blower for aeration purposes. The blowers have a firm capacity of 46,500 scfm (3 blowers @ 15,500 each).

The aeration diffuser system is comprised of stainless steel coarse bubble diffusers. The air stream from the coarse bubble diffusers provides oxygen to the microorganisms and also provides for mixing of the contents within each basin. The diffusers have a dirty water transfer efficiency of 4-5% which will provide an oxygen transfer rate of 1,925 to 2,400 lbs/hour.

6.8.2 Equipment Condition

A brief inspection of the aeration facility and related equipment was conducted as part of this equipment evaluation. The concrete basin appears to be in good shape with minimal concrete spalling. The bubble pattern for the aeration basin diffuser system was good for the basins that were in operation during our inspection. The stainless steel coarse bubble diffusers that were not in operation looked to be in very good condition.

The Roots Blowers are running approximately 10,000 to 12,000 hours before service to the bearings is required. Obviously there is a tremendous amount of redundancy with the existing blower system and taking a blower out of operation for service is not a problem.

The manually operated air valves do not close properly. There appears to be leakage even when the valves are fully closed. These valves should be inspected further by a manufacturer's representative to evaluate the need for repair or replacement. The cost to replace the existing air valves is approximately $140,000.

6.8.3 Aeration Options

Treatment within the activated sludge basin appears to be adequate, but this area of the plant consumes a great deal of electricity due to operation of the 800 horsepower blowers. During summer months, the activated sludge basin has experienced oxygen limited conditions, associated with the aeration capacity of using only one 800 hp blower. Because this time frame is short, the City doesn't turn on an additional blower as the costs of running this blower are substantial. However, as loading to the WRF continues to increase (associated with population growth) this time frame will grow longer. Aeration options must be evaluated to optimize the existing system while holding economic considerations paramount.

Several alternatives have been considered during the evaluation of the aeration system in order to reduce the energy consumption in this area of the plant: (1) do nothing, (2) turn on one additional blower during times when oxygen is limited, (3) add an additional smaller blower with a VFD to supplement the oxygen during the limited periods, (4) convert the entire system to fine bubble aeration, and (5) leave the existing coarse bubble diffusers in cells B and C and place fine bubble diffusers in cell A. The installation of variable speed drives on the existing blowers was explored briefly in order to ramp the blowers up and down based on dissolved oxygen levels in the basin. The cost of these units makes this alternative cost prohibitive since the payback is not favorable.

Option 1 - Do Nothing

The option to do nothing is eliminated because the oxygen limitation period will continue to grow as population increases. The City cannot ignore the increased loading and must appropriately treat wastewater as to not exceed their effluent permit limitations.

Option 2 - Turn on an Additional Existing Blower

To supplement the oxygen limited periods, an additional 800 hp blower could be started and operated. As mentioned, the City has been able to avoid performing this action as the oxygen limited period is relatively short. Also, the costs to start a second blower are expensive due to both the surcharge for starting the blower and the electrical costs for operating an 800 hp blower. The demand surcharge in the summer is approximately $3,830 per month to use a second blower. The electrical costs to operate the second blower for one month is $9,370, which equates to a total monthly cost of $13,200.

Option 3 - Add a Smaller Blower

To compensate for the oxygen limited periods, a smaller, variable speed blower could be installed. Under this option, the City would only turn on the smaller blower when needed and adjust the output depending upon basin conditions. Based upon predicted loadings, a 10,000 scfm blower (400 hp) with a turn down capability of 6,000 scfm could be utilized. However, as the City approaches the design flow of 21.0 mgd, the aeration required for treatment demands that a second exiting (800 hp) blower be operated. Therefore, the secondary blower is an option to supplement the existing blower(s) as needed. The cost to install the additional 400 hp blower is $890,000.

Option 4 – Replace coarse bubble diffusers with fine bubble diffusers

Another alternative currently being explored is converting from coarse bubble to fine bubble diffusers in order to increase treatment efficiency, oxygen transfer and reduce power costs. Many wastewater treatment facilities are switching from coarse bubble to fine bubble diffusers for efficiency reasons. There are advantages and disadvantages with this alternative some of the advantages of changing to fine bubble diffusers include:

Advantages:

- Exhibit high oxygen transfer efficiencies.
- Exhibit high aeration efficiencies (mass oxygen transferred per unit power per time).
- Easily adaptable to existing aeration basins.
- Lower VOC emissions.

There are also some disadvantages that must be evaluated under this alternative:

Disadvantages:

- Fine pore diffusers are susceptible to chemical or biological fouling and may require routine cleaning.
- Air flow patterns are critical to their performance.
- Must provide adequate mixing similar to coarse bubble system that was removed, not just oxygen transfer.

Option 4a - Replace Three of the Six Basins with Fine Bubble Diffusers

To increase the oxygen transfer efficiency (and thereby increase treatment capacity) fine bubble diffusers can be placed in three of the six aeration basins. The cost to install fine bubble diffusers is $1,090,000.

The design of the A and B cells is based on oxygen requirements for treatment as the controlling factor. The design of the C cells, due to the decreased loading, is based on mixing requirements as the controlling factor. Also, the existing blowers are inadequate as the pressure drop associated with fine bubble diffusers is greater than coarse bubble diffusers. The cost to add three 200 horsepower blowers for the fine bubble system is $520,000. The cost for both fine bubble aeration in three cells and three new blowers is $1,610,000.

Option 4b - Replace Diffusers with Fine Bubble in Cell A Only

The coarse bubble diffusers in the A cells could be replaced with fine bubble diffusers. However, the oxygen required for treatment is the controlling design factor in the A and B cells. Even though the oxygen required for mixing is the controlling design factor in the C cells, it is nearly equal to the treatment requirement. This option would require new blowers dedicated to the fine bubble diffusers in the A cells since the discharge pressure of the existing blowers is not adequate. The existing blowers could be used to deliver air to the B and C cells. They would be oversized since the air required would be cut in half. The cost to convert the A cells to fine bubble diffusers and purchase new blowers is $1,080,000. Since the existing blowers have no turndown capabilities, the blowers for the coarse bubble diffuser system would waste approximately $75,000 per year in electricity costs by supplying unneeded air.

A summary of the capital costs and electrical costs for the a design flow of 21 mgd is shown in the following table. A present worth for a 20 year design life (5% per year) was computed for comparison purposes.

Table 7 Aeration Alternatives Cost Analysis Summary

Aeration Alternative	Capital Costs(1)	Annual Electrical Costs(1)	Present Worth(2)
Option 2 – Two 800 hp blowers in service	$0	$317,000	$3,948,000
Option 3 – One 800 hp blower, one new 400 hp blower in service	$890,000	$240,000	$3,880,000
Option 4a – New fine bubble diffusers, three new blowers (3 basins)	$1,610,000	$77,000	$2,573,000
Option 4b – New fine bubble diffusers in A cells, two new blowers	$1,080,000	$197,000	$3,536,000
(1) Costs based on aeration necessary for 21 mgd (2) Rate of 5% per year used for present worth calculation			

6.8.4 Recommendations

Based upon the cost comparison in the previous table, option 4a is the most economical solution for the City of Sioux Falls. Although the initial investment in fine bubble diffusers and blower equipment is much greater than the other alternatives, the annual O&M savings make this the most beneficial alternative over the next 20 years. Keep in mind this alternative would only replace three of the six cells with fine bubble diffusers. It is recommended that coarse bubble diffusers be kept in three cells and one exiting 800 hp blower remains to be used during times when fine bubble maintenance is required.

As the operational need for a second existing 800 hp blower increases, the implementation plan for new fine bubble diffusers should be considered.

6.9 RAS pumps

There are five (5) return activated sludge pumps that pump from the final clarifiers back to the aeration basins. The pumps are Allis-Chalmers model 14x14x17.5 with impellers trimmed to approximately 15 5/8" diameter.

During the field inspection Pumps #1 & #3 were operating and it was noted that these pumps sounded a little rough. Gauge readings of 1 psi suction and 7 psi discharge were taken (these are approximate readings, gauge accuracy unknown). These pressure readings equate to 6 psi or 14' TDH. The pumps were designed to operate at 4700 gpm at 24' TDH with the right side of the curve ending at 18' TDH.

Assuming that the gauge readings are accurate, it is likely that the RAS pumps are running in a condition similar to the Brandon Road pumps described previously. Although cavitation isn't obvious, the unbalanced loading and accompanying bearing wear and shaft fatigue may still be taking place. A check of the normal RAS Pump operating hydraulics should be performed with accurate gauges prior to pump replacement in the CIP program

6.10 Final Clarifiers

The four (4) Final Clarifiers are located after the Aeration Basins and settle solids from the treated wastewater. The Final Clarifiers are 100-foot diameter with side water depths of 14 feet. The Clarifiers are center-fed with peripheral weirs. Settled sludge is directed by 'V' type plow arrangement to sludge suction tubes located on the arms (four tubes per arm, eight total). Settled sludge flows through the suction tubes by gravity to the center sightbox, adjustable slip tubes are provided to control the rate of flow. From the sightbox, the sludge is pumped by the Return Activated Sludge pumps.

6.10.1 Equipment Description

The Final Clarifier mechanisms were manufactured by Walker Process (Aurora, IL, Contract #UW00534) and provided with:

- Eurodrive variable speed drives, all but one (1) have been replaced with constant speed drives.
- Rotation Speed of two (2) complete arm revolutions per hour
- ¼" ASTM A36 steel center column, drive cage, sightbox, and truss arms, field coated
- 20 gauge brass squeegees on 'V' plow type flights.
- Over-torque protection provided by motor overloads and a Belleville spring load detection system

6.10.2 Equipment Condition

The plant runs three of the four final clarifiers during normal operation. All four final clarifiers are put into service during wet weather events when needed. Final Clarifier #1 was field inspected; it had been taken out of service several days prior to the inspection. This clarifier is in poor condition.

The center column, sight box, and influent well are in fair condition. The drive cage shows areas of significant corrosion. The truss arms show significant corrosion throughout with areas of severe corrosion and loss of metal thickness as shown in Figure 19.

Figure 19 Final Clarifier #1 Truss Arms

Significant corrosion has also occurred on the support arms for the scum box as shown in Figure 20. The concrete basin itself is in good condition. There are some pitted areas along with some spalling but no severe problems.

Figure 20 Final Clarifier #1 Scum Box Support Arms

The corrosion of the final clarifier mechanisms is advanced and is more severe than that in the primary and intermediate clarifiers. It may seem counterintuitive that the primaries (typically high in H_2S and therefore a likely corrosive environment) are not in the same condition as the finals, however the corrosive nature of wastewater following tertiary nitrification explains this phenomenon. The theoretical alkalinity demand for nitrification is 7.1:1 (lbs alkalinity : lb NH_3-N). As alkalinity is consumed the buffering capacity, or the ability to resist changes in pH, of water is decreased. As evident by the data collected at the WRF, the average plant effluent alkalinity is between 45 and 50 mg/L. At this low concentration, hydrogen ions in solution begin to lower the pH, due to the limited buffering capacity of the wastewater. The final clarifier effluent has a typical pH ranging between 6.5 and 7.0; this indicates acidity. Lime is currently fed by the WRF staff to counteract the alkalinity consumption. Prior to I/I reduction work by the City, lime feed was unnecessary due to the high alkalinity present in groundwater. It has been noted by plant staff that since the I/I correction work, the rate of corrosion in the final clarifiers has increased dramatically. This would indicate that the main reason for corrosion in the final clarifiers is due to alkalinity consumption and subsequent low pH conditions.

There are other potential reasons for corrosion, in addition to the low pH conditions. One possibility is denitrification. As sludge settles to the bottom of the final clarifiers, low oxygen conditions are created. In this anaerobic state, nitrifying bacteria will denitrify. Anaerobic bacteria will also reduce the naturally occurring sulfates in water to sulfides, leading to the production of H_2S and/or sulfuric acid. This acid production leads to additional corrosion.

Another potential culprit leading to the FC corrosion is the presence of electrolytes, most notably chloride. Electrolytes are ionic in nature and have an imbalanced distribution of electrons. The product of this imbalance is chemical dissociation.

The low pH condition in the final clarifiers is a favorable environment for the production of hydrochloric acid (as chlorides react with free hydrogen). The average chloride concentration for the Sioux Falls WRF effluent is 270 mg/L. This value is quite high when compared to standards (typically between 30 and 90 mg/L). Although the influent concentration is nearly identical, the reaction of chlorides would not be as prevalent upstream of the activated sludge basin (due to the higher pH conditions). In addition, chlorides will react with steel in these low pH conditions in an anode/cathode relationship due to the imbalance of electrical charges, similar to a battery. Because of the electrochemical activity occurring between steel and chloride, the steel will eventually corrode and pit. Therefore, chlorides may be contributing to corrosion in the final clarifiers due to the production of hydrochloric acid and electrochemical reactions. Sacrificial anodes are available that can be used to counteract the corrosion associated with chlorides. These anodes are attached to the steel mechanisms and replaced periodically. It is estimated that each final clarifier would contain 50 anodes and the total cost for all 4 clarifiers is $21,000. These anodes would have to be replaced every 5 to 7 years.

The access walkway, center platform, and railings are in very good condition. There was no apparent oil leakage from the main drive but inspecting the internal mechanical components for wear was beyond the scope of this inspection.

6.11 Gravity filters

Tertiary filtration is provided by eight (8) dual-media gravity filters, each measuring 34 feet by 17 feet by 8 feet deep. These filters remove a portion of the suspended solids that do not settle out in the final clarifiers.

6.11.1 Equipment Description

Flow is directed to all eight (8) gravity filters during normal operation. As solids accumulate in the media, water begins to backup in the filter. Solids will continue to accumulate and the water will continue to backup until a backwash is called for. During a backwash cycle, effluent is directed backwards through the filter media, flushing the accumulated solids out the backwash troughs and into the backwash basin.

The original gravity filter equipment and media were provided by Roberts Filter (Darby, PA; contract #1921) and include:

- 4” Thick Pre-cast Wheeler bottoms with porcelain spheres and thimbles (Shown in Figure 47).
- Media consisting of layered gravel, 12” of sand, and 24” of anthracite.
- Fiberglass troughs
- Rotary Surface Wash Mechanisms, 2 per filter
- Level Monitoring and Backwash Control System
- Pneumatic backwash valves

- Colt Quincy Air Compressor

Figure 21 shows the Wheeler underdrain and media gradation in the effluent filters. Figure 22 shows the typical underdrain layout.

Figure 21 Wheeler Underdrain

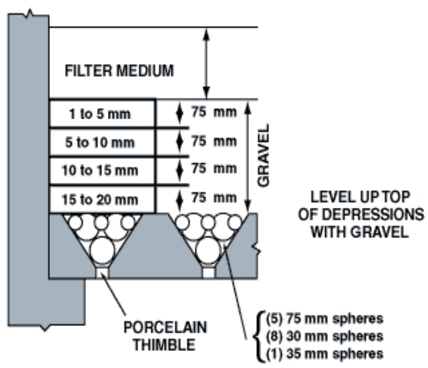
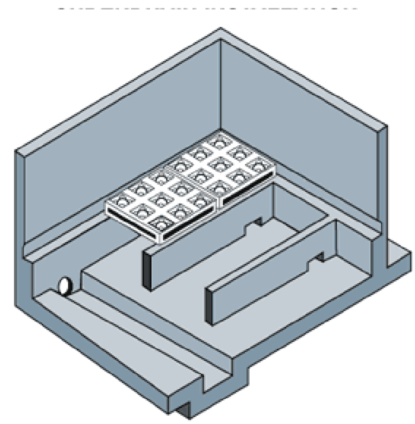


Figure 22 Typical Underdrain Layout



The Wheeler underdrain is typical of the false-bottom, open-plenum, single-pass type of underdrain. The Wheeler design uses cells across the entire bed bottom containing five ceramic balls, each. The ceramic balls lie in an inverted square pyramid that is open on the underside. One larger ball, measuring 3” in diameter is surrounded by four spheres 1-3/8” in diameter, each. The water passes through the balls as it does the sand and supporting gravel above it. During the backwash cycle, the water is forced through the balls by the specified backwash pressure, and the water creates an uplift force on the balls. These forces can cause the balls to spin, contact each other, and eventually wear down over time. These balls may need to be replaced in time. Also, there is typically not enough headloss in the nozzles or inverted pyramids to generate good flow distribution. The low headloss of the Wheeler underdrain is generally not adequate to control the media.

Once a filter of any type is in place, the life of the filter media is usually dependent on the support gravel. If the gravel begins to shift or "migrate”, the filter bed can become upset resulting in loss of media and poor performance.

6.11.2 Equipment Condition

Filter #5 was drained and inspected. The filter was originally supplied with a 24” layer of anthracite, little of which appeared to be remaining in the sample. Anthracite is the lightest media in the filter. In this type of filter, the anthracite is broken up over time and washes out with the waste during the backwash cycles. A filter of this type will normally lose up to 1” of anthracite a year so the lack of anthracite is within normal performance.

The surface of the media appeared to be distributed evenly across the filter without craters, hills, or separation near the walls. As the backwash cycle began there were no obvious sand boils. The even media layer prior to backwash and lack of sand boils during backwash indicate that the Wheeler bottoms are likely in fair to good condition.

A sample of the media was collected in Filter #5. This was a rough sample collection at one random location. The sand layer was roughly 1' thick with a deep gravel layer beneath. The original surface wash mechanisms were replaced at some point with a 'fixed nozzle' type surface wash. The filter nozzles, piping, and troughs appeared to be in good condition.

The field inspections of the filter were very preliminary. If no action is taken or the "Do Nothing" alternative is selected we would recommend moving forward with a more thorough level of inspection on the filters as follows:

Additional Filter Inspections

Level 1:

- Drain filters and inspect prior to backwashing. Look for craters, hills, separation by the walls.
- Inspect media surface as backwash begins, look for sand boils.
- Test depths from media surface to the coarse gravel, reference to a fixed elevation.
- Take representative number of samples.
- Send media sample to lab for sieve analysis – uniform coefficient, effective size

Level 2:

- Dig & inspect 6 under drain tiles spread throughout the filter.
- Check integrity of hoppers, porcelain balls and embedded porcelain nozzle at base of hopper.

Level 3 (outside specialties):

- Inspect base piers and undersides of the Wheeler underdrains. This can be done with a small camera inserted through the top of the hopper after the media is removed or it can be inspected from below. There doesn't appear to be an easy access point so a person or remote camera would have to enter through the 24" outlet pipe.

6.11.3 Effluent Filter Options

The effluent sand filters are slightly over loaded hydraulically compared to the original facility design; however, they are well within industry design standards. The TSS in the influent is below the design concentration and the removal efficiency is similar to design. Although the filters are currently performing satisfactorily, at higher influent flow rates, the effluent sand filters present a hydraulic bottle neck in the WRF.

At this time there are five general options for the existing filters. Any future regulatory requirements more stringent than existing limits may alter the recommendations for the filters. Although based on current plant performance data, the facility would meet the existing permit limits without the gravity filters in service, there is no guarantee that they are unnecessary at future flows and loads. The filters are a good performance safety factor against plant upsets at little financial operational costs.

Options for the existing gravity filters:

Option 1 – Do nothing

- At this time it appears that the filters are performing as required; however, the filters were identified in TM 2 as a limitation on future plant capacity during average day flows. The existing media has substantial biological growth and should be cleaned.

Option 2 – Add Anthracite

- The filters were originally installed w/ 24" of anthracite, little of which appeared to be remaining during inspection.
- 24" of anthracite would be added to all eight filter cells

COST - $240,000

Option 3 – Complete Rehabilitation of Wheeler System

3a Rehab only the out of service filter cell

- Remove media and replace with 12" sand and 24" anthracite
- Remove and replace support gravel
- Replace porcelain spheres
- Inspect wheeler bottoms and repair and/or line as needed
- Add 12" anthracite to other seven filter cells

COST - $279,000

3b Rehab all 8 filter cells

- Remove media and replace with 12" sand and 24" anthracite
- Remove and replace support gravel
- Replace porcelain spheres
- Inspect wheeler bottoms and repair and/or line as needed

COST - $1,400,000

Option 4 – Removal and Replacement of Wheeler System with New Filter Under Drain System

4a Replace with Block Underdrain System

- Demolition of existing Wheeler underdrain system
- Remove existing media and dispose
- New concrete support for block underdrain system
- Removal and replacement of wash troughs
- Media replacement
- Installation of block underdrain with air header
- Installation of airwash system
- Backwash controls modifications

COST: $2,310,000

4b Replace with False Bottom Underdrain System

- Demolition of existing Wheeler underdrain system
- Remove existing media and dispose
- False bottom concrete underdrain with nozzles
- Removal and replacement of wash troughs
- Media replacement
- Installation of airwash system
- Backwash controls modifications

COST: $2,228,000

Option 5 – Removal and Replacement of Wheeler System with Membrane Bioreactor

- Install membrane bioreactor.
- Modular cassettes for retrofit of existing filter bays.
- Zenon ultrafiltration (ZeeWeed 500) system or equal.

Option 5 is the most costly type of filtration system, but it achieves several objectives that the City may want to consider: (1) will meet all of the design objectives of the current gravity sand filters, (2) could delay the need for additional final clarification in the future, (3) the City will be well positioned for potential nutrient loading limitations in future NPDES permits such as phosphorus, etc. (4) could function as pre-treatment for a Reverse Osmosis (RO) water reuse facility if the City wants to sell treated effluent to prospective customers.

Filter Media Cleaning

The organic material attached to the media may be cleaned by super chlorinating at concentrations of 25 to 50 ppm. An acid wash may be performed to clean attached organic material along with iron, manganese, and calcium scale. Iron and manganese scale should not be an issue at the wastewater plant. Calcium scale due to the lime feed may be an issue but this typically shows up as a 'clumping' of the media particles. We did not see evidence of this in the filter media.

Super chlorination is an economical, first attempt, at removing the attached organic material. Liquid chlorine (sodium hypochlorite) or chlorine tablets may be the easiest method of applying chlorine to individual basins. The basin should be drained so that the water depth is approximately 2' above the media bed. Once the surface water is chlorinated, the water level is lowered to the media surface to allow the chlorinated water to penetrate the media bed. Maintain the chlorinated water in the media bed for a day and then gradually drain to dilute the chlorine concentration in the waste stream. Performing a sludge retention analysis of the media before and after the chlorination will provide a gauge of chlorination effectiveness. Super chlorinating 2' water in the filter to 50 ppm will require 3.6 gallons of sodium hypochlorite (12% solution) per basin or about 30 gallons total for the eight units. Chemical costs for this chlorination would be approximately $1,000 depending on shipping volumes and concentrations.

An in-basin acid wash can be performed on the media if chlorinating proves inadequate. Acid washing will remove mineral scale on the media particles. Sulfamic acid and oxalic acid are common acids used for wastewater sand media filters. Acid wash of the media should be explored if there is scale remaining on the media after super chlorination.

Underdrain System

It is recommended that the underdrain be replaced with a dual parallel lateral underdrain. The dual parallel lateral underdrain was developed to solve the flow distribution problems of other systems. It consists of a feeder lateral (central) and two compensating laterals (one on each side) which are parallel with each other. Figures 23 and 24 show a dual parallel lateral underdrain and a typical dual parallel lateral filter.

Figure 23 Dual Parallel Lateral Underdrain

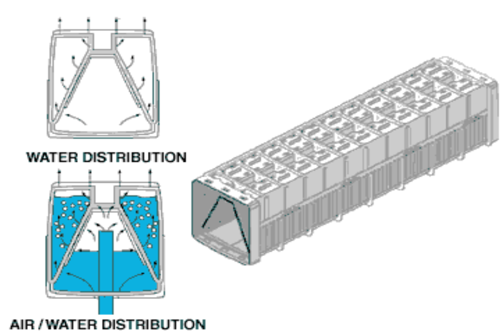
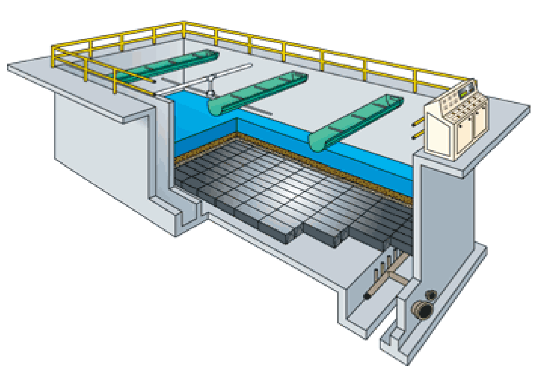


Figure 24 Typical Parallel Lateral System



The unique dual parallel lateral design enables the universal underdrain block to accomplish difficult-to-achieve uniform distribution without introducing high headloss (pressure drop). The universal underdrain introduces less than 16 inches of headloss when backwashing at a 15 gpm/sf (24 inch rise) in a lateral 20 feet long. An additional benefit of the universal underdrain is its unique ability to meter and uniformly distribute air. Widely spaced nozzles release large volumes of air creating disruptive "explosions" propelling media to the surface where it can be washed away. The triangular shape of the primary lateral collects incoming air uniformly along its length. Small dispersion orifices then distribute the air evenly to all the discharge orifices.

Media

The existing media gradation consists of a layer of support gravel, 12" of sand and 24" of anthracite. When the underdrains are replaced, a media pilot study should be conducted to determine the optimum media material and gradation. The effluent limits at the Sioux Falls WRF may not require a fine media such as anthracite. A coarse blend of media may be more economical to increase filter run times.

Underdrain manufacturers would likely be interested in participating in a pilot test at the facility. Specifics regarding a possible media pilot testing program follow.

- Anticipated pilot unit run time is 8 weeks
- Pilot unit would consist of a 29-foot semi trailer and includes all the necessary process equipment, pumps, valves, instruments, laboratory and ancillary equipment, etc. to function as a stand-alone testing unit. Connection of supply and waste lines would need to be completed in order to provide between 12 and 40 gpm.
- Pilot would investigate the applicability of utilizing deep-bed monomedia sand filtration with the block underdrain.
- Monomedia, deep-bed filters are becoming the trend now in wastewater, as they are composed of larger sand grains and have the capacity to store large amounts of solids, thereby lengthening filter run times. In addition, the media reacts better if the filter is hit with elevated levels of TSS. Backwash rates are lower (6gpm/sf) and they are run as a prolonged concurrent air-water wash.
- Anticipated cost is approximately $20,000.

6.12 Disinfection Alternatives

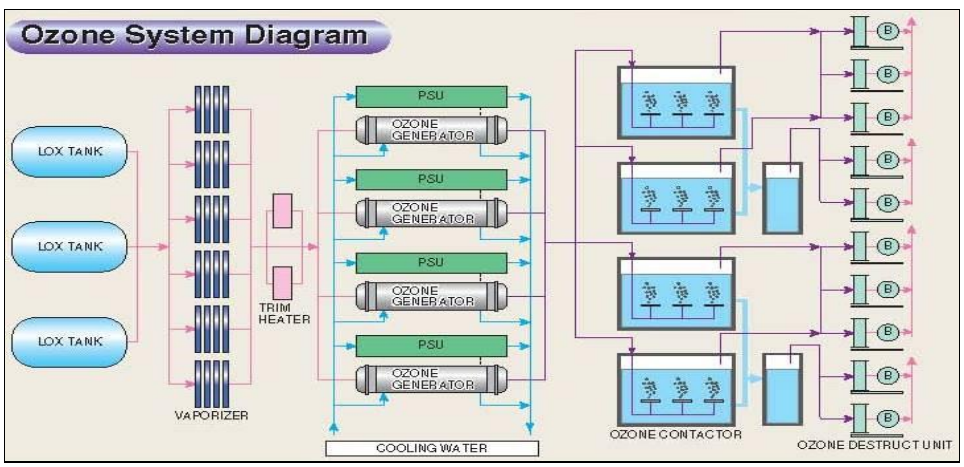
The City of Sioux Falls currently uses gaseous chlorine for effluent disinfection. Gaseous chlorine is a safety concern for not only accidental release at the facility but also intentional release. For that reason it is listed as a chemical of concern by government agencies as a homeland security risk. Even though there are no regulations prohibiting the use of gaseous chlorine at wastewater facilities at this time, it may be on the horizon. Also, the facility may want to consider alternate means of disinfection for operator safety and facility security reasons.

The main alternatives to gaseous chlorine for disinfection include ozone, ultraviolet radiation, and sodium hypochlorite.

<u>Option 1 – Ozone</u>

Ozone is an unstable gas that can destroy bacteria and viruses. It is formed when oxygen molecules (O_2) collide with oxygen atoms to produce ozone (O_3). Ozone is generated by an electrical discharge through dry air or pure oxygen. Ozone must be generated onsite because it decomposes to elemental oxygen in a short amount of time. After generation, ozone is diffused into a contact chamber containing the wastewater to be disinfected. The off-gases from the contact chamber must be treated to destroy any ozone remaining before release into the atmosphere. Figure 25 shows a schematic of the ozonation process.

Figure 25 Typical Ozone Disinfection System

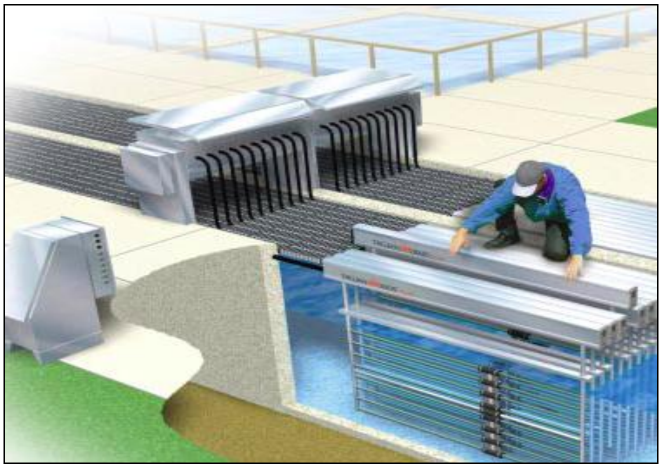


The Sioux Falls WRF produces low BOD and TSS effluent, which makes ozonation a viable alternative. Ozone disinfection is generally only economically feasible at medium- to large-sized plants. The capital costs to construct an ozone generation, contact, and destruct facility in Sioux Falls is approximately $9,000,000. The operational costs are high due to the amount of electricity required for the ozone generation process, estimated at approximately $160,000 per year.

Option 2 – Ultraviolet Disinfection

An ultraviolet disinfection system transfers electromagnetic energy to an organism's genetic material, destroying the organism's ability to reproduce. The effectiveness of a UV disinfection system depends on the characteristics of the wastewater, the intensity of UV radiation, the amount of time the microorganisms are exposed to the radiation and the reactor configuration. A typical UV disinfection system is shown in Figure 26.

Figure 26 Typical UV Disinfection System

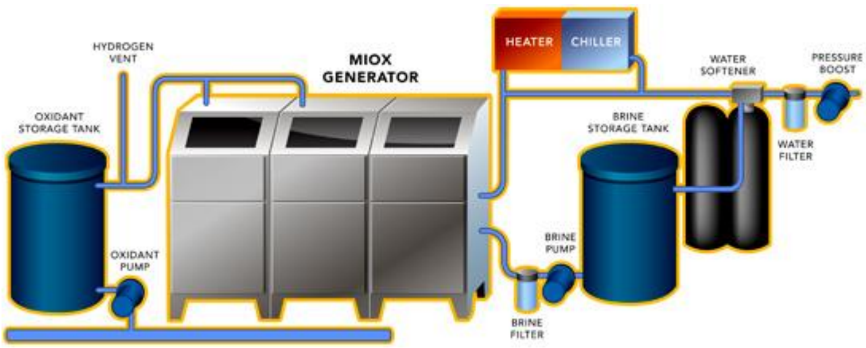


The Sioux Falls WRF produces low solids effluent, which makes UV disinfection a viable alternative. The UV system could likely be retrofit to the existing chlorine contact basin, reducing capital costs. The cost to provide a self-cleaning UV disinfection system in the existing chlorine contact basin is approximately $2,006,000.

Option 3 – On-Site Sodium Hypochlorite

On-site hypochlorite systems typically consist of utilizing the process of salt electrolysis to produce a chlorine disinfectant solution. Major components are bulk salt storage, generation cabinets and solution storage tanks. The following diagram shows a typical system layout.

Figure 27 Typical Sodium Hypochlorite System



Based on a future peak day flow of 28 mgd, (peak hourly flow of 54 mgd) it is anticipated that four 500 pound per day units will be eventually necessary, although three would be initially installed to meet existing needs. Two units would be on-line with one redundant unit on stand-by. The approximate cost to install three units is $1.6 million. This includes generation units and all ancillary equipment. No allowances have been made for building additions which may be necessary to accommodate the equipment.

Table 8 Disinfection Alternative Costs

Treatment	Capital Cost	Annual O&M
Ozone[1]	$9,000,000	$160,000
Ultraviolet	$2,006,000	$34,400
Sodium Hypochlorite[1]	$1,600,000	$65,000
Gaseous Chlorine	$0	$32,200

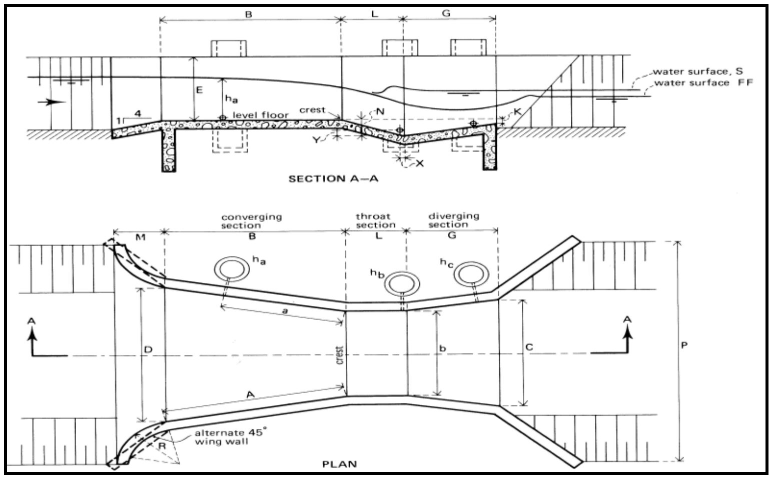
[1] Costs do not include building to house equipment if needed.

6.13 Effluent Flow Meter

The effluent flows are measured with a Parshall flume. Parshall flumes are designed so that flow velocities increase through the throat and cause a hydraulic jump, or standing wave, just downstream of the flume (see 'water surface FF' in Figure 28). Parshall flume installations are designed so that the discharge levels are not sufficient to reduce flows. This condition is called 'free flow' and allows for an accurate flow reading based on a single upstream level measurement (point H_a in Figure 28).

'Submerged-flow' occurs when downstream levels backup to the point where flow velocities through the flume are reduced, causing an increase in flow depth. As the flow becomes submerged, the standing wave will move towards the throat (water surface S in Figure 28). High submergence conditions require both upstream (H_a) and downstream (H_b) level measurements for accurate flow measurement.

Figure 28 Parshall Flume



The effluent flume is configured so that the discharge drops vertically into a channel before taking a right turn and entering a pipe. This is an unconventional configuration which causes turbulence and increased levels in the discharge channel. Based on the operating characteristics below, the effluent flume may at times operate in a submerged condition.

Location	Effluent
Flume Size	48”
Approach Channel Elevation	1324.0
Discharge Pipe Diameter	48”
Discharge Pipe C.L	1322.0
Downstream Weir Elevation	1323.5
Available Downstream Head	0.5’

The submergence ratio of a flume (H_b/H_a) is the ratio of the downstream depth (H_b) to the upstream depth (H_a), measured at the locations shown in Figure 28. The discharge of a 48” Parshall flume is reduced when the submergence ratio exceeds 0.70 (Bureau of Reclamation Water Measurement Manual 2001 edition). At submergence levels greater than 0.70 a correction factor based on downstream head needs to be used. Additional checking of the upstream (Ha) and downstream (Hb) levels should be done at the effluent flume to see if submergence is occurring at the higher flow rates.

The upstream flow in the effluent flume is also cause for concern. The Parshall flume discharge equations are based on smooth, even, flow entering the flume. The flow drops into the upstream channel at a right angle. This causes significant turbulence in the approach channel and an uneven flow profile through the throat. There is also a small volume expansion caused by the entrained air. There are no correction factors for these conditions. The induced error caused by the upstream conditions may be small (<10%) but the only way to know is to compare it to another flow measurement such as the influent flume.

Although the effluent flume installation is poor, a Parshall flume is a very forgiving measuring device so the accuracy may not be significantly compromised. It may be difficult to prevent submergence of the flume do to the weir elevation at the cascade aerator. Laying a straight pipe behind the effluent flume may reduce the turbulence but probably not the submergence. Submergence can be corrected for but may require an additional downstream flow meter or additional control system logic.

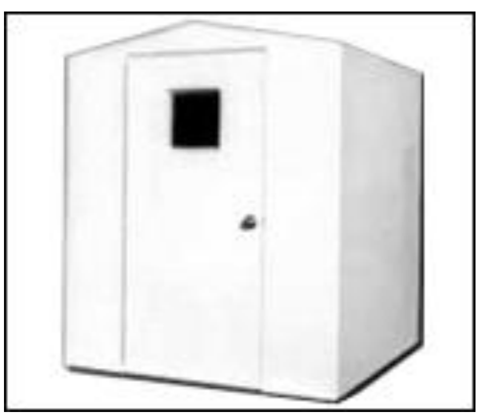
Another possible option is to measure flow over the weir at the cascade aerator. The existing weirs are approximately 10’ each which is too large for accurate measurement at the plant flows. These weirs could possibly be replaced with square-notched weirs, with each weir approximating a 5’ weir with end contractions. With this arrangement, flow could be measured with a single meter located in the center of the structure. The acceptability of this option with the approving agency has not been investigated.

6.14 Sampling

The existing plant samplers are located in the Headworks Building, Process Pump Station and in the In Plant Pump Station. The sample flow is delivered to the samplers via sample lines run throughout the facility. Some of the sample flows are pumped and some are gravity flow only. Based on our evaluation of historical plant data it appears that some biological treatment is occurring in the sample lines and thus giving inaccurate results. The influent and effluent samples, which are reported as part of permitting requirements, are not affected.

The most common practice for new facilities is to locate influent and effluent samplers directly at their respective sampling points. This minimizes sample tubing lengths and is can more reliably achieve a representative sample. Most plant influent and effluent samplers are composite type but these can also be provided with individual bottles for sequential sampling if needed. Outdoor samplers include heaters to maintain correct sample temperatures during cold weather as shown in Figure 29.

Figure 29 Typical Sampler Enclosure



We recommend that the City continue to sample the internal process components of the WRF to monitor treatment performance. Remote sampling facilities have been installed by WRF personnel to achieve representative samples of the internal processes.

6.15 Emerging Contaminants

There is increasing discussion amongst regulatory agencies and the public regarding endocrine disrupting chemicals (EDCs) which are being found in significant concentrations in wastewater treatment plant effluent. Although EDC's have not been truly defined by the USEPA they are in the process of defining exactly what an EDC is and those chemicals that meet the toxicity definition will be classified as such in the coming years. Generally EDC's are natural or synthetic chemicals such as pharmaceuticals, personal care products, pesticides, etc that interfere with or mimic the hormones responsible for growth or development of an organism. To date the effects of EDCs on human health is unknown, but there is evidence of its negative effect on wild life health.

There are advanced technologies on the market that will provide for EDC removal if EPA and/or the SD DENR decides to regulate EDC levels in the City's NPDES permits. The advanced technologies for EDC removal include:

- Activated Carbon Adsorption
- Ozonation
- Advanced Oxidation Processes (AOPs) – UV/Peroxide, UV/Ozone, etc.
- Reverse Osmosis (RO) and Tight Nanofiltration (NF) Systems

There is a tremendous amount of research currently being conducted by the Water Environment Federation (WEF) and the American Water Works Association (AWWA) regarding EDCs. We have been in contact with various vendors in order to budget for the cost of a treatment system for EDC removal if necessary some day at the Sioux Falls WRF. The two most feasible options at this time for EDC treatment are ozonation or UV-Peroxide. The use of these systems for EDC treatment is new technology and design information is limited to pilot studies. The following shows the costs for implementing ozone and UV for disinfection purposes, the additional costs to provide treatment for EDCs and the total costs. The existing annual costs for disinfection at the facility are also shown for comparison purposes.

Table 9 Emerging Contaminant Costs

Treatment	Capital Cost	Annual O&M
Ozone	$18,870,000	$448,000
Ultraviolet/Peroxide	$18,230,000	$295,400

6.16 Gravity Thickeners

The purpose of the gravity thickeners is to increase the sludge thickness and optimize the use of the digesters by minimizing heating requirements and sludge processing through reduced sludge volumes. There are two, 55' diameter, gravity thickeners at the WRF.

6.16.1 Equipment Description

The gravity thickeners receive sludge from the primary clarifiers. The plows on the bottom of the rotating truss arms serve to agitate and thicken the settled sludge and also to direct it to the central sludge hopper. From the central sludge hopper the thickened sludge is pumped out by the sludge pumps located in the tunnel. The thickeners were manufactured by Keene/Amwell of Aurora, IL, model 42H, serial numbers 90031-1 and -2.

6.16.2 Equipment Condition

One gravity thickeners is in service during normal plant operation. Gravity Thickener #2 was taken out of service and field inspected. As shown in Figure 30, the truss arms showed severe metal deterioration.

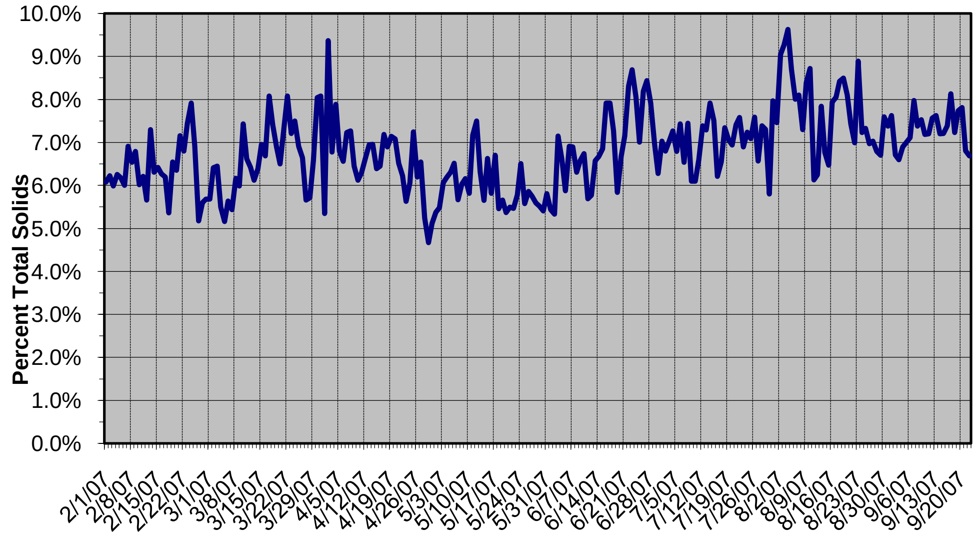
Figure 30 Gravity Thickener Truss Arms

The deterioration was especially evident at the end portion of the truss arms and on the vertical supports. The influent well appears to have lost nearly 50% of its metal thickness in areas. The corrosion on the influent well and center column are likely occurring in the lower areas as well. A thorough cleaning and inspection of the lower portions of the influent well, center column, and drive cage was not performed as part of this inspection.

6.16.3 Equipment Performance

The design goal for the gravity thickeners is to thicken the various sludge sources—raw, humus and waste activated—from 3% to 6.0 to 6.5%. Listed below are the 2007 thickened sludge concentrations from the gravity thickeners. The gravity thickeners appear to be meeting the design objectives.

Figure 31 2007 Gravity Thickener Percent Solids (One in Service)



6.17 Digester mixing system and treatment capacity

The anaerobic digestion system at the Sioux Falls WRF is comprised of three (3) anaerobic digesters and one (1) secondary digester. Each digester is 65 feet in diameter, has a side water depth of 31 feet and an active volume of 0.77 million gallons. Total primary digester capacity is 2.3 million gallons.

6.17.1 Equipment Description

The original anaerobic digestion system was comprised of the following components:

- Draft Tube Mixing System
- Digester Covers
- Digester Heating
- Sludge Recirculation Pumps
- Sludge Transfer Pumps

<u>Draft Tube Mixing System</u> – The primary digesters were equipped with a draft tube mixing system as part of the original equipment design.

<u>Digester Floating Covers</u> – Each primary digester is equipped with a floating steel cover. The secondary digester is equipped with a gas holder type floating steel cover.

Digester Heating – The primary digestion system is heated using heat exchangers and waste heat generated by two (2) 445 kW engine generators used for energy recovery. The raw primary sludge is heated up to temperatures of approximately 98 deg. F before it enters the primary digesters. The heat exchangers have the ability to heat sludge at a rate of 2.0 million BTUs/hr. The engine generators are fueled by methane gas which is produced from the digestion process.

Sludge Recirculation Pumps – Sludge recirculation pumps are used to move the contents from within the digesters through the aforementioned heat exchangers and back into digesters again. Heat is transferred to the sludge utilizing hot water from within the heat exchangers.

Sludge Transfer Pumps – These pumps are used to transfer sludge between the various digesters.

6.17.2 Equipment Condition

The digestion equipment is starting to show signs of aging. Listed below is a summary of the major components and the condition of each.

Draft Tube Mixing System – The draft tube mixing system is not working and thus mixing of the digester contents is done through recirculation pumping alone. Further discussion on this subject can be found below under equipment performance.

Digester Floating Covers – The floating steel covers are starting to show signs of metal deterioration due to age. A metal thickness evaluation of the covers should be conducted using ultrasonic testing methods. Once the thickness is known a determination of cover life can be projected. The cost to retain the services of a testing company to evaluate the four digester covers is $2,000. This first phase of investigation would include a test of the covers in random locations to pinpoint possible corrosion areas. If corrosion areas are identified, a more detailed grid-pattern testing procedure could be used. A detailed grid-pattern test of one digester cover would cost $3,000-$4,000. By doing “spot-checks” of the covers initially, the detailed grid testing can be focused to the areas of concern only, saving substantial testing costs.

Digester Heating – The digester heating system appears to be in good condition at this time however some parts that are required for replacement may be difficult to find according to City Staff.

Sludge Recirculation Pumps – The sludge recirculation pumps are showing signs of wear and age. They are reaching the end of their useful life and should be programmed for replacement in the next five years.

Sludge Transfer Pumps – The sludge transfer pumps are also showing signs of wear and should also be programmed for replacement.

6.17.3 Equipment Performance

Table 10 shows performance data from the WRF anaerobic digestion system. All performance data appears to be within the range of acceptable design standards.

Table 10 Anaerobic Digester Performance Data

Anaerobic Digester	Facility Design	Facility Performance	Recommended Standard
Solids Retention Time	20 days	28.5 days	10 days [(2)]
Volatile Solids Reduction	40%	56.9%	60.0% [(2)]
Volatile Solids Loading Rate	0.11 $lbs/ft^3/day$	0.11 $lbs/ft^3/day$	0.13 $lbs/ft^3/day$ [(2)]
Raw Sludge % Solids	5%	6.4%	-
Gas Production per Volatile Solids Destroyed	15.80 ft^3/lb	13.94 ft^3/lb	12-18 ft^3/lb [(2)]

[(1)] Great Lakes-Upper Mississippi River Recommended Standards for Wastewater Facilities
[(2)] Metcalf & Eddy Wastewater Engineering Treatment and Reuse

Draft Tube Mixing System – The gas mixing system is not in operation due to equipment failure. The only mixing currently taking place in the digesters is from the sludge recirculating pumps used for heating.

Results from the lithium tracer study, conducted by South Dakota State University, indicate that adequate mixing for treatment is occurring in the digesters. However, the lithium is a soluble chemical and would not reflect mixing characteristics of settable solids and grit. This is supported by the fact that volatile solids reduction and methane gas production appear to be within acceptable standards. However, settable solids and grit do accumulate on the bottom and decrease the overall digester volume. Reference books indicate the mixing requirement for keeping grit and other solids from settling is five times the mixing requirement for maintaining treatment. A new mixing system in the primary digesters would keep the solids from settling; however, the settling would occur in the secondary digesters. There are inline screens to remove solids larger than 3mm but smaller particles such as grit would not be separated. Grit removal options should be investigated during preliminary design if a new mixing system is desirable at the facility.

Based on the accumulation of solids and grit in the digesters and the fact that more digester volume will be required at some point due to growth of the City, a new mixing system should be installed. The new mixing system should be similar to the Vaughn Rotamix System. We suspect that a new mixing system may also aid the current treatment which will potentially produce more digester gas and thus provide a pay back on electrical usage at the WRF.

Digester Floating Covers – The covers are currently providing adequate containment of gas and solids however they are beginning to show signs of wear and should be evaluated in greater depth through the use of ultrasonic technology.

Digester Heating – Performance of the digester heating system is adequate at this time.

Sludge Recirculation Pumps – The sludge recirculation pumps are working however due to wear and tear they should be replaced in the next five years.

Sludge Transfer Pumps – The sludge transfer pumps are working however due to wear and tear they should be replaced in the next five years.

6.18 Sludge Pumping

The facility's existing piston transfer pumps are difficult to operate and maintain. The existing recirculation pumps could be reconfigured so they would perform both transfer and recirculation operations. The modifications to reconfigure the existing recirculation pumps include automated valves, piping changes, fittings and labor. The estimated cost to modify the recirculation pumps is $60,000.

7.0 FUTURE FLOW AND LOADING PROJECTIONS

7.1 Population Projections

The City of Sioux Falls planning department has projected the following City population growth. The growth shown below is approximately 1.4% per year to the year 2030.

Year	Projected
2007	144,000
2010	148,500
2015	156,000
2020	170,500
2025	185,000
2030	199,600

7.2 Future Conditions

To predict future influent flow and loading conditions at the Sioux Falls Water Reclamation Facility, the City Planning Department was contacted to determine population projections through the year 2030.

7.2.1 Average Annual Daily Flow and Loading

The average annual daily flow (ADF) and loadings are the influent conditions averaged over a year. The 2000-2006 average annual flows and loadings were shown previously in Table 2. These figures include current industrial flow and load contributions.

Historical wastewater flows and loads based on City population and existing per capita contributions on an annual average basis are shown in Table 3. The per capita flows and loads for Sioux Falls are within the typical ranges expected for Cities of similar size.

Future projections for average annual daily flow and loadings are shown in Table 11 on the following page. The projections reflect industrial contributions similar to current industrial flow and load conditions projected to the year 2030.

Table 11 Projected Average Annual Daily Flows and Loads

Year	Population	ADF (mgd)	BOD (lb/d)	TSS (lb/d)	NH3-N (lb/d/cap)	TKN (lb/d/cap)
2007	144,000	15.26	31,680	31,680	2,880	5,760
2010	148,500	15.74	32,670	32,670	2,970	5,940
2015	156,000	16.54	34,320	34,320	3,120	6,240
2020	170,500	18.07	37,510	37,510	3,410	6,820
2025	185,000	19.61	40,700	40,700	3,700	7,400
2030	199,600	21.16(1)	43,912	43,912	3,992	7,984
Projection Value(2)		**106 gal/cap/d**	**0.22 lb/cap/d**	**0.22 lb/cap/d**	**0.02 lb/cap/d**	**0.04 lb/cap/d**

(1) The projected flow or load is outside of proposed re-rate capacity (21.0 mgd).

(2) Projection Value was determined in Table 3 from existing WWTF data.

7.2.2 Projected Average Daily Dry Weather Flow

The average daily dry weather flow (ADDF) is when the ground water is at or near normal and a runoff condition is not occurring. The present average daily dry weather flow is used as a baseline flow. The planned population increase and resultant projected flow is added to the baseline dry weather flow. A per capita flow of 85 gallons per day was anticipated based on current dry weather (the ADDF for December of 2006 was used for the representative dry weather data). Table 12 below illustrates the average dry weather flow determination for the year 2030.

Table 12 2030 Average Daily Dry Weather Flow Calculations

Condition	Flow
1. Present average daily dry weather flow (ADDF)	13.55 MGD
2. Population increase 57,100 @ 85 gpcd (85 gpcpd does not include 2006 industrial flow of approximately 1.1 mgd)	4.85 MGD
3. Average flow from planned industrial increase	1.10 MGD
4. Estimated average flow from other future unidentified industries	0.50 MGD
5. Average flow from other future increases	0.50 MGD
Average daily dry weather design flow [(1)+(2)+(3)+(4)+(5)]	**20.50 MGD**

The method presented above was used for each of the 5 year planning increments to the year 2030. Table 13 shows the projected average dry weather flows.

Table 13 Average Daily Dry Weather Flow Projections

Year	Population	Population Increase	ADDF
2006	142,500	0	13.55
2010	148,500	6,000	14.46
2015	156,000	13,500	15.50
2020	170,500	28,000	17.13
2025	185,000	42,500	18.76
2030	199,600	57,100	20.50

7.2.3 Projected Peak Month Flow

Peak month flow (PMF) is the average daily flow for the wettest consecutive 30 days. The present average dry weather flow (13.55 mgd) is used as a baseline flow. The planned population increase and resultant projected flow is added to the baseline dry weather flow. A per capita flow contribution of 100 gallons per day was used based on current wet weather flow and published design criteria. The average infiltration and average inflow were estimated from flow records from April of 2006. The average flow from planned and unplanned industry and other unknown flows were projected to be similar to existing industrial flows. The average dry flow, infiltration, inflow, future residential flow, and future industrial flows were added to estimate a design peak monthly flow. Table 14 below illustrates the peak month flow determination for the year 2030.

Table 14 2030 Peak Month Flow

Condition	Flow	
1. Present average daily dry weather flow (ADDF)	13.55	MGD
2. Average infiltration	2.00	MGD
3. Average inflow	4.33	MGD
4. Population increase 57,100 @ 100 gpcd	5.71	MGD
5. Average flow from planned industrial increase	1.10	MGD
6. Estimated average flow from other future unidentified industries	0.50	MGD
7. Average flow from other future increases	0.50	MGD
Peak Month Flow (PMF) [(1)+(2)+(3)+(4)+(5)+(6)+(7)]	**27.69**	**MGD**

The method presented in Table 14 was used for each of the 5 year planning increments to the year 2030. Table 15 shows the peak month flow projections.

Table 15 Peak Month Flow Projections

Year	Population	Population Increase	PMF
2006	142,500	0	19.88
2010	148,500	6,000	20.88
2015	156,000	13,500	22.03
2020	170,500	28,000	23.88
2025	185,000	42,500	25.73
2030	199,600	57,100	27.69

7.2.4 Projected Peak Hourly Wet Weather Flow

A peak hourly flow is the flow during the peak hour of the day at a time when the ground water is high and a five-year storm is occurring. The peak hourly flow is used in 10 State Standards for a number of peak event sizing considerations for processes including clarifiers, chlorine contact tanks and effluent filters. A number of flow conditions are used to first determine existing inflow and infiltration and ultimately a design peak hourly flow. The flows listed in Figure 1 were influent plant flows downstream of the 12.5 million gallon equalization basin and Brandon Road Pump Station. The projected peak flows shown on the following pages reflect the flow capacity required in excess of the existing equalization basin.

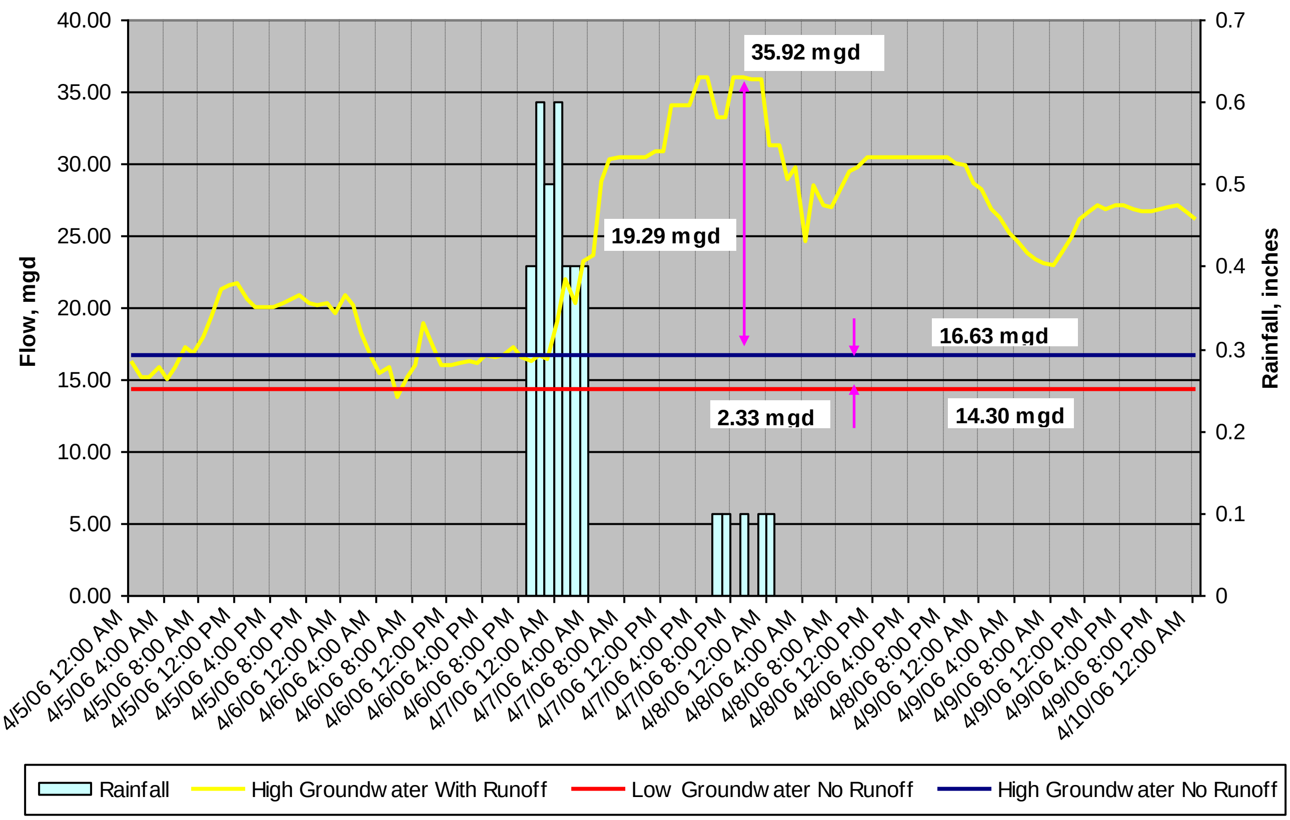
A number of rain events were evaluated to determine peak flow conditions. A list of storm events studied follows.

- ***May 29, 2004*** – This was a 25 to 50 year storm event. Plant influent data from this storm is limited. Inflow and infiltration reduction in the collection system have occurred since so this would not be representative of future flows.
- ***June 16, 2004*** – Many project this as a 100 year storm event, however, the official rain gauges were damaged in the storm. Official rain amounts classify this event as a 25 year storm. Plant inflow and infiltration reduction in the collection system have occurred since so this would not be representative of future flows.
- ***April 6, 2006*** – This storm was between a 5 and 10 year event. The plant has good hourly flow records and equalization basin flow records. The rainfall data does not appear to be flawed.
- ***March 31, 2007*** – This storm was between a 2 and 5 year event. Although flow data for this storm is adequate, at least a 5 year event was needed for projections.

- ***August 4, 2007*** – This storm was between a 2 and 5 year event. Although flow data for this storm is adequate, at least a 5 year event was needed for projections.

Based on the rain events studied, the April 6, 2006 storm appears to be the event to project future peak flows. The influent plant flows during the storm were plotted with the hourly rainfall data. The influent plant flow during the storm represents a period when groundwater is high and an inflow event is occurring (yellow line in Figure 32). Also, the daily flow when groundwater is low and an inflow event is not occurring was plotted on the same graph (red line in Figure 32). This corresponded to winter flows; February 6, 2006 was used in this analysis. The daily flow when groundwater is high and an inflow event is not occurring was also plotted (blue line on Figure 32). This corresponds to spring flows when rain is not occurring; May 18, 2006 was used in this analysis. These flows are shown in Figure 32.

Figure 32 Peak Flow Analysis



The flows shown in Figure 32 were used to calculate existing infiltration and inflow. The hourly flow during high ground water and no inflow (16.63 mgd) minus the dry weather flow (14.30 mgd) equates to the existing infiltration, 2.33 mgd. The peak hourly flow during high ground water period (April 2006 storm) was 35.92 mgd. That value minus the infiltration and dry weather flow (16.63 mgd) equals the peak hourly inflow, 19.29 mgd. These calculations along with the peak flows attributed to population, industrial and unforeseen growth are shown below in Table 16.

Table 16 2030 Peak Hourly Wet Weather Flow

Condition	Flow	
1. Present peak hourly dry weather flow (Red Line Fig. 59)	14.30	MGD
2. Present peak hourly flow during high ground water period (Blue Line Fig. 59)	16.63	MGD
3. Present peak hourly dry weather flow [same as (1)]	14.30	MGD
4. Present peak hourly infiltration	2.33	MGD
5. Present hourly flow during high ground water period and runoff conditions (Yellow Line Fig. 59)	35.92	MGD
6. Present hourly flow during high ground water and no runoff at same time of day as (5) measurement (Blue Line Fig. 59)	16.63	MGD
7. Present peak hourly inflow (5)-(6)	19.29	MGD
8. Present peak hourly inflow adjusted for a 5-year 1-hour rainfall event	19.29	MGD
9. Present peak hourly infiltration [same as (4)]	2.33	MGD
10. Peak hourly infiltration cost effective to eliminate	0.00	MGD
11. Peak hourly infiltration after rehabilitation	2.33	MGD
12. Present peak hourly adjusted inflow [same as (8)]	19.29	MGD
13. Peak hourly inflow cost effective to eliminate	0.00	MGD
14. Peak hourly inflow after rehabilitation (where rehabilitation is cost effective)	19.29	MGD
15. Population increase 57,100 @ 100 gpcd times 2.5 (peaking factor)	14.28	MGD
16. Peak hourly flow from planned industrial increase	2.20	MGD
17. Estimated peak hourly flow from future unidentified industries	1.00	MGD
18. Peak hourly flow from other future increases	1.00	MGD
Peak hourly wet weather design flow (PHF) [(1)+(11)+(14)+(15)+(16)+(17)+(18)]	**54.40**	**MGD**

The method presented above was used for each of the 5 year planning increments to the year 2030. Table 17 shows the peak month flow projections.

Table 17 Peak Hourly Flow (PHF) Projections

Year	Population	Population Increase	PHF
2006	142,500	0	35.92
2010	148,500	6,000	38.22
2015	156,000	13,500	40.90
2020	170,500	28,000	45.32
2025	185,000	42,500	49.75
2030	199,600	57,100	54.40

7.2.5 Projected Peak Instantaneous Wet Weather Flow (PIF)

A peak instantaneous flow is defined in this study as the flow during the peak hour of the day at a time when the ground water is high and a twenty five-year storm is occurring. The peak instantaneous flow is often used to determine sizing for screening, grit and flow equalization. The peak hourly flow was adjusted to a 25-year storm event to achieve a peak instantaneous flow, as shown in Table 18 below.

Table 18 Peak Instantaneous Wet Weather Flow

Condition	Flow	
1. Peak hourly wet weather design flow	54.40	MGD
2. Present peak hourly inflow adjusted for a 5-year 1-hour rainfall event	19.29	MGD
3. Present peak inflow adjusted for a 25-year 1-hour rainfall event	24.55	MGD
Peak instantaneous wet weather design flow (PIF) [(1)–(2)+(3)]	**59.66**	**MGD**

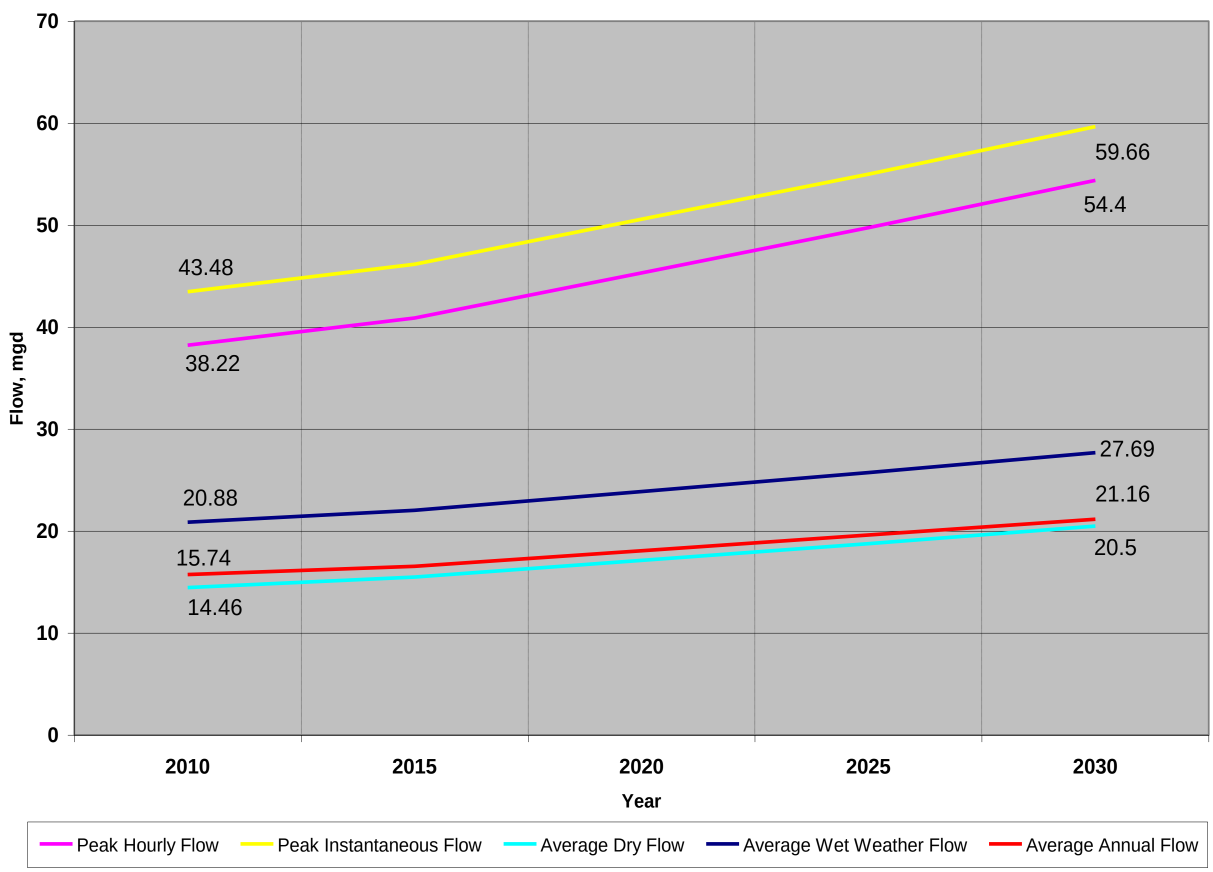
The method presented above was used for each of the 5 year planning increments to the year 2030. Table 19 shows the peak instantaneous flow projections.

Table 19 Peak Instantaneous Flow (PIF) Projections

Year	Population	Population Increase	PIF
2006	142,500	0	41.18
2010	148,500	6,000	43.48
2015	156,000	13,500	46.16
2020	170,500	28,000	50.58
2025	185,000	42,500	55.01
2030	199,600	57,100	59.66

The flow projections from 2010 through 2030 were plotted for Average Annual Flow, Average Daily Dry Weather Flow, Peak Month Flow, Peak Hourly Flow, and Peak Instantaneous Flow are shown in Figure 33 below.

Figure 33 2010 through 2030 Flow Projections



7.2.6 Projected "25-year, 24-hour Storm" Simulation

The "25-year, 24-hour Storm" simulation was generated as follows:

- May 7th was used for the base wet-weather flow since equalization was off-line and it represented a typical wet-weather day.
- The April 7th, 2006 storm was used to determine the rainfall induced I/I by subtracting the base wet-weather flow.
- The April 7th, 2006 storm was adjusted to actual flows by adding in the calculated flow determined by changes in the equalization level/volume. (The existing equalization and clarifier drain valves are maintained in the open position. As such, the equalized flow meter measurement is not valid.)
- The April 7th, 2006 rainfall induced I/I was then increased to a 25-year, 24-hour storm which was a factor of 26.3%.

The following figures show the design flow for a 25-year 24-hour storm event for 2020 and 2030.

Figure 34 2020 25-year 24-hour Design Flow

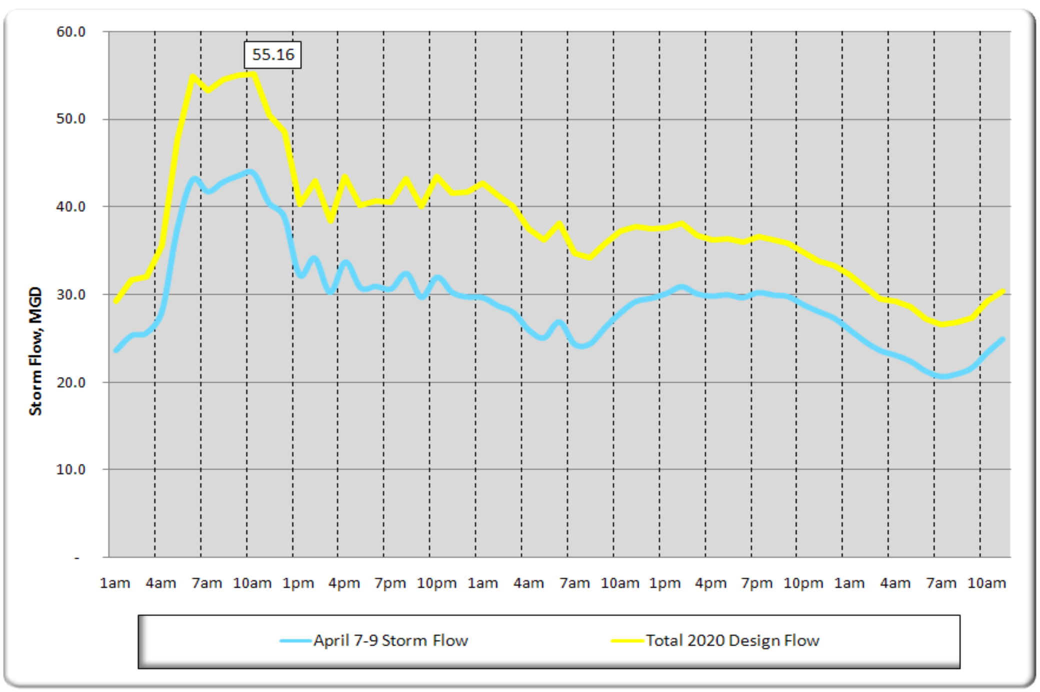
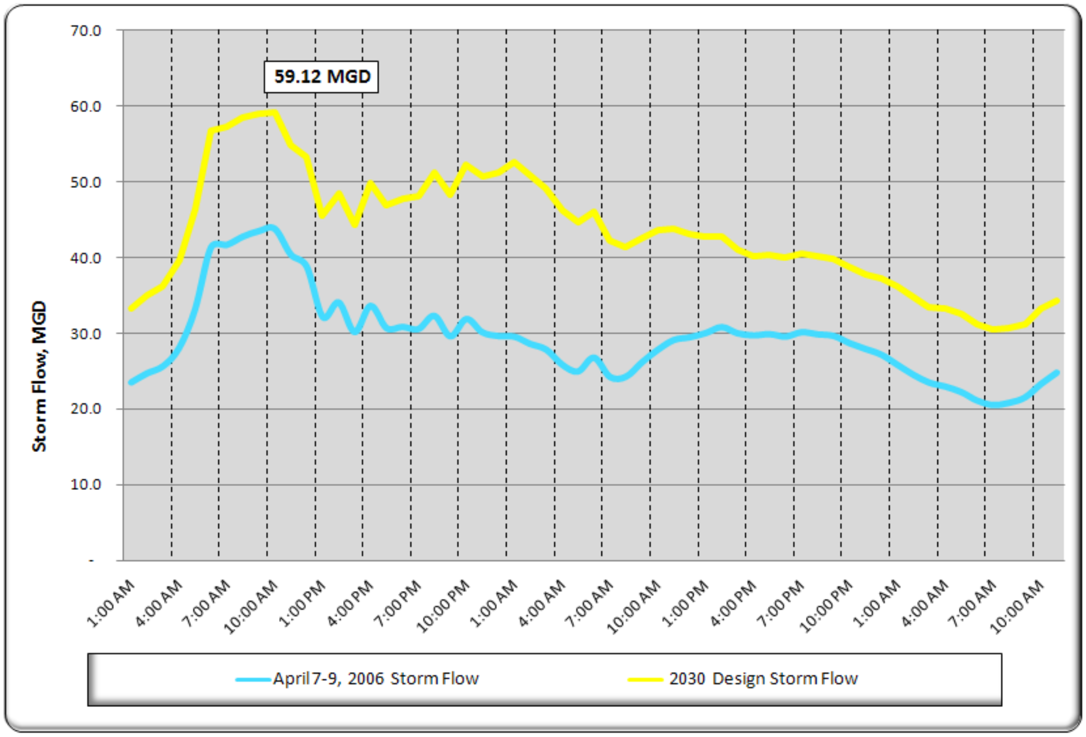


Figure 35 2030 25-year 24-hour Design Flow



8.0 ALTERNATIVES ANALYSIS

Based on the future flows predicted in Section 7.0 and the re-rated facility capacity from Section 5.0, expansion alternatives to meet future flows can be developed.

8.1 Capacity Availability

The capacity available for each unit process is shown in Table 20 below. The capacities are based on regulatory calculations unless noted otherwise. Based on the flow projections listed in Table 11 for ADF and Table 17 for PH, each capacity will be reached in the approximate year shown. The yellow highlights correspond to average daily capacities; the green highlights correspond to peak hourly flow capacities and the orange highlights correspond to equipment limitations.

Table 20 Unit Process Capacity Summary

Unit Process	Constraint	Capacity	
Effluent Filters(1)	Average Daily	22.00	mgd
Primary Clarifiers	Average Daily	25.43	mgd
FS Intermediate Clarifiers	Peak Hourly	25.90	mgd
SS Intermediate Clarifiers	Peak Hourly	25.90	mgd
First Stage TF(1)	Distributor Capacity	27.00	mgd
Second Stage TF(1)	Distributor Capacity	30.00	mgd
Primary Clarifiers	Peak Hourly	30.52	mgd
Process Pump Station(2)	Pumping Capacity	31.30	mgd
Final Clarifiers	Peak Hourly	31.83	mgd
Effluent Filters	Peak Hourly	33.29	mgd
Brandon Road Pump Station(3)	Pumping Capacity	35.00	mgd
Chlorine Contact(4)	Peak Hourly	41.00	mgd
Screening	Peak Hourly	52.00	mgd
Activated Sludge	Peak Hourly	-	mgd
Aerated Grit Chamber	Peak Hourly	72.00	mgd

(1) Flow capacity is based on hydraulic model and field testing.
(2) Flow capacity of PPS with bypass valve to filters 50% open based on HDR field testing.
(3) New flow capacity of BRPS due to changes at headworks facility.
(4) Flow capacity is based on chlorine dosing upstream of effluent filters.

8.2 Expansion Alternatives

The existing WRF is well positioned at this time to handle ***average annual daily flows*** (ADF) now and into the future as shown in Table 20. However, during wet weather conditions the WRF is unable to maintain the recommended standards for weir loading, surface overflow rates and detention times in the various sedimentation tanks (primary, intermediate and final clarifiers). Also, field testing and modeling of various facilities at the WRF have revealed hydraulic limitations during high flows. Some of the facilities that show restrictive tendencies at higher flows include the first and second stage trickling filter distributor arms, process pumping station and the effluent filters. Even though the recommended standards cannot be met during peak flow events and hydraulic restrictions occur, it must be noted the WRF has not violated its NPDES Permit requirements. The Re-rate Document further explains the current peak flow capacity (35.0 mgd) determination.

To handle peak wet weather flows at the WRF today and into the future, various alternatives were evaluated. Two basic conceptual alternatives were evaluated to allow the WRF to handle projected peak flows:

- construct equalization storage which would maintain peak flows to the WRF at the current capacity (35 mgd) or
- expand unit processes throughout the plant to meet projected flows.

Each conceptual alternative included variations for location, phasing, and ancillary improvements necessary.

8.2.1 Alternative No. 1 – Construct Equalization Storage Basin

As mentioned previously peak flows at the WRF can be difficult to handle at times. One alternative to handle the peak wet weather flows now and in the future is to construct equalization storage. This would allow only the peak flow capacity (35.0 mgd) to reach the plant during wet weather events.

Sizing

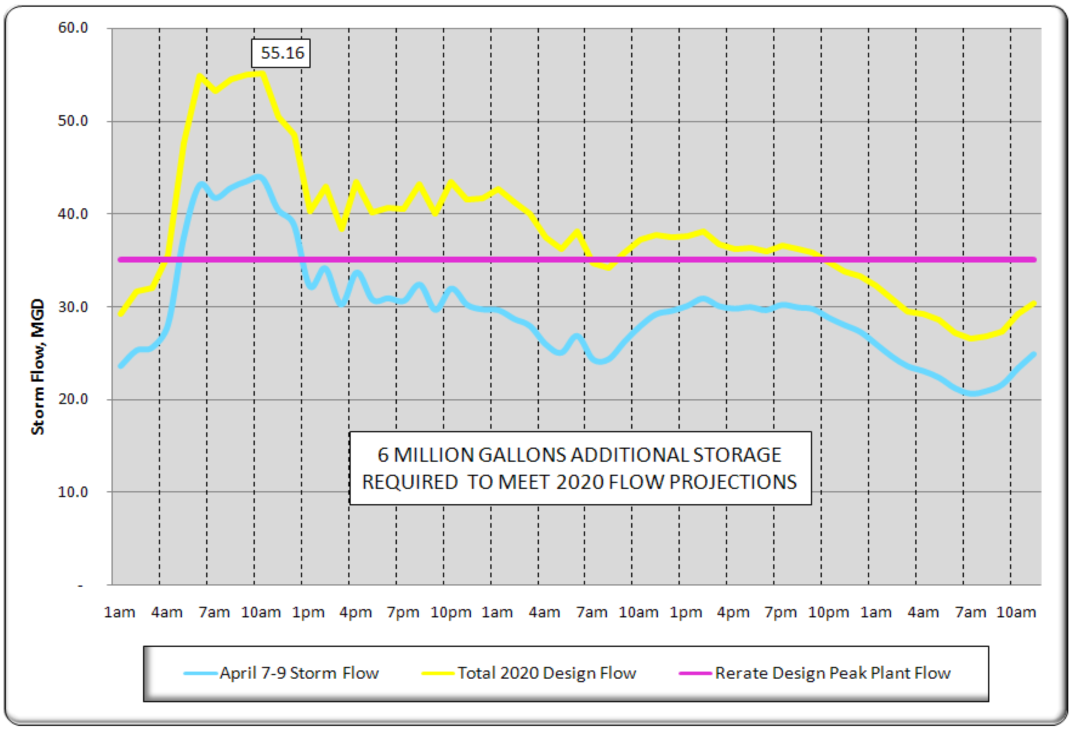
The 25-year, 24-hour storm event determined in Section 7.2.6 can be used with projected future growth to determine equalization sizing. A phased approach with an intermediate design year of 2020 was evaluated to extend the useful life of the Brandon Road Pump Station. Future EQ sizing reflects continuing to utilize the existing EQ basin.

- 2020 and 2030 events were plotted based on the base flow plus the 25-year, 24-hour storm plus the expected flows due to growth for 2020 and 2030, respectively.

- Equalization was determined as the difference between the WRF "Re-Rate" peak capacity of 35 MGD and the curve generated as described in above items and shown on the following page.

As noted, the 25-year, 24-hour storm will be equalized to the formal WRF "Re-Rate" peak capacity of 35 MGD. As shown in Figure 36, the additional equalization required for the 25-year 24-hour event for years 2010 through 2020 is approximately 6 million gallons.

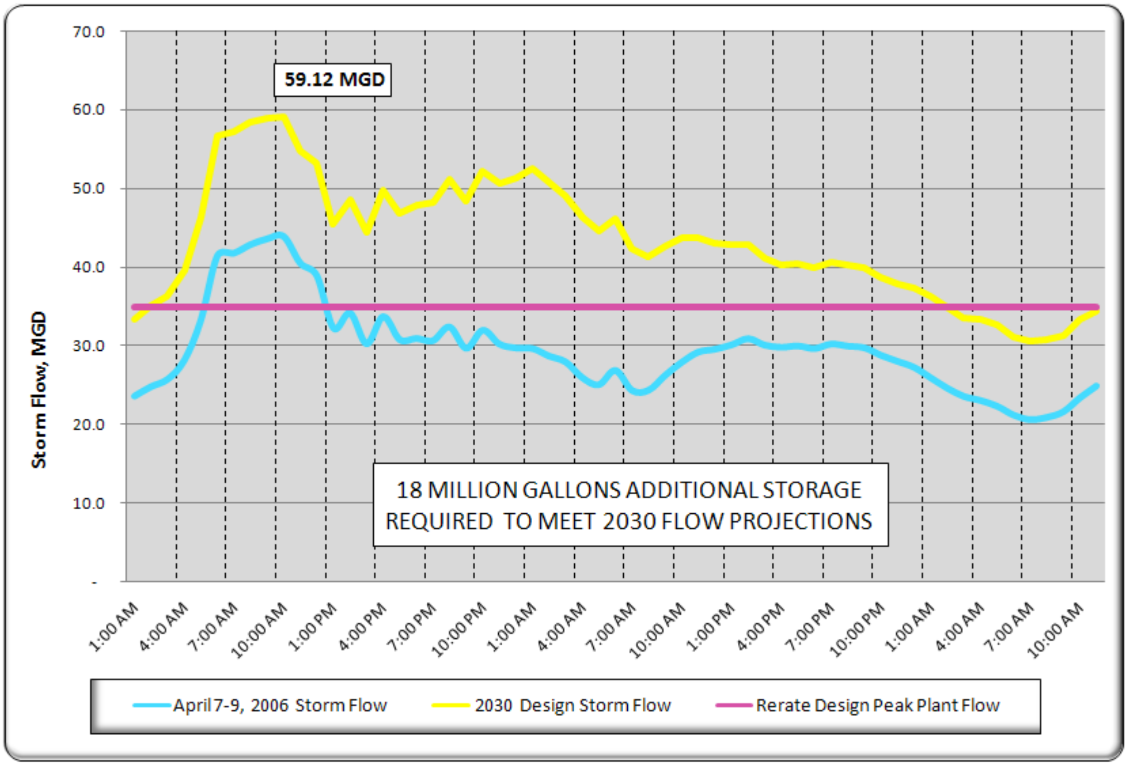
Figure 36 2010 through 2020 Storm Equalization Projections



The existing 12 million gallon equalization basin would continue to be used in conjunction with the new 6 million gallon basin. The 6 million gallons shown in the previous figure would be constructed as soon as possible and would be able to handle peak flows to approximately the year 2020. By that point, additional flow equalization would be necessary.

As shown in Figure 37, the equalization required in addition to the existing 12 million gallon EQ basin for the 25-year 24-hour event to reach 2030 is approximately 18 million gallons.

Figure 37 2010 through 2030 Storm Equalization Projections



The 18 million gallons storage needed to meet 2030 flows is in addition to the existing 12 million gallon EQ basin in service. The project could be phased as shown in Figure 34 with 6 million gallons constructed now for the design year 2020. An additional 12 million gallons would be necessary by the year 2020 to reach 2030 design flows.

Basis of Design

It is recommended the equalization alternative utilizes a phased approach for the design years 2020 and 2030. The following shows the basin sizing for each design year:

Design Year	EQ Capacity Needed
2010-2020	6 million gallons
2020-2030	12 million gallons
TOTAL	18 million gallons

Location

There are two main options for location of a new equalization basin: at the WRF or at the existing equalization basin site. The proposed equalization storage basin could potentially be constructed at the WRF. EQ storage at the plant would be screened since the new screening improvements have capacity for future wet weather flows. After screening, the wastewater would transfer by gravity to the equalization basin. Flow would be re-introduced to the WRF via the process pump station. Process pump station piping and capacity needs would be determined during preliminary design of the system. The City owns the land surrounding the WRF; there are many potential sites for an EQ basin.

Construction of additional equalization storage at the existing equalization basin site on East Chambers Street is another location alternative. This alternative is attractive for the first phase since it would extend the life of the Brandon Road Pump Station. By constructing equalization upstream of Brandon Road, the lift station life can be extended closer to 2020. The space available at the site may not accommodate the full 18 million gallons of storage due to other uses of the area.

Based on the overwhelming benefit of prolonging the upgrades to the Brandon Road Pump Station, it is recommended that the phase 1 equalization storage of 6 million gallons be constructed at the existing EQ basin site. Phase 2 equalization should be located at the WRF, which would take advantage of the screening and grit removal processes before storage.

Estimated Costs

The recommended alternative for equalization storage consists of a two phased addition of equalization storage both at the existing EQ site and at the WRF. A summary of the equalization alternative and a cost estimate is shown in the following table. The costs shown include estimated capital costs, administration and engineering in 2008 dollars.

Table 21 Equalization Alternative Summary

	Design Year	EQ Capacity Needed	Location	Preliminary Estimated Costs
Phase 1	2010-2020	6 mil gal	Existing EQ Site	$8,700,000
Phase 2	2020-2030	12 mil gal	WRF	$19,180,000
	Total	18 mil gal		$27,880,000

By constructing equalization storage upstream of Brandon Road Pump Station, the life of the station can be extended to approximately 2015-2020, depending on city growth. At that time, upgrades to the pump station and the forcemain will be necessary. It is estimated the lift station upgrades would cost approximately $4,400,000. As previously mentioned, the Brandon Road Force Main lacks redundancy. Also, pipe velocities in the force main begin to exceed recommended standards at projected future flows. A second force main would decrease frictional loss and increase the capacity of the Brandon

Road Pump Station. It is estimated a second forcemain would cost approximately $8,060,000.

8.2.2 Alternative No. 2 - Expand Various Process Components at WRF

One alternative that must be explored is the possibility of adding additional process components to meet peak flow projections. As previously mentioned, the facility has adequate average day flow capacity but limited peak flow capacity. The following unit process capacities are exceeded at the projected peak hourly 2030 design flow of 54.40 mgd.

- Brandon Road Pump Station (35.0 mgd)
- Primary Clarifiers (30.5 mgd)
- First Stage Trickling Filter Distributor Arms (27.0 mgd)
- First Stage Intermediate Clarifiers (25.9 mgd)
- Second Stage Trickling Filter Distributor Arms (30.0 mgd)
- Second Stage Intermediate Clarifiers (25.9 mgd)
- Process Pump Station (31.3 mgd)
- Final Clarifiers (31.8 mgd)
- Effluent Filters (33.3 mgd)
- Chlorine Contact Tank (41.0 mgd)

To meet the peak hourly flow projection of 54.4 mgd in 2030, the following unit process expansion would have to occur.

Brandon Road Pump Station – Brandon Road Lift Station would have to be upgraded to meet a peak hourly flow of 54.4 mgd or higher by replacing the existing pumps.

Primary Clarifiers – 3 additional 90-foot diameter primary clarifiers would have to be added to reach the peak flow capacity.

First Stage Trickling Filter Distributor Arms – The existing trickling filter distributor arms would be replaced with units able to handle 54.4 mgd. This upgrade will be done regardless due to equipment condition and age issues and will not be added to the costs for this alternative.

First Stage Intermediate Clarifiers – 2 additional 105-foot diameter intermediate clarifiers would be added to reach the peak hourly flow capacity.

Second Stage Trickling Filter Distributor Arms - The existing trickling filter distributor arms would be replaced with units able to handle 54.4 mgd. This upgrade will be done regardless due to equipment condition and age issues and will not be added to the costs for this alternative.

Second Stage Intermediate Clarifiers - 2 additional 105-foot diameter intermediate clarifiers would be added to reach the peak flow capacity.

Process Pump Station – The process pump station would have to be upgraded to meet a peak hourly flow of 54.4 mgd or higher. Upgrades to process pump station are currently underway and will not be included in this estimate.

Final Clarifiers – 2 additional 100 foot diameter final clarifiers would be added to meet a flow capacity of 56.52 mgd. This is based on surface overflow rates proved adequate by performance data and utilizing effluent filters for final polishing.

Effluent Filters – The effluent filters would most likely need to be expanded to reduce the amount of flow bypassed around the filters under peak conditions.

Chlorine Contact Tank – The chlorine contact tank would have to be expanded due to hydraulic limitations of the structure causing effluent flume submergence.

Estimated Costs
A preliminary cost estimate of the WRF process expansion alternative is shown in the following table. The costs shown include estimated capital costs, administration and engineering in 2008 dollars.

Table 22 WRF Expansion Alternative

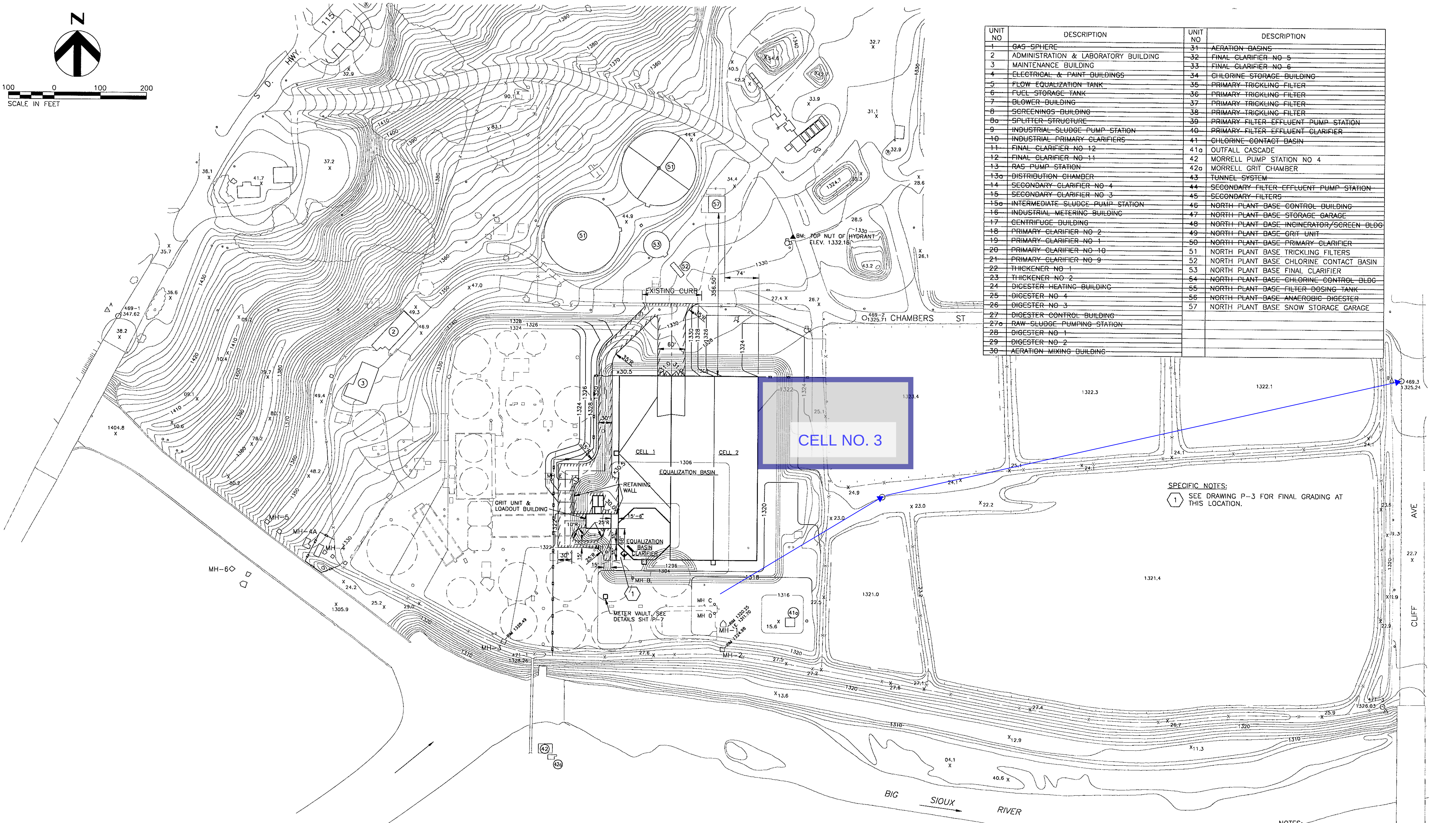
Process	Preliminary Estimated Costs
Brandon Road Pump Station	$5,900,000
Primary Clarifiers	$7,300,000
First Stage Intermediate Clarifiers	$5,800,000
Second Stage Intermediate Clarifiers	$5,800,000
Final Clarifiers	$5,600,000
Effluent Filters	$6,100,000
Chlorine Contact	$1,100,000
Piping/Splitter Structure Upgrades	$4,500,000
Total	$39,100,000

8.2.3 Recommendations

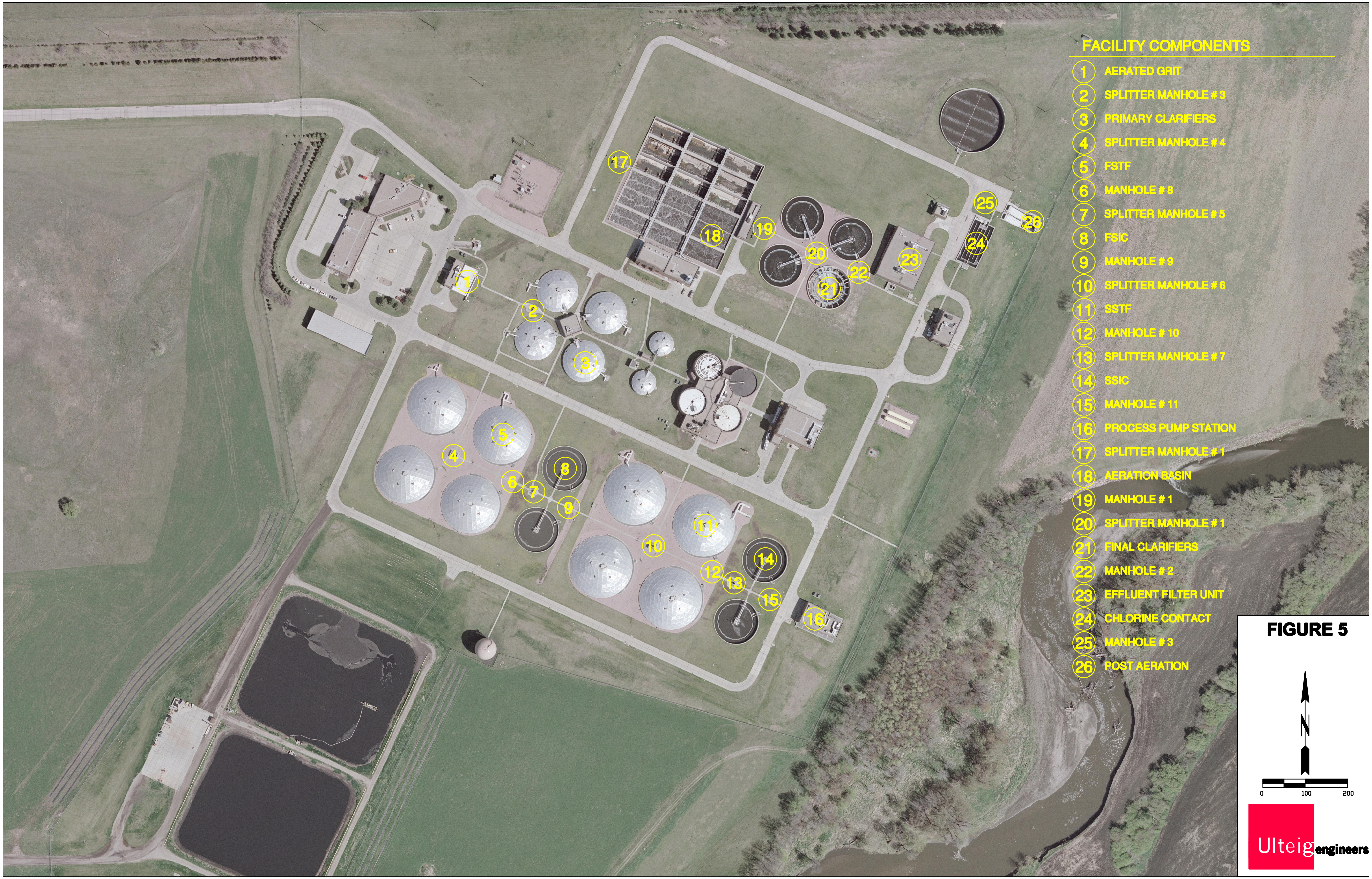
The recommended alternative for the Water Reclamation Facility to meet the City of Sioux Fall's wastewater treatment needs to the year 2030 is to construct equalization storage in two phases. The first phase would be located at the existing equalization basin site consisting of approximately 6 million gallons of storage. The second phase would be located at the WRF and would be 12 million gallons. Figure 38 on the following page shows the anticipated location of the 6 million gallon equalization basin storage for Phase 1 improvements. Figure 39 shows two potential locations for Phase 2 equalization at the WRF. There are other options for locations at the WRF that should be considered during preliminary design of Phase 2.

A summary of the recommendations due to capacity or condition issues detailed throughout this report include:

- Trickling Filter Distributor Arms – First and Second Stage Trickling Filter distributor arms have significant corrosion that limits the capacity of the processes. (See Section 6.5)
- Final Clarifier Mechanisms – The corrosive environment has caused significant damage to the final clarifier mechanisms. (See Section 6.10)
- Gravity Thickener Mechanism Rehabilitation – The Gravity Thickener Mechanisms have deteriorated over time and need immediate rehabilitation. Long term improvements include replacement. (See Section 6.16)
- Digester Mixing System – The existing mixing system is not operated and grit deposition in the digesters is an operational issue. A new mixing system to keep grit in suspension is recommended. (See Section 6.17)
- Gravity Filter Replacement – Replacement of the existing Wheeler bottom gravity filters with a more applicable filter technology. (See Section 6.11)
- Lime Feed System – The existing lime slaking and pumping system is in poor condition. Upgrades to the existing lime feed system are recommended. (See Section 6.7)
- Aeration System - The aeration system valves are in poor condition and need replacement. Switching to Fine Bubble Aeration appears to be cost effective in the long term. (See Section 6.8)
- Flow Measurement – At peak flows, the effluent flume is submerged, causing inaccurate flow measurement. (See Section 6.13)
- Brandon Road Pump Station and Forcemain – Capacity for future projected flows require upgrades in the long term. (See Sections 6.1-6.2)



UNIT NO	DESCRIPTION	UNIT NO	DESCRIPTION
1	GAS SPHERE	31	AERATION BASINS
2	ADMINISTRATION & LABORATORY BUILDING	32	FINAL CLARIFIER NO 5
3	MAINTENANCE BUILDING	33	FINAL CLARIFIER NO 6
4	ELECTRICAL & PAINT BUILDINGS	34	CHLORINE STORAGE BUILDING
5	FLOW EQUALIZATION TANK	35	PRIMARY TRICKLING FILTER
6	FUEL STORAGE TANK	36	PRIMARY TRICKLING FILTER
7	BLOWER BUILDING	37	PRIMARY TRICKLING FILTER
8	SCREENINGS BUILDING	38	PRIMARY TRICKLING FILTER
8a	SPLITTER STRUCTURE	39	PRIMARY FILTER EFFLUENT PUMP STATION
9	INDUSTRIAL SLUDGE PUMP STATION	40	PRIMARY FILTER EFFLUENT CLARIFIER
10	INDUSTRIAL PRIMARY CLARIFIERS	41	CHLORINE CONTACT BASIN
11	FINAL CLARIFIER NO 12	41a	OUTFALL CASCADE
12	FINAL CLARIFIER NO 11	42	MORRELL PUMP STATION NO 4
13	RAS PUMP STATION	42a	MORRELL GRIT CHAMBER
13a	DISTRIBUTION CHAMBER	43	TUNNEL SYSTEM
14	SECONDARY CLARIFIER NO 4	44	SECONDARY FILTER EFFLUENT PUMP STATION
15	SECONDARY CLARIFIER NO 3	45	SECONDARY FILTERS
15a	INTERMEDIATE SLUDGE PUMP STATION	46	NORTH PLANT BASE CONTROL BUILDING
16	INDUSTRIAL METERING BUILDING	47	NORTH PLANT BASE STORAGE GARAGE
17	CENTRIFUGE BUILDING	48	NORTH PLANT BASE INCINERATOR/SCREEN BLDG
18	PRIMARY CLARIFIER NO 2	49	NORTH PLANT BASE GRIT UNIT
19	PRIMARY CLARIFIER NO 1	50	NORTH PLANT BASE PRIMARY CLARIFIER
20	PRIMARY CLARIFIER NO 10	51	NORTH PLANT BASE TRICKLING FILTERS
21	PRIMARY CLARIFIER NO 9	52	NORTH PLANT BASE CHLORINE CONTACT BASIN
22	THICKENER NO 1	53	NORTH PLANT BASE FINAL CLARIFIER
23	THICKENER NO 2	54	NORTH PLANT BASE CHLORINE CONTROL BLDG
24	DIGESTER HEATING BUILDING	55	NORTH PLANT BASE FILTER DOSING TANK
25	DIGESTER NO 4	56	NORTH PLANT BASE ANAEROBIC DIGESTER
26	DIGESTER NO 3	57	NORTH PLANT BASE SNOW STORAGE GARAGE
27	DIGESTER CONTROL BUILDING		
27a	RAW SLUDGE PUMPING STATION		
28	DIGESTER NO 1		
29	DIGESTER NO 2		
30	AERATION MIXING BUILDING		


FACILITY COMPONENTS
1 AERATED GRIT
2 SPLITTER MANHOLE # 3
3 PRIMARY CLARIFIERS
4 SPLITTER MANHOLE # 4
5 FSTF
6 MANHOLE # 8
7 SPLITTER MANHOLE # 5
8 FSIC
9 MANHOLE # 9
10 SPLITTER MANHOLE # 6
11 SSTF
12 MANHOLE # 10
13 SPLITTER MANHOLE # 7
14 SSIC
15 MANHOLE # 11
16 PROCESS PUMP STATION
17 SPLITTER MANHOLE # 1
18 AERATION BASIN
19 MANHOLE # 1
20 SPLITTER MANHOLE # 1
21 FINAL CLARIFIERS
22 MANHOLE # 2
23 EFFLUENT FILTER UNIT
24 CHLORINE CONTACT
25 MANHOLE # 3
26 POST AERATION
FIGURE 5
N
0 100 200
Ulteig engineers

9.0 CAPITAL IMPROVEMENTS PLAN

To assess the various process equipment at the Sioux Falls WRF and make recommendations for capital improvements, an equipment evaluation matrix was set up based on our field inspections and process analysis.

All major process equipment, identified by City Personnel in the request for proposals, has been evaluated based on condition, performance, existing capacity, opportunity for optimization and redundancy. In the matrix shown in Table 23, the blue areas identify inadequate conditions that, in our opinion, need attention at some point in the future at the WRF. Definitions for condition, performance, capacity, optimization and redundancy follow Table 23.

Table 23 Equipment Assessment Matrix

Process Equipment	**Condition**	**Performance**	**Capacity**	**Optimization**	**Redundancy**
Primary Clarifiers					
Intermediate Clarifiers					
Final Clarifiers					
Gravity Thickeners					
Trickling Filter Distributors (FSTF)					
Trickling Filter Distributors (SSTF)					
Effluent Filters					
Digester Mixing System					
Grit Aeration System					
Activated Sludge System					

Additional Process Equipment	**Condition**	**Performance**	**Capacity**	**Optimization**	**Redundancy**
Brandon Road Pumps					
Eddy Current Drives at BRPS					
Mechanical Screens at BRPS					
Brandon Road Force Main					
Lime Feed System					
RAS Pumps					
Sludge Pumping					
Flow Measurement					
Sampling					
Disinfection					
Emerging Contaminants					

Inadequate Condition	
Adequate Condition	
Unknown or N.A.	

Definitions for the equipment evaluation matrix are as follows:

Condition – Physical condition of the equipment based on visual inspection, mechanical inspection and discussions with City Personnel.

Performance – Design performance of individual process equipment based on accepted industry standards.

Capacity – Capacity of process equipment based on existing 2007 conditions. Not future projections.

Optimization – The opportunity to optimize process equipment for more efficient operation and/or reduced operating costs.

Redundancy – The ability to take various process equipment out of operation and maintain normal operations at the WRF.

The goal of the equipment matrix is to provide City Personnel with an easy visual tool to help rank the areas of the WRF that are the most critical in terms of equipment replacement and short term capital improvement planning. Listed on the next page are the budgeted costs for various process components listed in the equipment evaluation matrix. These improvements do not include any items that address hydraulic or organic capacities for future growth.

9.1 Opinion of Probable Costs

The short and long range improvements based on condition and capacity limitations with associated cost estimates are shown in the table on the following page.

Table 24 Short Range and Long Range Improvements

		Short Range Improvements	
Design		**Improvements**	**Total**
Years	1	Trickling Filter Distributor Arms (4 FSTF @ 135')	$1,270,000
2009-2013	2	Trickling Filter Distributor Arms (4 SSTF @ 145')	$1,330,000
	3	Final Clarifier Mechanisms (4)	$2,060,000
	4	Gravity Thickener Mechanism Rehabilitation (2)	$100,000
	5	Digester Mixing System	$480,000
	6	Gravity Filter Replacement	$2,960,000
	7	Hydrated Lime System	$830,000
	8	Aeration System Valve Replacement	$140,000
	9	2010-2020 Flow Equalization Storage Basin	$8,700,000
		Subtotal	**$17,900,000**

		Long Range Improvements	
		Improvements	**Total**
Design	10	Fine Bubble Aeration for 3 Basins and Blowers	$1,610,000
Years	11	Flow Measurement	$50,000
2014-2030	12	Gravity Thickener Mechanisms Replacement	$390,000
	13a	Emerging Contaminants - UV/Peroxide	$18,870,000
	13b	Emerging Contaminants - Ozone	$18,230,000
	14	Brandon Road Pump Station Improvements	$4,400,000
	15	Brandon Road Force Main	$8,060,000
	16	Digester Pumping System Modifications	$60,000
	17	2020-2030 Flow Equalization Storage Basin	$19,180,000
		Subtotal	**$52,620,000**

Appendix

Facility Capacity Re-Rate Report

Facility Capacity Re-Rate Report
Water Reclamation Facility
City of Sioux Falls, South Dakota

UEI Project No. 407.013

February 2008

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

Signature ______________ Thomas J. Welle
Typed or Printed Name

Date 2/19/08 License No. 7867

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4808 South Technopolis Drive
Sioux Falls, SD 57106

Sioux Falls, SD • Fargo, ND • Bismarck, ND • Detroit Lakes, MN • Minneapolis, MN

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1.0 INTRODUCTION

The City of Sioux Falls Water Reclamation Facility (WRF) was constructed in three phases from 1980 through 1986. The first phase involved the construction of the tertiary treatment processes to bring the City into compliance with new water quality standards set by the EPA for the Big Sioux River. The second phase included the construction of the solids handling facilities along with the administration and maintenance buildings. The third phase provided the primary and secondary treatment processes.

The City has been meeting effluent limits since the facility was fully operational in 1986; however, current wastewater flows exceed the original design capacity while organic and nitrogen loadings are below design capacity.

The City retained Ulteig Engineers to study the facility design and performance information and recommend a rated capacity. This rated capacity will be presented to the South Dakota Department of Environment and Natural Resources (SD DENR) for review. The City, Ulteig and the SD DENR will collaboratively determine a true and formal capacity of the WRF.

This report is the culmination of the WRF re-rate study. The major components discussed in this report include the following:

- ✓ Original facility design information
- ✓ Existing conditions and facility performance data
- ✓ Facility design and performance analysis and comparison with standard criteria
- ✓ Hydraulic model development for the WRF
- ✓ Hydraulic model field testing and calibration
- ✓ Hydraulic deficiencies based on modeling
- ✓ Review of organic modeling equations for trickling filters
- ✓ Discussion of organic model development
- ✓ Determine process deficiencies based on organic model
- ✓ Determine capacities of unit processes based on regulatory standards
- ✓ Determine recommended capacity rating of WRF based on hydraulic model, organic model and regulatory standards.

2.0 FACILITY INFORMATION

The City of Sioux Falls has a flow equalization basin that dampens diurnal flow to the treatment facility. The equalization basin is located at the old wastewater treatment facility site and has approximately 12 million gallons of storage. The basin is upstream of the Brandon Road Pump Station.

The Brandon Road Pump Station conveys the majority of the domestic and industrial wastewater from the City of Sioux Falls to the Water Reclamation Facility (WRF). At the WRF, rotary fine screens installed in 2007 pre-treat the wastewater to remove large materials that could damage or plug equipment. The screenings removed are washed and pressed before disposal. After screening, the wastewater enters an aerated grit chamber where sand and gravel are removed to minimize wear on equipment. Grit removed from the wastewater is washed and dewatered before disposal in the landfill.

The facility has four Primary Clarifiers to remove settable solids and scum from the wastewater. The Primary Clarifiers are 90 feet in diameter and have 8-feet of sidewater depth. The Clarifiers are center-feed with peripheral weirs. Settled solids (sludge) and scum are collected by a rotating arm and are pumped to the solids handling units.

Secondary treatment is accomplished by two stages of Trickling Filters. The four First Stage Trickling Filters are 135 feet in diameter and are 7 feet deep. The four Second Stage Trickling Filters are 145 feet in diameter and are 7 feet deep. The Trickling Filters contain Sioux Quartzite Rock media, distributor arms and an underdrain system. The microorganisms on the media of the Trickling Filters remove pollutants in the waste stream. Each stage of Trickling Filters is followed by two 105 foot diameter Intermediate Clarifiers with side water depths of 10 feet. The Intermediate Clarifiers remove biomass that sloughs off of the Trickling Filter media by gravity settling.

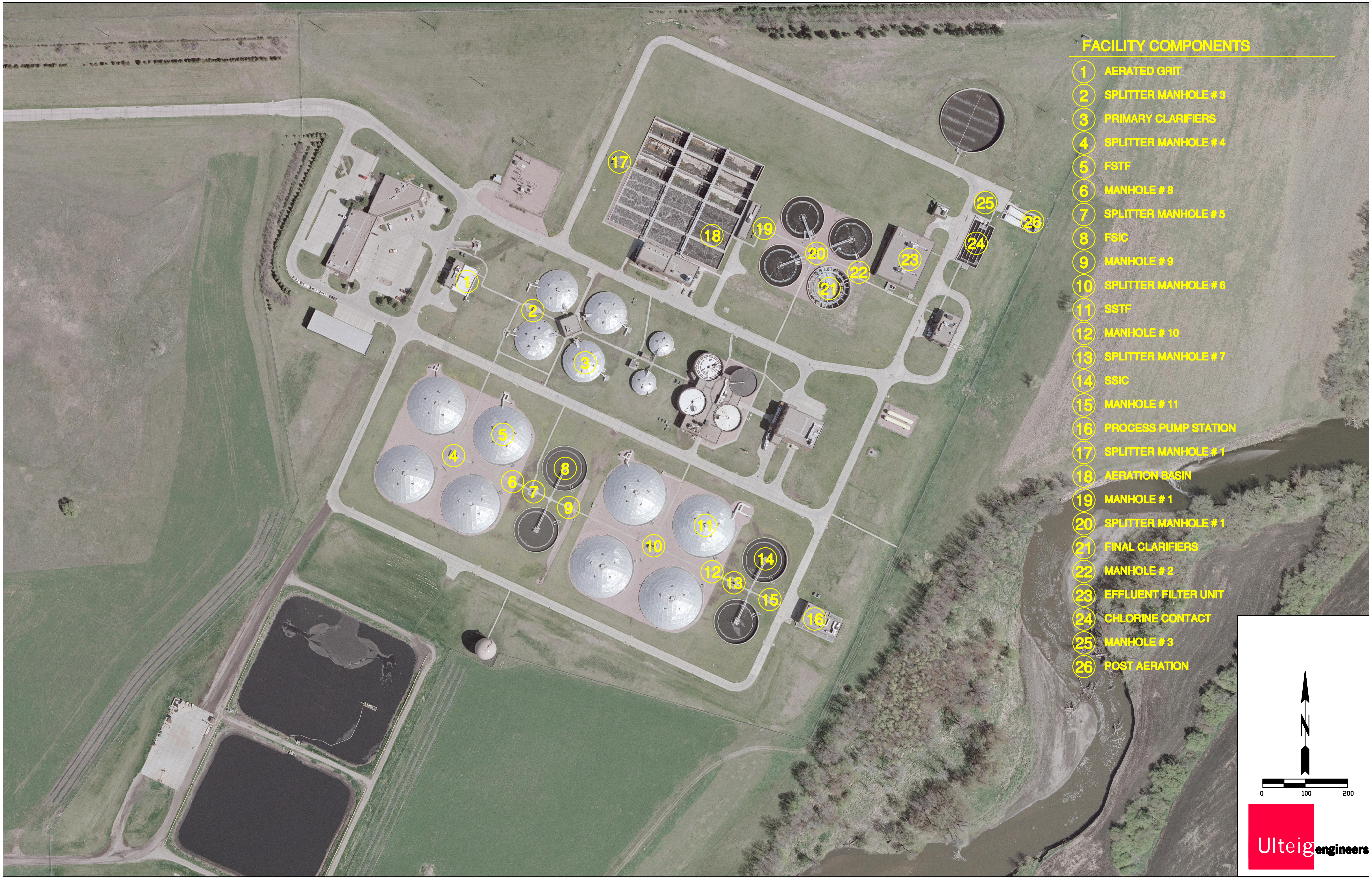
Tertiary treatment for ammonia removal is accomplished in the Activated Sludge System. Microorganisms or “activated sludge” in the basin is mixed with the wastewater from the secondary treatment process. Coarse bubble diffusers supply air to provide oxygen to the microorganisms and mixing. In the aerobic environment, the microorganisms nitrify ammonia to allow the facility to meet permit requirements. The effluent from the Aeration Basins flows into four 100-foot diameter Final Clarifiers with side water depths of 14 feet. The Final Clarifiers settle solids from the treated wastewater. The sludge from the Final Clarifiers is returned to the activated sludge process or wasted.

Effluent from the Final Clarifier flows to the Effluent Filter Unit for final polishing. Eight dual-media gravity filters 34 feet by 17 feet by 8 feet deep further remove any pollutants remaining in the water to ensure compliance with permit requirements. Filtered water flows to the Chlorine Contact Basin where chlorine is added for disinfection. Residual chlorine in the wastewater is removed by sulfur dioxide. A Cascade Aeration Unit increases the dissolved oxygen in the water before final discharge to the Big Sioux River. Figure 1 on the following page shows the facility components on an aerial photo.

The design influent conditions of the facility are shown in Table 1.

Table 1 Facility Influent Design Conditions

Parameter	Value	
Average Daily Flow	13.43	mgd
Peak Instantaneous Flow	27.00	mgd
BOD Loading (avg)	48,443	lb/d
TSS Loading (avg)	34,059	lb/d
TKN Loading (avg)	5,419	lb/d


FACILITY COMPONENTS
1 AERATED GRIT
2 SPLITTER MANHOLE # 3
3 PRIMARY CLARIFIERS
4 SPLITTER MANHOLE # 4
5 FSTF
6 MANHOLE # 8
7 SPLITTER MANHOLE # 5
8 FSIC
9 MANHOLE # 9
10 SPLITTER MANHOLE # 6
11 SSTF
12 MANHOLE # 10
13 SPLITTER MANHOLE # 7
14 SSIC
15 MANHOLE # 11
16 PROCESS PUMP STATION
17 SPLITTER MANHOLE # 1
18 AERATION BASIN
19 MANHOLE # 1
20 SPLITTER MANHOLE # 1
21 FINAL CLARIFIERS
22 MANHOLE # 2
23 EFFLUENT FILTER UNIT
24 CHLORINE CONTACT
25 MANHOLE # 3
26 POST AERATION
N
0 100 200
Ulteig engineers

A number of flow conditions are considered in the design and evaluation of wastewater treatment facilities. The following flows are used in this study of the Sioux Falls WRF.

- *Average Daily Flow* – The average daily flow is the average of the daily volumes received for a continuous 12 month period expressed as a volume per unit time (million gallons per day or mgd).

- *Maximum Day Flow* – The maximum day flow is the largest volume of flow received during a continuous 24 hour period expressed as a volume per unit time (mgd).

- *Maximum Month Flow* – The maximum month flow is the largest volume of flow received during a month long period expressed as a volume per unit time (mgd).

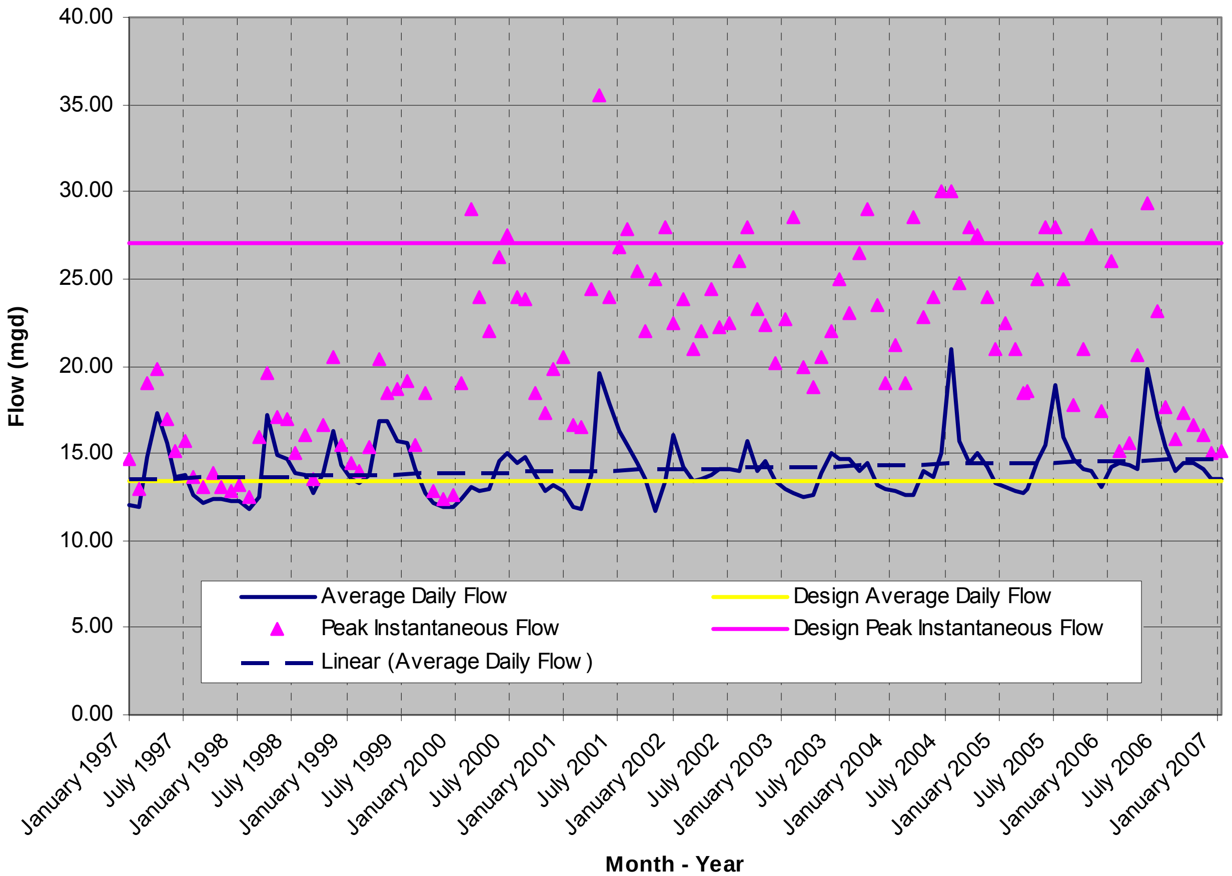
- *Peak Hourly Flow* – The peak hourly flow is the largest volume of flow to be received during a one hour period expressed as a volume per unit time (mgd).

- *Peak Instantaneous Flow* – The peak instantaneous flow is the instantaneous maximum flow rate received as a volume per unit time (mgd).

3.0 EXISTING CONDITIONS

The daily flows for the WRF were complied from 1997 through 2006. Influent flow from 1997 to 2006 is shown in Figure 2.

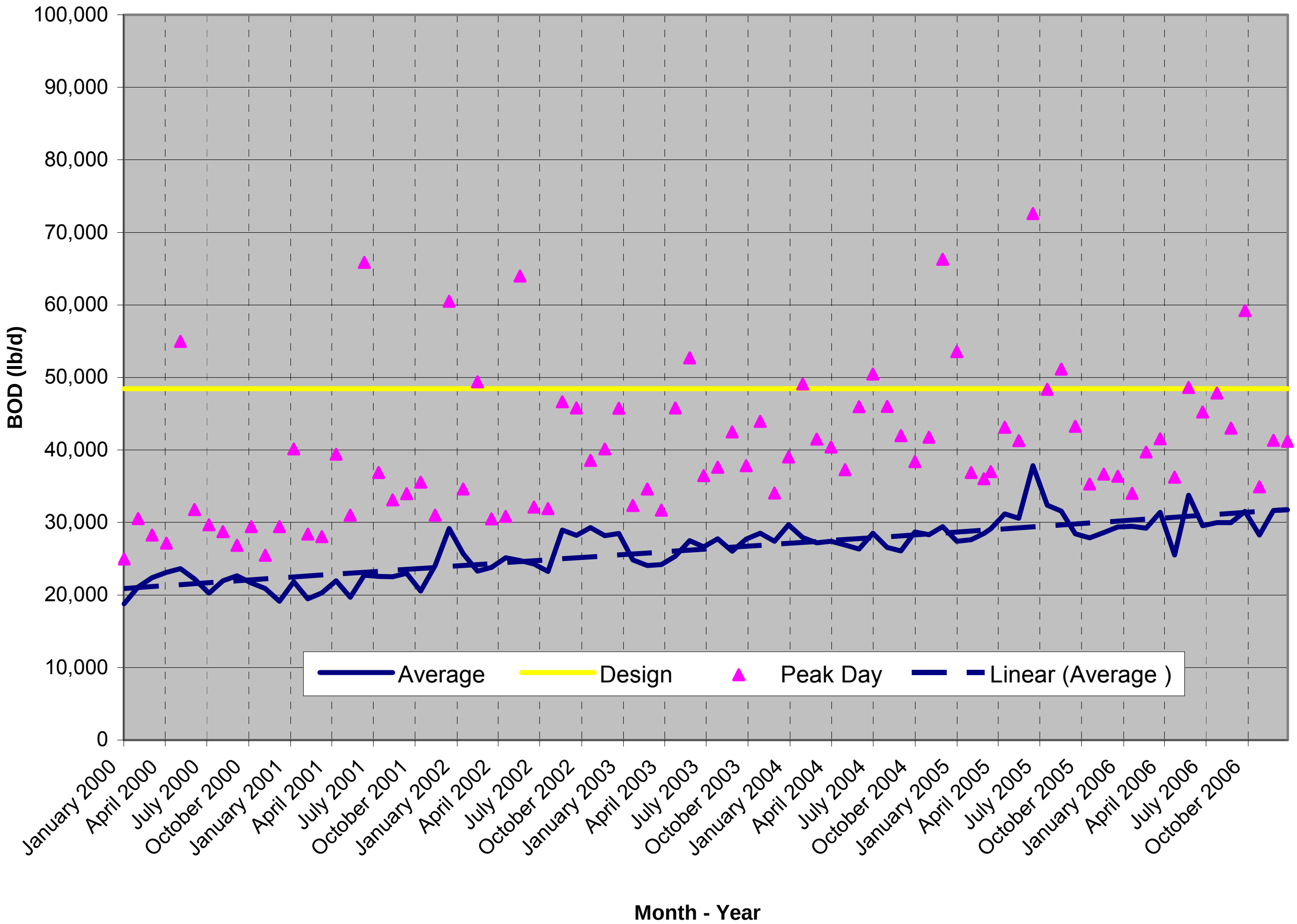
Figure 2 Influent Flow



The Average Daily Flow at the WRF has been consistently higher than the Design Average Daily Flow of 13.43 mgd. The flow trend has been increasing slightly over the past ten years, at approximately 157,000 gallons per day per year. Peak flows have been higher than the design Peak Hourly Flow of 27.0 mgd over the past 10 years. The peak instantaneous flow recorded for the studied time frame was 58.50 mgd on June 16, 2004. This value and the June 17, 2004 peak flow of 46.20 mgd were removed from Figure 2 for clarity but are noted as the peak instantaneous flow values of the facility.

The influent BOD loadings to the WRF were complied for the years of 2000 through 2006. The following figure shows the influent loadings at the WRF for the past 6 years.

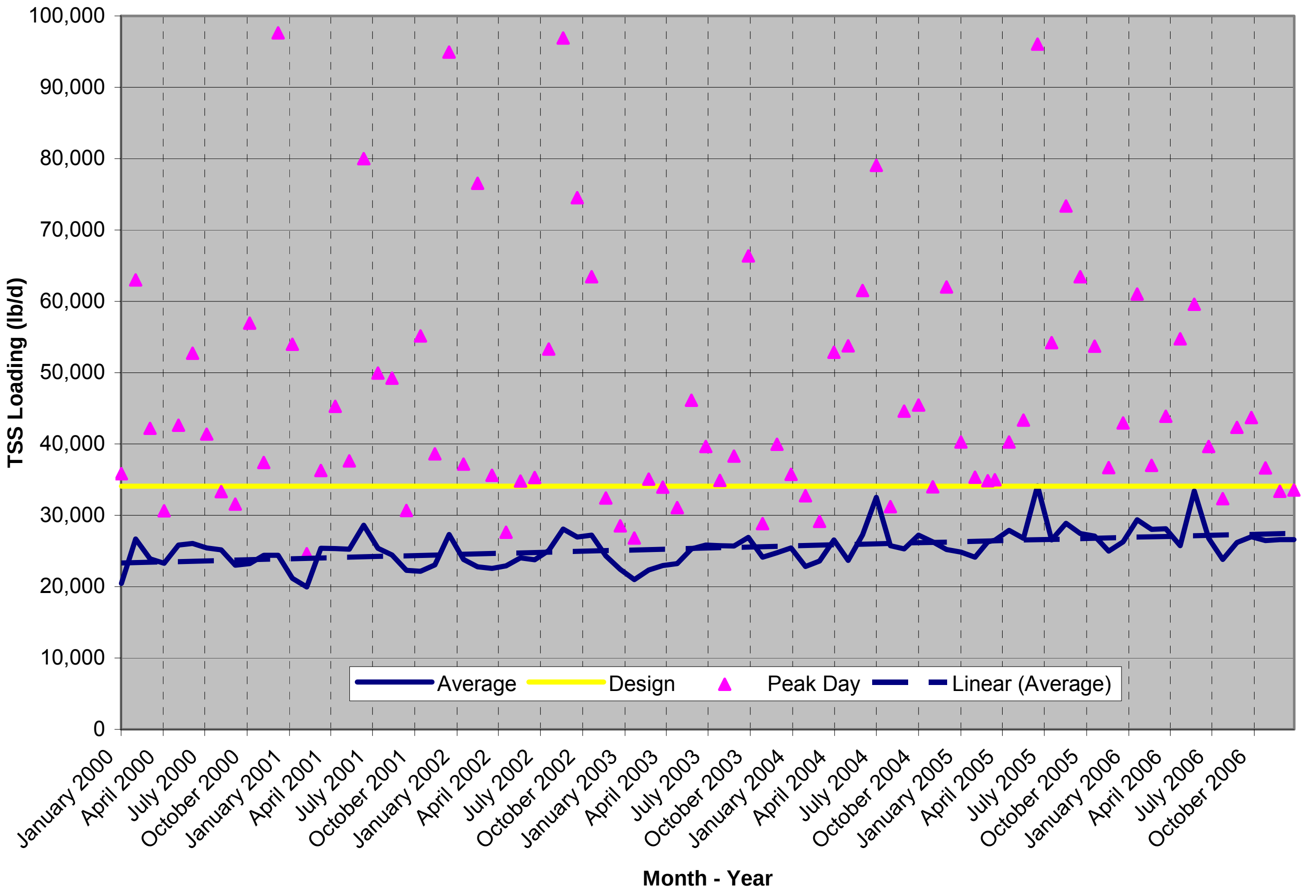
Figure 3 Influent BOD Loading



The average BOD loading to the WRF has been consistently below the Design Average Loading of 48,442 lb/d. The average 2006 BOD loading was 30,163 pounds per day. The loading trend has been steadily increasing at approximately 1,240 pounds per day per year based on the sampling data collected prior to 2007.

The influent TSS loadings to the WRF were complied for the years of 2000 through 2006. The following figure shows the influent loadings at the WRF for the past 6 years.

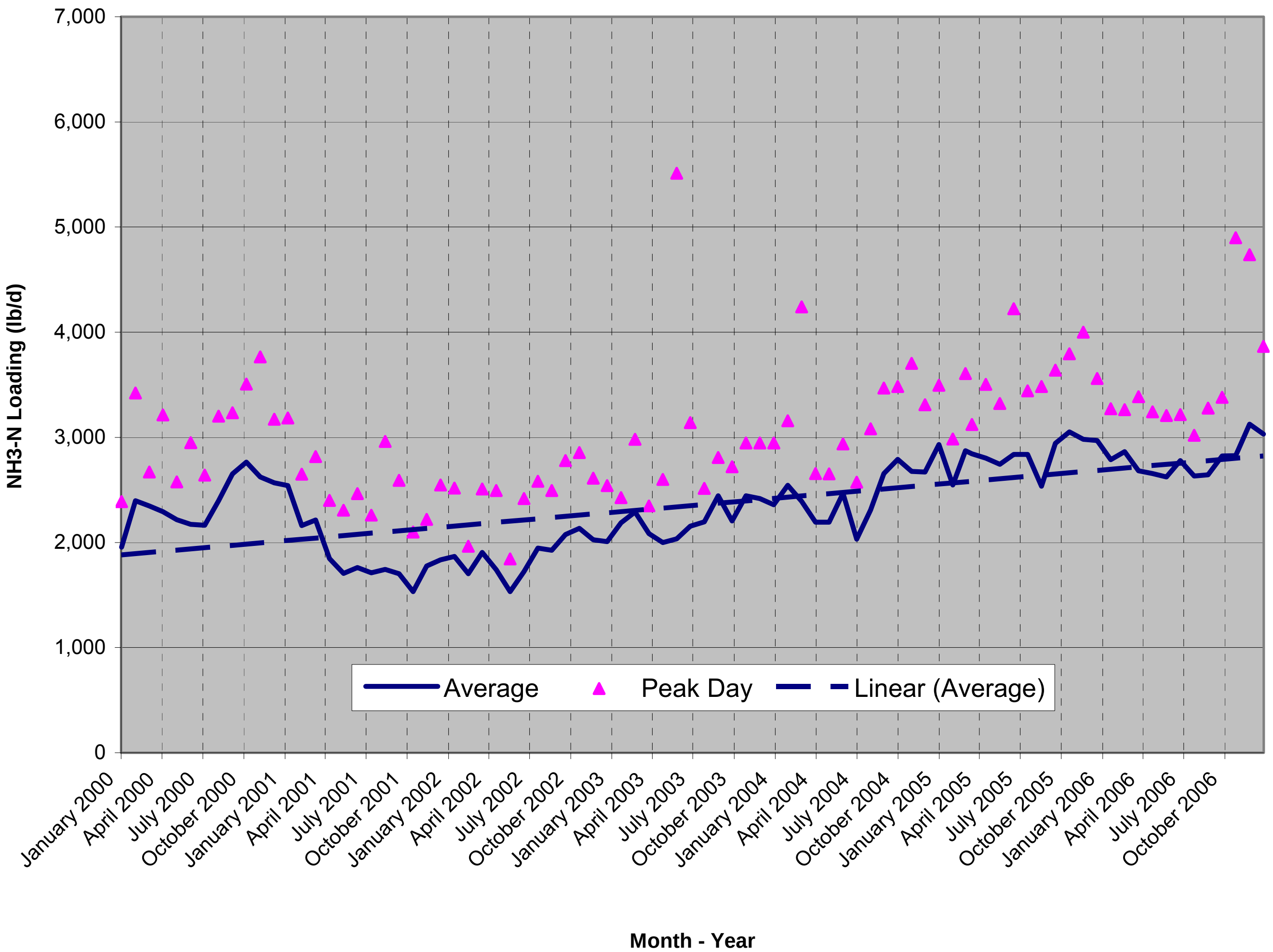
Figure 4 Influent TSS Loading



The Average Daily TSS loading to WRF has been consistently below the Design Average Daily Loading of 34,059 lb/d. The TSS loading trend has been increasing at approximately 431 pounds per day per year based on the sampling data collected prior to 2007.

The influent ammonia loadings to the WRF were complied for the years of 2000 through 2006. The following figure shows the influent loadings at the WRF for the past 6 years.

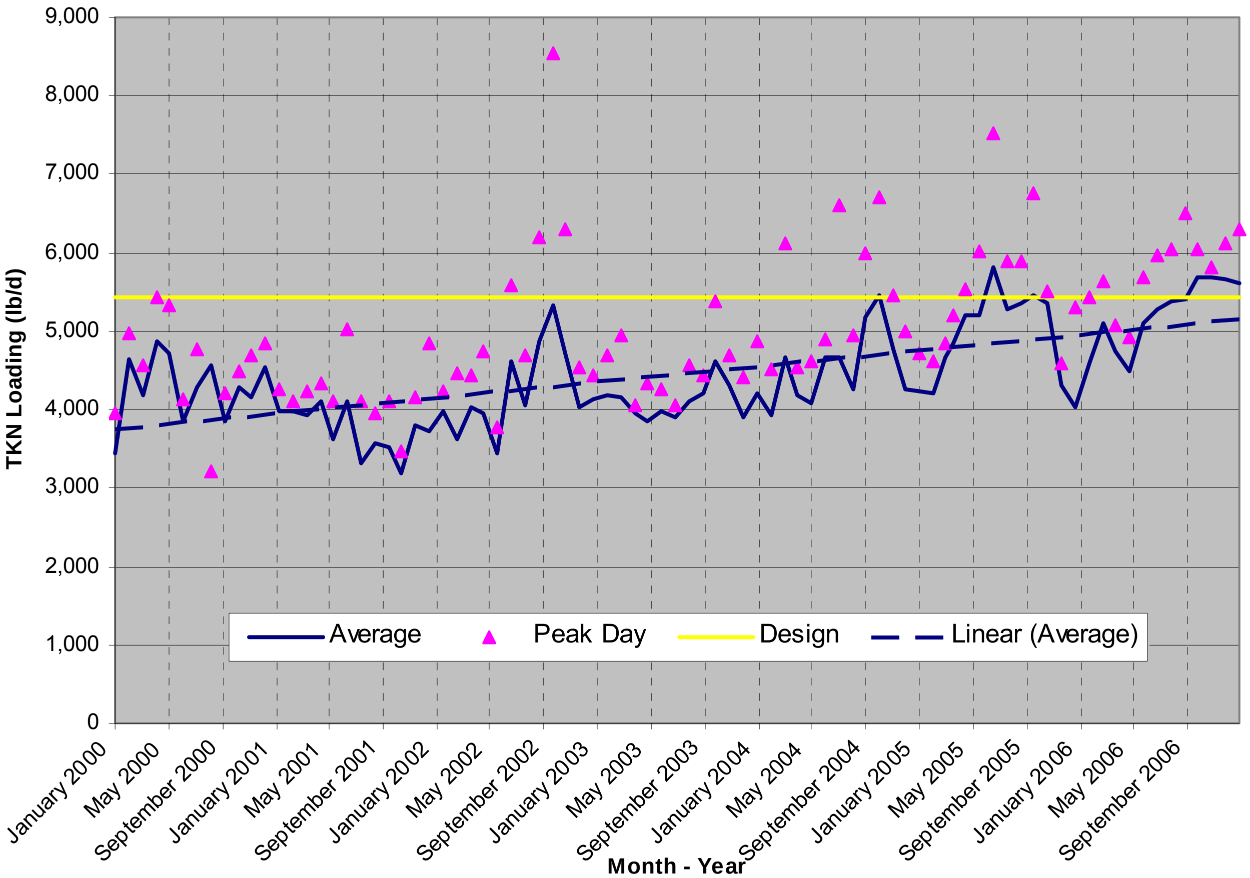
Figure 5 Influent NH_3-N Loading



The average daily ammonia loading to the facility has been steadily increasing at approximately 59 pounds per day per year over the past six years based on the sampling data collected prior to 2007.

The influent TKN loading to the WRF were compiled for the past six years. The following figure shows the influent TKN loading from 2000 through 2006.

Figure 6 Influent TKN Loading



The average daily TKN loading to the WRF has been under the average design loading of 5,419 most of the time. However, there have been some occurrences of the daily loading exceeding the design loading. The TKN loading to the facility has been steadily increasing at approximately 134 pounds per day per year over the past 6 years.

The flow, BOD loading, TSS loading, ammonia loading and TKN loading were compared to the design values for the years 2000 through 2006 and are shown in Table 2. Also, the new sampling data (discussed in Section 4.0) from September 2007 through December 2007 is shown.

Table 2 Influent and Design Conditions

Parameter	2000	2001	2002	2003	2004	2005	2006	2007 Sep-Dec
Flow								
Average (mgd)	13.56	14.65	13.97	13.63	14.57	14.45	14.92	15.37
Percent of Design	101%	109%	104%	102%	108%	108%	111%	114%
Max Month (mgd)	14.98	19.61	15.66	15.05	21.02	18.87	19.88	16.50
Max Month/Ave Day	1.10	1.34	1.12	1.10	1.44	1.31	1.33	
Peak Instantaneous (mgd)	29.00	35.50	28.00	29.00	58.50	28.00	29.30	
BOD								
Average (lb/d)	21,481	22,315	26,108	26,636	27,558	30,254	30,163	34,874
Percent of Design	44%	46%	54%	55%	57%	62%	62%	72%
Max Month (lb/d)	23,647	29,163	29,308	29,681	29,433	37,843	33,738	38,208
Max Day (lb/d)	54,959	65,869	63,996	52,711	66,282	72,592	59,215	66,112
TSS								
Average (lb/d)	24,318	24,188	24,499	24,442	25,913	27,239	27,335	31,711
Percent of Design	71%	71%	72%	72%	76%	80%	80%	93%
Max Month (lb/d)	26,678	28,608	28,074	26,908	32,503	34,144	33,390	33,532
Max Day (lb/d)	97,641	134,353	96,928	79,062	79,062	96,065	61,051	44,343
NH_3-N								
Average (lb/d)	2,379	1,877	1,882	2,234	2,488	2,831	2,790	3,054
Percent of Design	-	-	-	-	-	-	-	-
Max Month (lb/d)	2,764	2,542	2,135	2,446	2,932	3,051	3,125	3,165
Max Day (lb/d)	3,766	3,184	2,857	5,513	4,240	4,224	4,898	4,205
TKN								
Average (lb/d)	4,285	3,738	4,232	4,113	4,526	4,973	5,227	5,915
Percent of Design	79%	69%	78%	76%	84%	92%	96%	109%
Max Month (lb/d)	4,874	4,112	5,338	4,619	5,463	5,818	5,687	6,662
Max Day (lb/d)	5,421	5,015	8,548	5,383	6,717	7,518	6,512	8,085

The facility is below design conditions except for influent flow in 2006. The TKN loading was above the design loading, and the TSS loading was near the original design value based on the data collected from September through December 2007.

4.0 FACILITY PERFORMANCE

The WRF facility performance was analyzed to determine the unit process efficiencies and any "weak links" in the system.

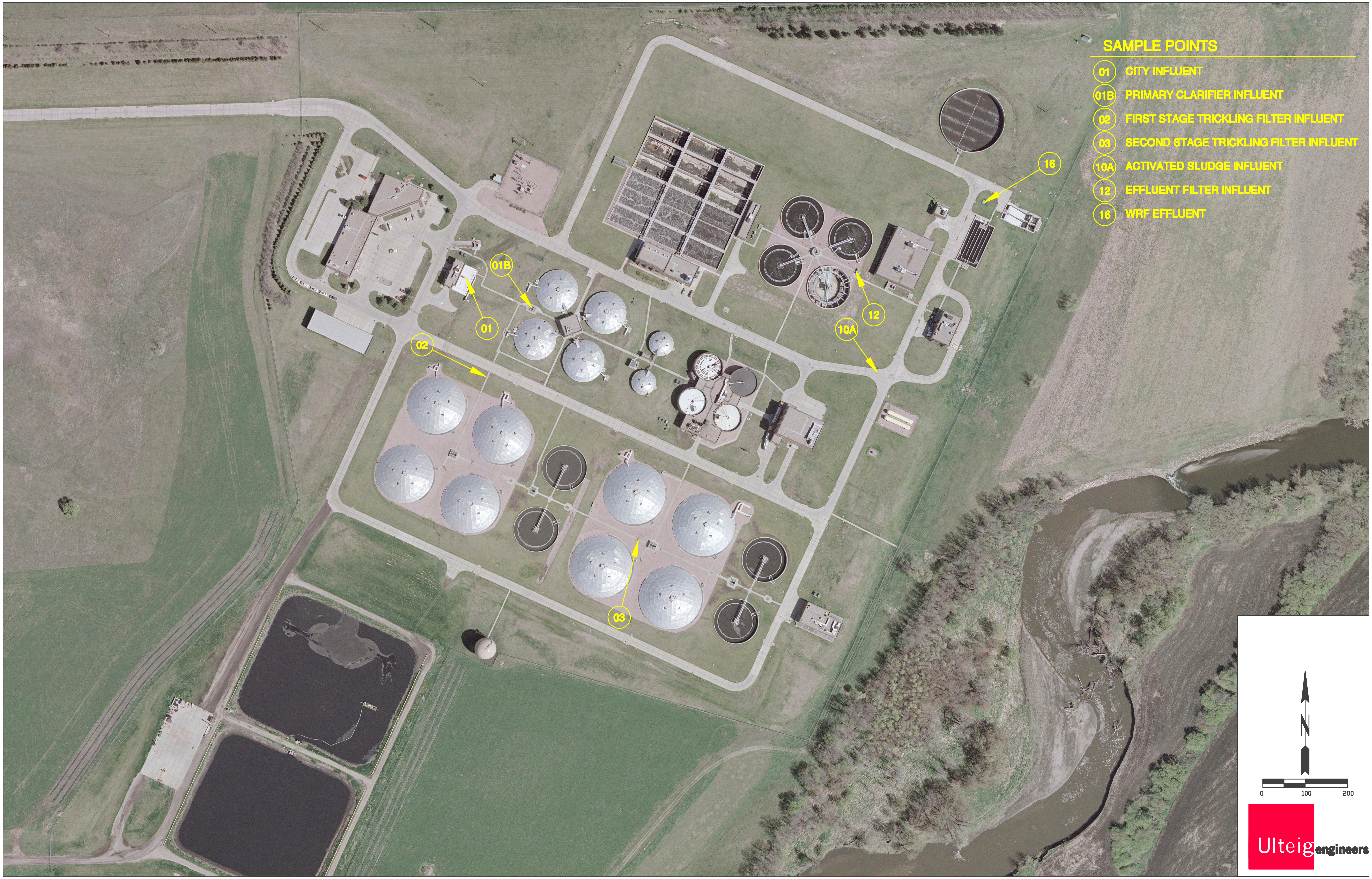
4.1 Facility Samples

The WRF routinely samples between processes to gauge performance of the individual treatment units. The following sample points throughout the facility were used in the process analysis:

01	City Influent
01B	Primary Clarifier Influent
02	First Stage Trickling Filter Influent
03	Second Stage Trickling Filter Influent
10A	Activated Sludge Influent
12	Effluent Filter Influent
16	WRF Effluent

Figure 7 on the following page shows the sample points throughout the facility.

During analysis of data for the Master Plan, it was discovered some sample points may not be representative of the actual conditions. The samplers for points 01, 02, 03, and 10A were located in the process pump station. The sample line lengths were long (especially points 01 and 02), and it was speculated that treatment occurred in the sample lines. Additional samplers were placed at the actual sample sites to compare with the existing, remotely located samplers. The sample sites and sample lines are shown in Figure 8 on Page 14.



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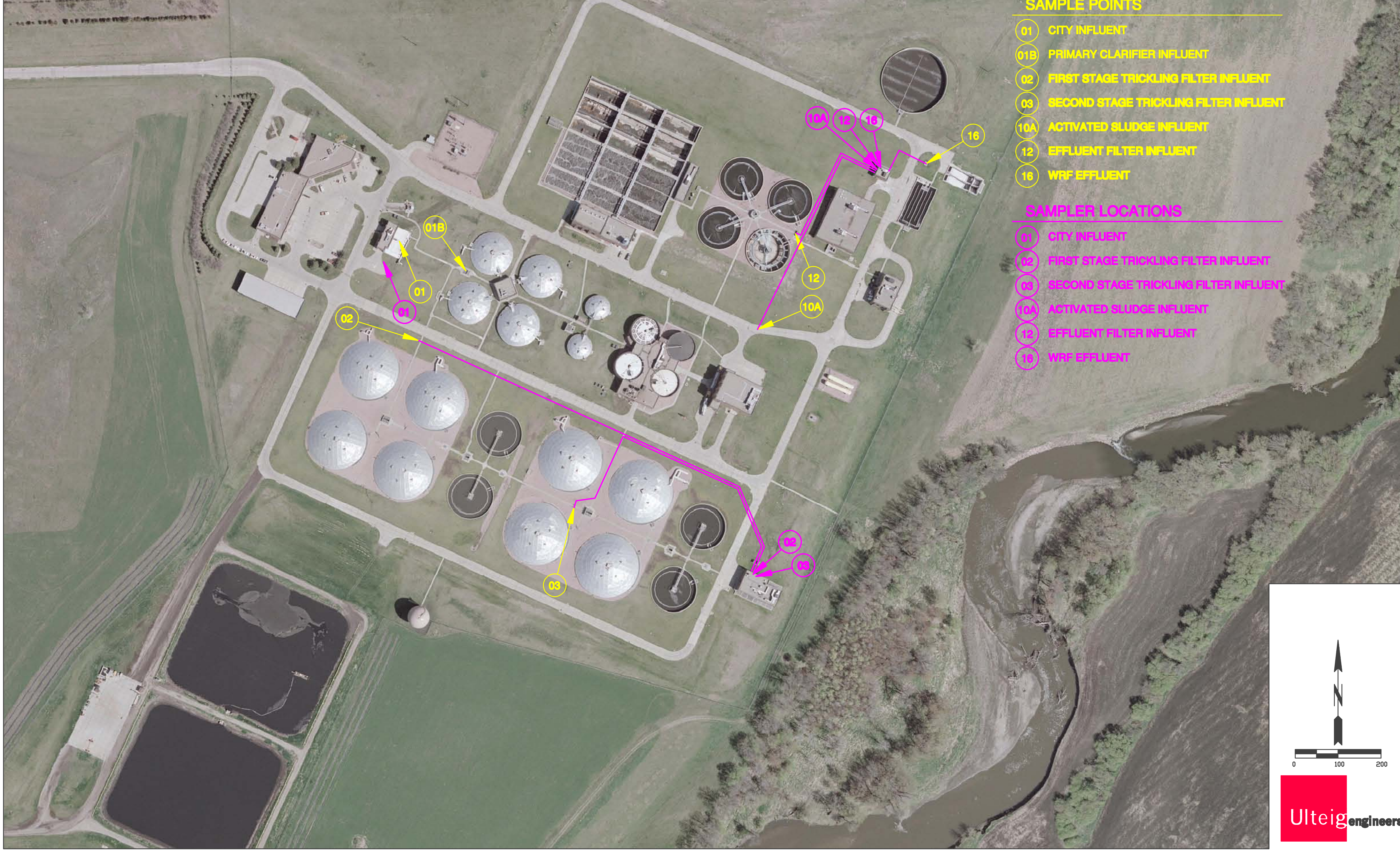

SAMPLE POINTS
01 CITY INFLUENT
01B PRIMARY CLARIFIER INFLUENT
02 FIRST STAGE TRICKLING FILTER INFLUENT
03 SECOND STAGE TRICKLING FILTER INFLUENT
10A ACTIVATED SLUDGE INFLUENT
12 EFFLUENT FILTER INFLUENT
16 WRF EFFLUENT
SAMPLER LOCATIONS
01 CITY INFLUENT
02 FIRST STAGE TRICKLING FILTER INFLUENT
03 SECOND STAGE TRICKLING FILTER INFLUENT
10A ACTIVATED SLUDGE INFLUENT
12 EFFLUENT FILTER INFLUENT
16 WRF EFFLUENT
10A
12
16
16
01B
01
01
02
12
10A
02
03
03
N
0
100
200
Ulteig engineers

The original remote sample points and new sample points were tested concurrently for comparison purposes. Figures 9, 10 and 11 show the sample discrepancies between the sample points. The 'N' designates the new sample point.

Figure 9 TBOD Concentrations for Points 01 and 01N

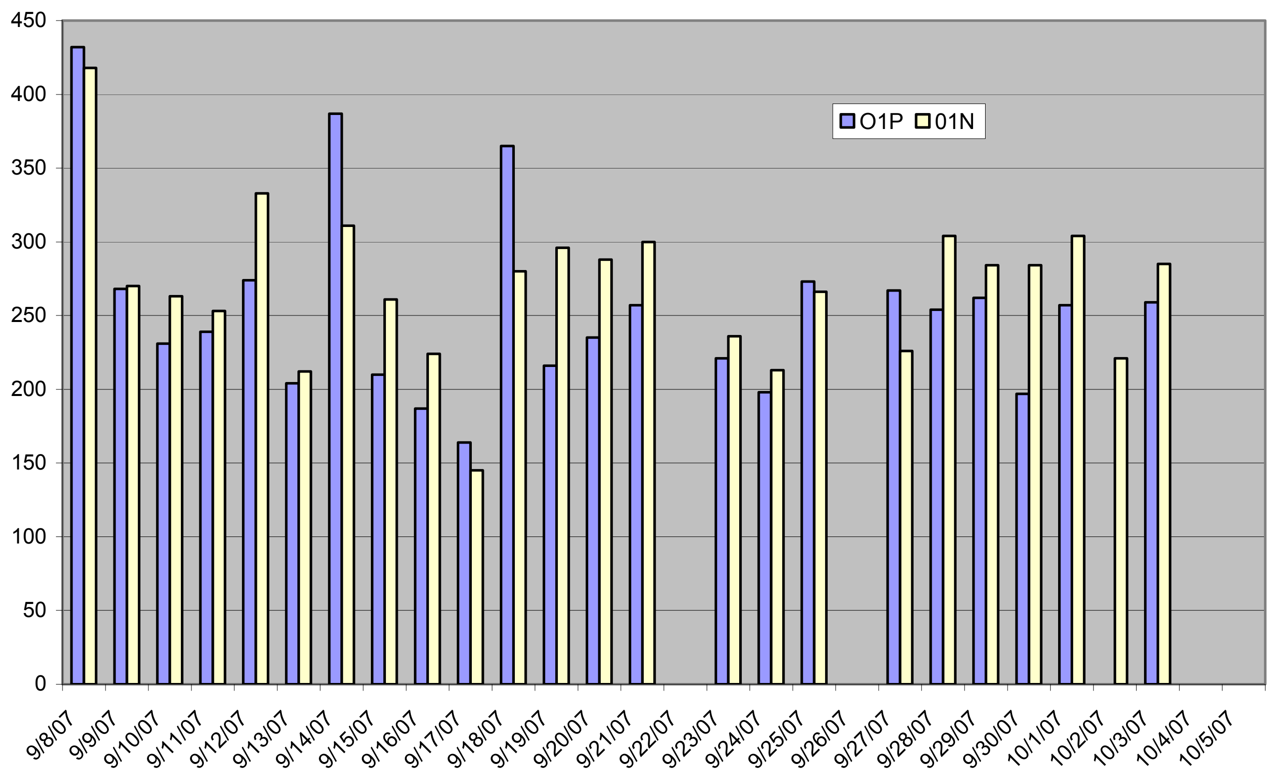


Figure 9 shows the samples remotely located are not consistent with the samples taken at the site for sample point 01 (Plant Influent). The existing sample point 01 is sometimes higher and sometimes lower than the new sample point 01N. The TSS, TKN and NH3 tests also did not correlate from old sample point to new sample point. Sample point 01N is considered more representative of the WRF influent for all parameters and will be used for all subsequent analyses.

Figure 10 TBOD Concentrations for Sample Points 02 and 02N

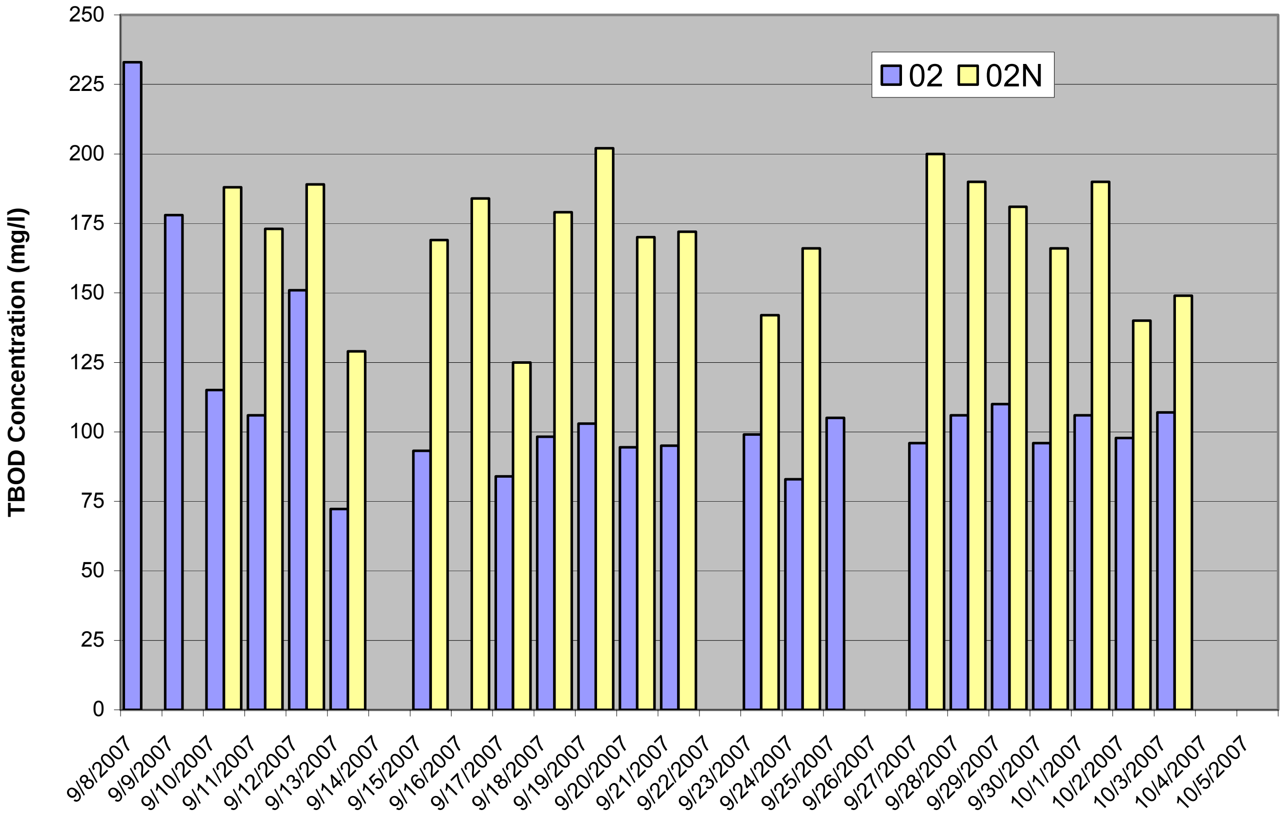


Figure 10 shows the samples remotely located are drastically different from the samples taken at the site for sample point 02 (First Stage Trickling Filter Influent). The old sample point remotely located, 02, is substantially lower than the new sample point 02N. It is speculated that treatment occurred in the long sample line. The TSS, TKN and NH3 tests also did not correlate from old sample point to new sample point. Sample point 02N is considered more representative of the FSTF influent for all parameters and will be used for all subsequent analyses.

Figure 11 TBOD Concentrations for Sample Points 03 and 03N.

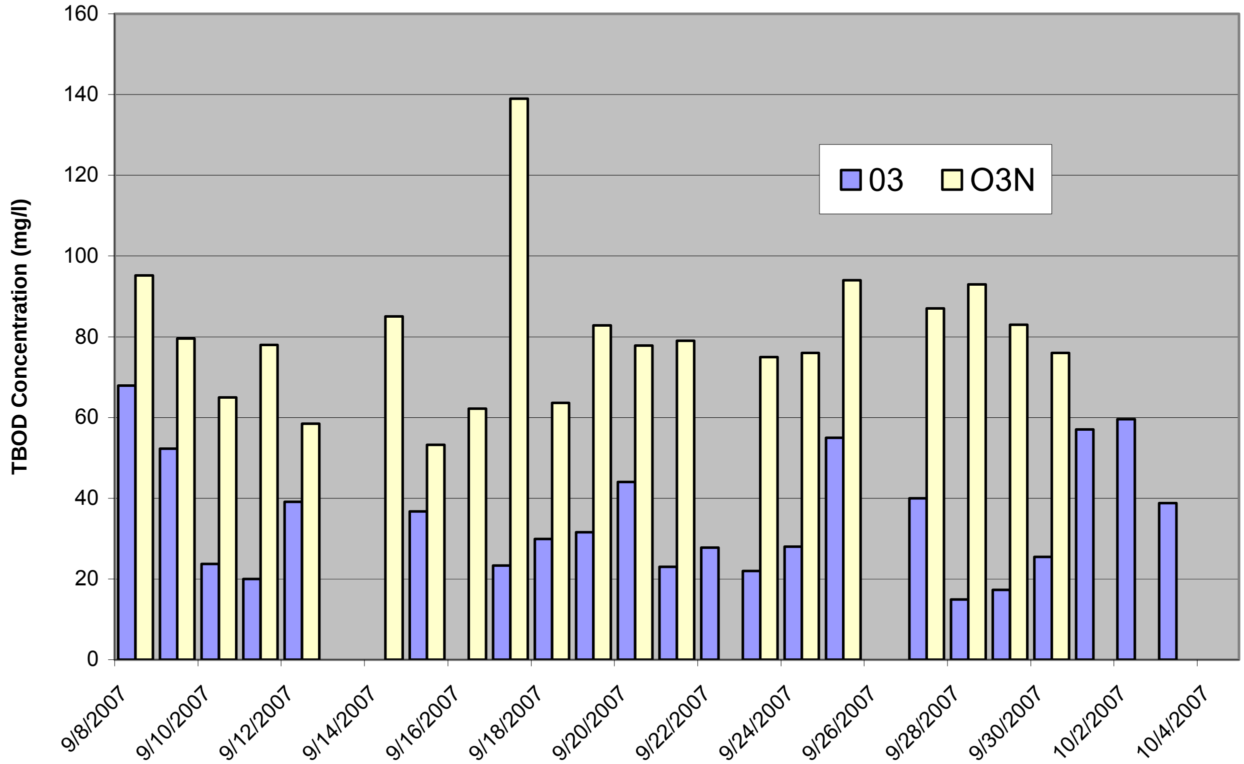


Figure 11 shows the samples remotely located are also drastically different from the samples taken at the site for sample point 03 (Second Stage Trickling Filter Influent). The old sample point remotely located, 03, is substantially lower than the new sample point 03N. It is again suspected that treatment occurs in the long sample line. The TSS, TKN and NH3 tests also did not correlate from old sample point to new sample point. Sample point 03N is considered more representative of the SSTF influent for all parameters and will be used for all subsequent analyses.

Figure 12 shows the old sample point 10A and new sample point 10AN located in the process pump station. This sample represents the activated sludge influent.

Figure 12 TBOD Concentrations for Sample Points 10A and 10AN

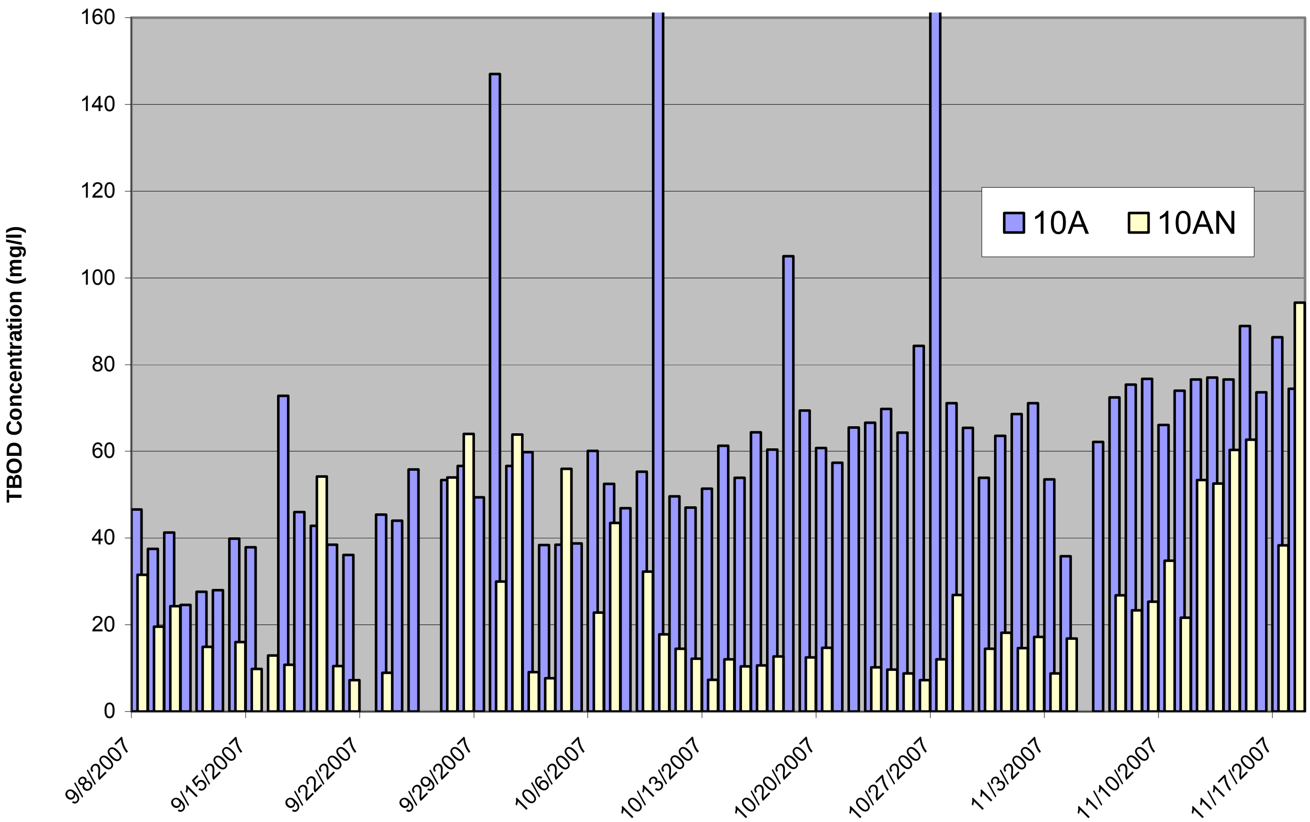


Figure 12 shows the samples remotely located are also drastically different from the samples taken at the site for sample point 10A (Activated Sludge Influent). The old sample point remotely located, 10A, is substantially higher than the new sample point 10AN. The TSS tests also did not correlate from old sample point to new sample point. However, the TKN and NH3 tests were more comparable. It was initially suspected that the new sample point in the process pump station was not representative of the actual BOD and TSS conditions, due to turbulent conditions in the sampler. Although the existing sample point was suspected as being unrepresentative, the existing sample point 10A was confirmed as more representative of the Activated Sludge influent for all parameters and will be used for all subsequent analyses.

The Effluent Filter Influent (12) and Plant Effluent (16) sample points were taken both at the actual location and remotely. The results were comparable confirming that the existing samplers were representative; therefore, the existing sampler protocol remotely located will be continued.

4.2 Unit Process Performance

Figure 13 shows the average total BOD, carbonaceous BOD, soluble BOD and TSS through the facility. Figure 13 includes only samples taken at new locations from September 8, 2007 through December 31, 2007.

Figure 13 BOD, CBOD, SBOD and TSS through WRF

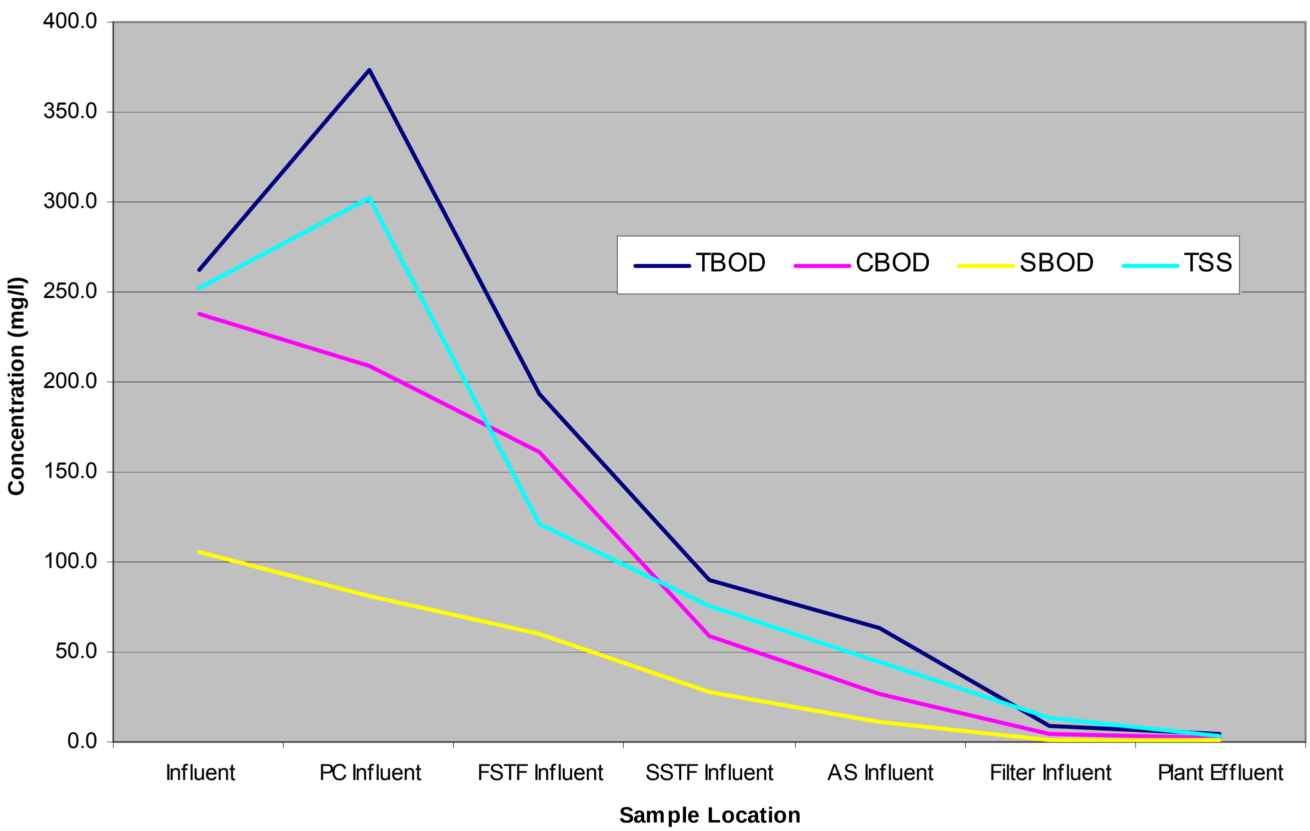
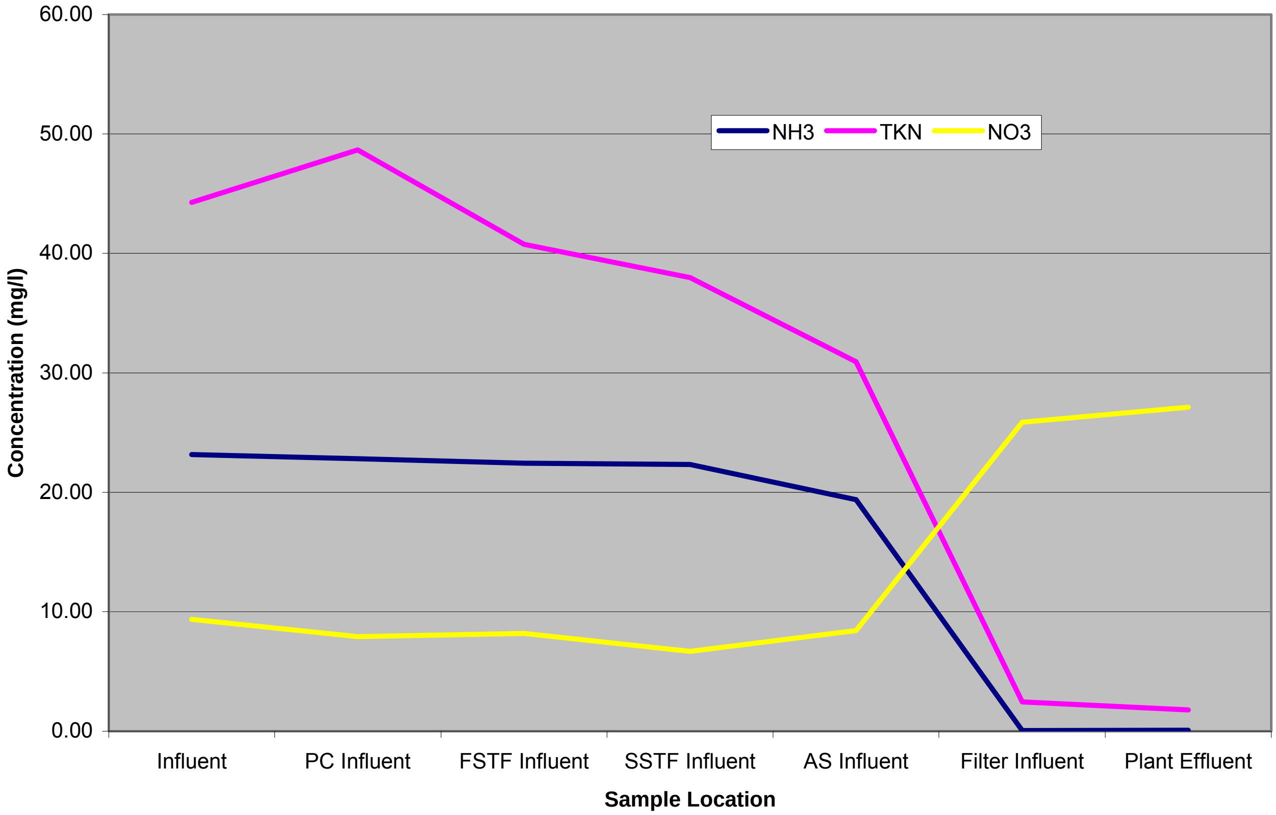


Figure 13 shows the influent organic strength increasing after the in-plant waste is added. CBOD and SBOD do not increase following in-plant waste addition because these parameters are less dependent upon solids. The return waste is high in solids content, and therefore only TBOD and TSS are affected. The primary clarifiers and first stage trickling filter processes reduce the organic strength substantially. Figure 13 shows the WRF processes are efficient at reducing organic strength.

Figure 14 shows the ammonia and TKN reduction through the facility unit processes. The data included was collected from September 8, 2007 through December 31, 2007.

Figure 14 Ammonia and TKN Reduction through WRF



As evident by the data in Figure 14, ammonia and TKN reduction occurs in the activated sludge process. Because the activated sludge basin was designed as the nitrification step at the WRF, the majority of TKN and ammonia are expected to be reduced in this process, as evident by Figure 14. The activated sludge basin appears to be nitrifying effectively as the TKN and ammonia are reduced below 3 and 1 mg/L in the WRF effluent, respectively. Nitrate concentrations rise following activated sludge, as would be expected following organic ammonia oxidation. The activated sludge system is generally operated at one-half the total available volume. There are no performance concerns related to the activated sludge basin at this time.

The overall facility has had excellent removal efficiencies throughout the years. The 2006 and September through December 2007 removal efficiencies are as follows:

Parameter	2006	2007
BOD	98.4%	98.5%
TSS	98.8%	98.4%
Ammonia	99.0%	99.7%
TKN	96.5%	96.0%

5.0 HYDRAULIC MODEL

A hydraulic model is a useful tool to evaluate a facility's current hydraulic bottlenecks and potential future deficiencies. A reliable hydraulic model begins with an analysis of the existing structures, processes and piping arrangements and their theoretical headloss contributions based on accepted industry equations. After the theoretical model is established, a field calibration must be conducted to verify actual conditions are consistent with the theoretical model.

5.1 Hydraulic Model Development

To evaluate the hydraulic capacity and performance of the Sioux Falls WRF, a hydraulic model was developed to predict the water surface elevation (WSEL) occurring in structures throughout the plant at various influent flow rates. By predicting the WSEL a hydraulic profile can be developed for each flow scenario. The goal of the hydraulic model for the WRF is to identify hydraulic bottlenecks or deficiencies throughout the system. These deficiencies include but are not limited to flooding of weirs, flooding of structures, backflow situations, incorrectly closed or partially closed valves, and so on. By identifying the deficiencies that are evident at a particular flow rate, a hydraulic capacity for the WRF can be determined. Based on the established hydraulic capacity, recommendations can be made to alleviate hydraulic constraints or perform improvements that may allow the WRF to increase its hydraulic capacity.

The hydraulic model for the Sioux Falls WRF examines all structures involved in the treatment of the liquid-stream portion of the plant. The model begins at the post aeration basin and ends at the influent Parshall Flume located in the Grit/Screening Building. Though the model appears counterintuitive to the natural progression of water through the plant, the proper way to predict the WSEL in a structure is to calculate headloss occurring upstream of the structure.

The hydraulic model contains all plant structures including: pressure and gravity pipes, channels, bends, fittings, valves, tanks, weirs, treatment units, basins, splitter structures, manholes, and flow measurement devices. The model was developed by consulting record drawings documented during the original construction of the WRF. The invert, weir and wall elevations and characteristics for each item from the record drawings were inserted into the model such as pipe diameters and materials, weir types and lengths, tank and basin dimensions, valve types and sizes, and channel properties. An extensive field survey of the WRF was conducted by Ulteig Engineers as part of this project to evaluate the accuracy of the elevations in the record drawings and to identify and resolve any discrepancies between the field survey and the record drawings. Once the elevations of inverts, weirs, tanks, etc. were verified or corrected by the field survey, this information was entered into the model. By applying industry accepted headloss and flow equations to the WRF structures, such as Hazen-Williams and Manning, and quantifying the proper flow-split scenarios based on the number of treatment units in service (i.e. one final clarifier down for maintenance) the model was assembled.

5.2 Hydraulic Model Calibration

By inserting an influent flow rate into the model, a theoretical hydraulic profile for the WRF is produced. Field measurements were conducted for three different flow rates across the WRF. By comparing theoretical model results to the field measured results, a calibrated hydraulic model is generated that can be utilized to predict system capacities and identify hydraulic deficiencies at the Sioux Falls WRF.

5.2.1 Field Testing Program

To calibrate the theoretical hydraulic profile generated by the model, a field testing program was implemented. During field testing the water surface elevations (WSEL) of three (3) separate flow rates were measured at various structures. Three flow rates were selected to ensure an accurate, three-point calibration for the model. The flow rates requested by Ulteig Engineers were 15 MGD, 18 MGD, and 27 MGD. These rates were selected based on typical average day and peak design flow rates. The 18 MGD flow rate was selected as a mid-range-value to provide a three point calibration to the model.

Prior to field testing at the elevated flow rates (i.e. 18 and 27 MGD), WRF personnel retained wastewater in the collection system equalization basin. The additional storage volume was necessary to sustain the flow rates throughout each of the higher test periods. For the duration of each field test, pumps at the Brandon Road Lift Station were adjusted to transport the appropriate, requested flow rate to the headworks of the Sioux Falls WRF. Over the course of each test, the actual influent flow rate to the WRF was regularly monitored so the true flow rate could later be input into the hydraulic model. The actual average influent flow rates recorded over the course of all three field tests were as follows: 14.73 MGD, 17.72 MGD, and 27.125 MGD.

During each field test, the WSEL at each structure was carefully measured and recorded. The theoretical model was then adjusted or “calibrated” to reflect the actual WSEL measured at each of the structures for all three flow rates. Calibration was accomplished by adjusting (within industry accepted ranges) pipe friction factors (Hazen/Williams "C" factor between 90 and 130) and channel friction factors as well as headloss and flow equations.

5.2.2 Acceptable Levels of Calibration

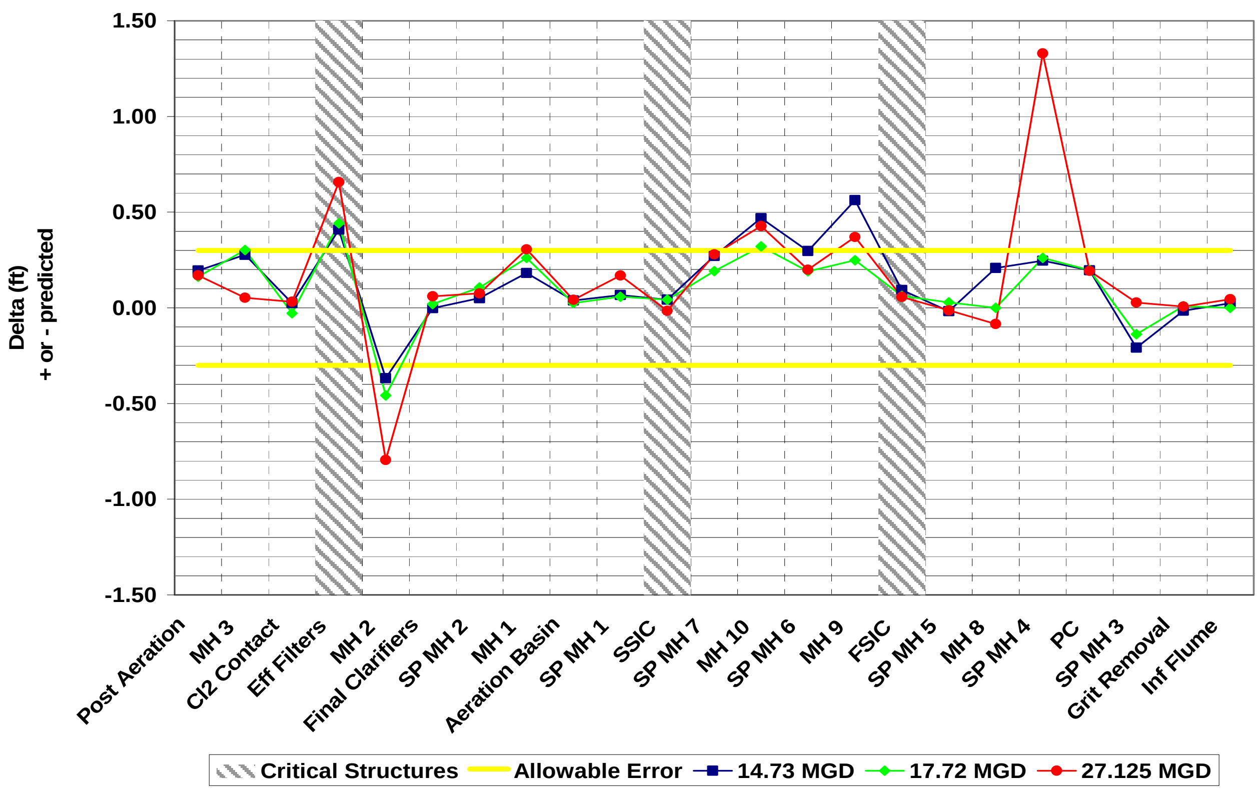
Due to wave-action experienced in several structures, the field recorded WSEL will often be valid over a range of elevations. In addition, the headloss equations used to simulate the hydraulic characteristics of each structure are generally empirical relationships, and therefore every hydraulic simulation will fundamentally have some degree of error built into the model. Wave-action experienced through splitter manhole No. 3 is illustrated in Figure 15.

Figure 15 Typical Wave-action in Splitter Manhole No. 3

Though all structures are significant to the final calibrated model, and subsequently simulated hydraulic profile, certain structures must have a greater level of calibration since these highly critical structures tend to determine the hydraulic capacity of the entire facility. Structures considered hydraulically critical are first identified via the observations noted by the field crew during the calibration measurements and also through the historical record of the WRF. Structures identified as "bottlenecks" are those which have historically flooded or backed-up during periods of high flow. WRF personnel were essential in helping determine which structures required in-depth evaluation. Conversely, the acceptable level of calibration must apply to all structures within the facility to ensure an adequate level of confidence in the model's accuracy.

Based on the WSEL measurements taken in the field and the significance of non-critical structures, the acceptable level of calibration for the Sioux Falls WRF is 0.3 feet plus-or-minus. Due to the range of WSELs measured at each flow rate (as a result of wave-action during field measurements), the level of accuracy was determined to be 0.3 feet. Therefore, the final hydraulic simulation results must be within 0.3 feet of the field recorded WSELs for all structures at all three flow rates to ensure confidence in the model's accuracy. The difference between the simulated and field recorded WSEL is often referred to as "delta." Critical structures should be calibrated to a delta less than the accepted value (i.e. 0.3ft) due to their significance to the hydraulic capacity of the facility. Figure 16 below illustrates the delta results of the model before calibration.

Figure 16 Sioux Falls Hydraulic Model Prior to Calibration



Due to the elevated WSELs experienced through splitter manhole No. 4 at higher flow rates, additional field measurements were necessary to calibrate the WSEL. The manufacturer's headloss equation for the FSTF distributors was input into the hydraulic model, however it could not be fit to the field measured data at the high flow rate of 27.125 mgd. The effluent pipes from splitter MH 4 and/or the first stage trickling filter distributors experience higher headloss than theoretically predicted. The elevated headloss may be due to substantial corrosion in the FSTF distributors (as shown in Figure 17), restrictions in the distributor arm nozzles, or due to the piping arrangement between splitter manhole No. 4 and the FSTFs; however, without further field testing the exact cause for flow restriction could not be definitively stated.

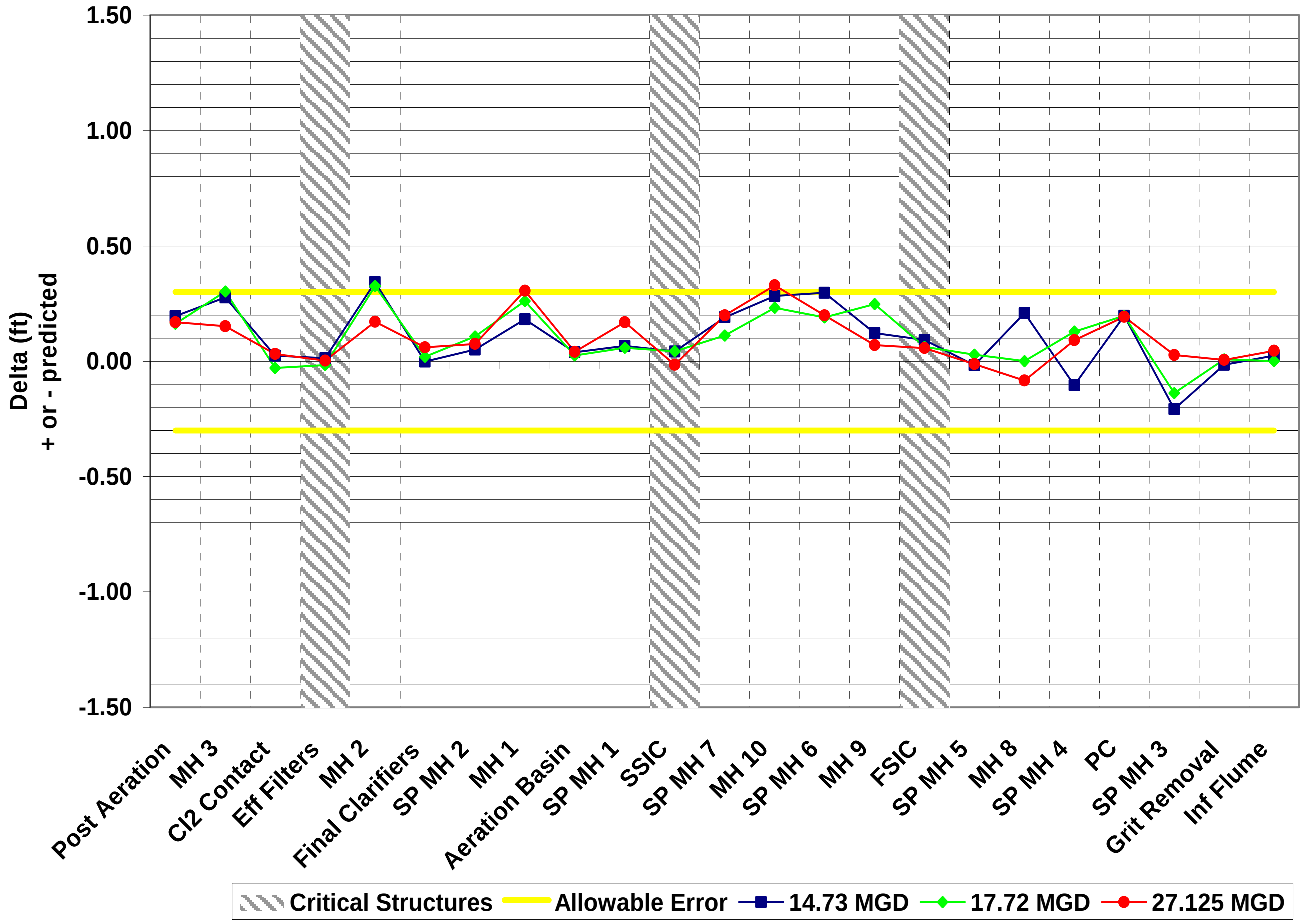
Figure 17 First Stage Trickling Filter Distributor Corrosion

Additional field testing was performed on October 11, 2007 to confirm or disprove the assumption that flow restriction was occurring in the FSTF distributor arms. The end caps of the distributor arms were removed and a flow rate of 28.0 mgd was sent to the WRF from the Brandon Road Lift Station. After the end caps were removed, the flow was no longer restricted and free flow conditions existed over the weirs in Splitter Manhole No. 4. Therefore, the elevated headloss condition and submergence at higher flow rates seen in Splitter Manhole No. 4 are caused by the physical condition of the FSTF distributor arms.

To develop a relationship for the existing FSTF/splitter manhole No. 4 headloss scenario, two additional flow rates (20.4 mgd and 24.2 mgd) were field tested at the WRF. The WSEL was measured in splitter manhole four, and a subsequent equation for headloss in the FSTF distributors and splitter manhole 4 was developed. The field derived headloss equation provides a better fit than the manufacturer's original equation as the manufacturer's equation was developed for a new distributor. The same back-up was not experienced in the second stage trickling filters. This is possibly due to the elongated distributor arms, or to the fact the FSTF created a bottleneck which dampened the high flows during field testing. Further field testing was conducted with total bypass of the FSTFs. At 28.0 mgd free fall conditions existed over the weirs in Splitter Manhole No. 6; however, the weirs were nearly submerged. Therefore, at flow rates greater than 28.0 mgd, it appears the SSTF distributor arms may be a hydraulic constraint. In their present condition, it is doubtful whether the SSTF distributor arms can convey the manufacturer's rated flow capacity of 46.0 mgd without causing weir submergence in Splitter Manhole No. 6.

Along with the field derived equation for the FSTF/splitter manhole No. 4 headloss scenario, the model was calibrated by adjusting pipe and channel friction factors as well as headloss and flow equations. In Figure 18, the post calibration delta results are illustrated.

Figure 18 Sioux Falls Hydraulic Model After Calibration



Once the hydraulic model is calibrated within the acceptable level, reliable simulations can be undertaken. With the aid of a calibrated model, hydraulic deficiencies can be identified and analyzed at given flow rates. Scenarios to alleviate system deficiencies can be analyzed to potentially increase influent flows.

To hydraulically analyze the Sioux Falls WRF, flows were input using in a step-wise fashion. The model was analyzed up to the peak hourly flow rate of 48.6 mgd. The model will display error or warning messages if a structure overflows, if bypass occurs, if weirs submerge, or if surcharging takes place for a particular flow rate. By utilizing the calibrated hydraulic model, the hydraulic capacity for each structure within the WRF can be evaluated, and the subsequent hydraulic capacity for the WRF can be determined.

The model was calibrated with all process components in service with the exception of the following: 3 of the 4 primary clarifiers in service, 3 of the 6 activated sludge basins in service, 7 of the 8 effluent filters in service. The model was executed with all process components in service and can be modified for any combination of unit processes in service. The worst case scenario for the effluent filters was examined whereby maximum headloss conditions were assumed to exist through the filter media (i.e all filters were analyzed at their dirtiest). The results of the hydraulic analyses for the facility components are presented in Table 3.

Table 3 Hydraulic Deficiencies of the Facility Components

Structural Component/Unit Process	Flow (mgd)	Observed Conditions
Effluent Filters	22.0	Bypass begins to Cl_2 Contact
Second Stage IC(1)	26.5	Weirs submerged
Manhole 11	26.5	Surcharging begins
First Stage Trickling Filter	27.0	Distributor capacity exceeded
Splitter Manhole 4	27.0	Surcharging begins
Chlorine Contact	28.0	Effluent flume submerged
Second Stage Trickling Filters	28.0 – 30.0	Distributor capacity exceeded
Primary Clarifiers(2)	30.0	Weirs submerged
Splitter Manhole 7	31.0	Weirs submerged
Splitter Manhole 3(2)	31.0	Weirs submerged
Splitter Manhole 4(3)	35.0	Structure freeboard limited
Splitter Manhole 1	36.0	Weirs submerged
Primary Clarifiers(2)	37.0	Structure freeboard limited
Aerated Grit Removal(2)	38.0	Effluent weir begins to flood
Chlorine Contact	41.0	Structure freeboard limited
Effluent Filters	41.0	Effluent weir begins to flood
First Stage IC	46.0	Weirs submerged
Splitter Manhole 5	48.0	Weirs submerged

(1) Due to limitations of process pump station operations.

(2) These limitations are directly related to the back-up experienced in splitter manhole No. 4 as a result of the hydraulic limitations of the FSTF distributors.

(3) Once overflow conditions occur, model reliability is reduced due to the inability to simulate overflow quantities.

An aerial photo showing the facility components is shown in Figure 1 of this report.

6.0 ORGANIC MODEL

The hydraulic capacity of the WRF pipes and unit processes will be determined using the hydraulic model developed as discussed in the previous section. However, the ability to send water through the facility does not ensure adequate treatment in the unit processes. An organic model of the secondary treatment system was investigated to determine a predictability of treatment at various flows and loadings.

6.1 Sampling Plan

During analysis of data for the organic model it was discovered some sample points may not be representative of the actual conditions. Additional samplers were placed at the actual sample sites to compare with existing samplers remotely located, as was discussed in Section 4.1. Sample points 01, 02, and 03 will be taken at the new locations at the site. Sample points 10A, 12 and 16 will be taken at the existing locations. All sample points for the organic model are representative of the actual conditions. The data used is from September 8, 2007 to December 31, 2008.

6.2 Organic Model Development

The secondary treatment process at the Sioux Falls WRF includes First Stage Trickling Filters and Second Stage Trickling Filters, each with a set of Intermediate Clarifiers. The secondary process was further described in previous sections. Figure 2 of this report shows the flow to the WRF has been higher than the original design whereas the BOD load to the WRF has been lower than the original design. With the lower BOD loading to the WRF, the loading to the secondary treatment process is also well below the design values. Thus, the emphasis for modeling the organic removal in secondary treatment process has been focused on the First Stage Trickling Filters.

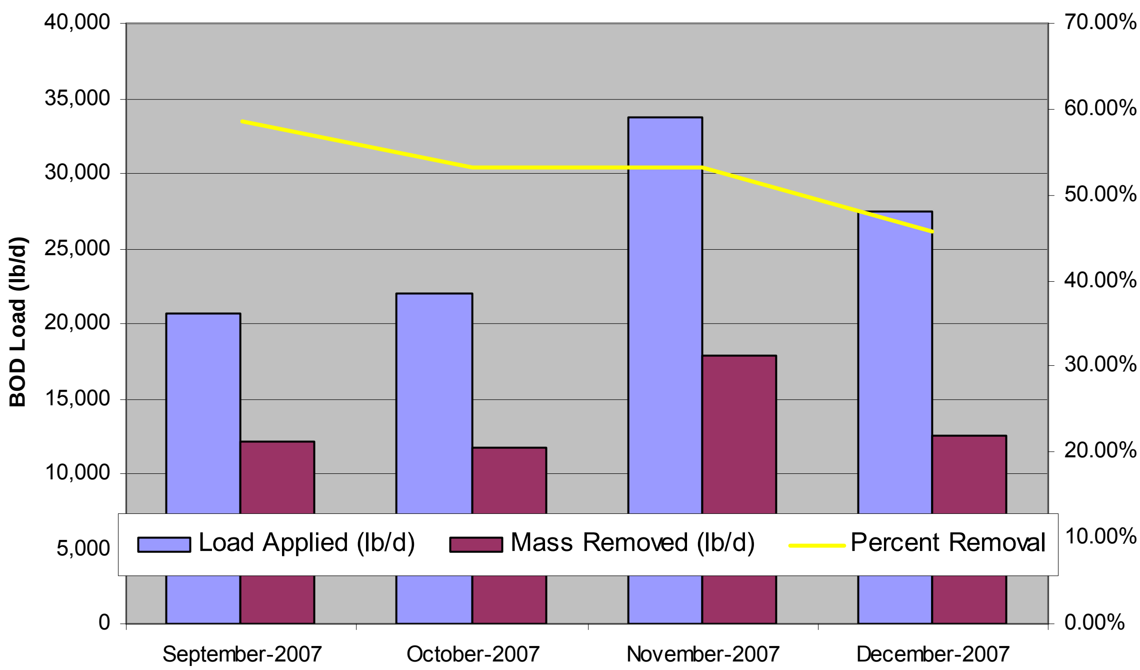
Several BOD efficiency or rate kinetic equations for removal of TBOD, CBOD, and SBOD have been preliminarily evaluated and compared. As stated in American Society of Civil Engineers (ASCE) Manual of Practice (MOP) #8, "All Velz, Schultze, Germaine, Eckenfelder and other researcher's reaction equations are fundamentally the same equation and have similar limitations. They have all been empirically derived and thus background data are influenced by a host of variables, such as:

- Hydraulic loading rate
- Dosing mode
- Temperature

- Soluble waste fraction
- Biodegradability
- Media depth
- Ventilation
- Model configuration and other variables including operation."

The variables listed above influence the treatability of the waste and the organic reduction efficiency of a trickling filter. Hydraulic loading rate, organic loading rate and temperature were studied to determine the most influential variables affecting the Sioux Falls FSTF performance. Figure 19 shows the average monthly BOD loading to the FSTF and the corresponding mass of BOD removed.

Figure 19 FSTF Loading, Mass Removed and Percent Removal (September – December 2007)



The previous figure shows that as loading to the FSTF increases, the mass of BOD removed also increases. However, the percent of BOD removal in the First Stage Trickling Filter process does not necessarily increase. Based on the limited data, the mass of BOD removed appears to be a better indicator of trickling filter performance rather than percent removal.

Figure 20 shows the monthly average BOD removal efficiency compared to temperature.

Figure 20 BOD Removal Compared to Temperature (September – December 2007)

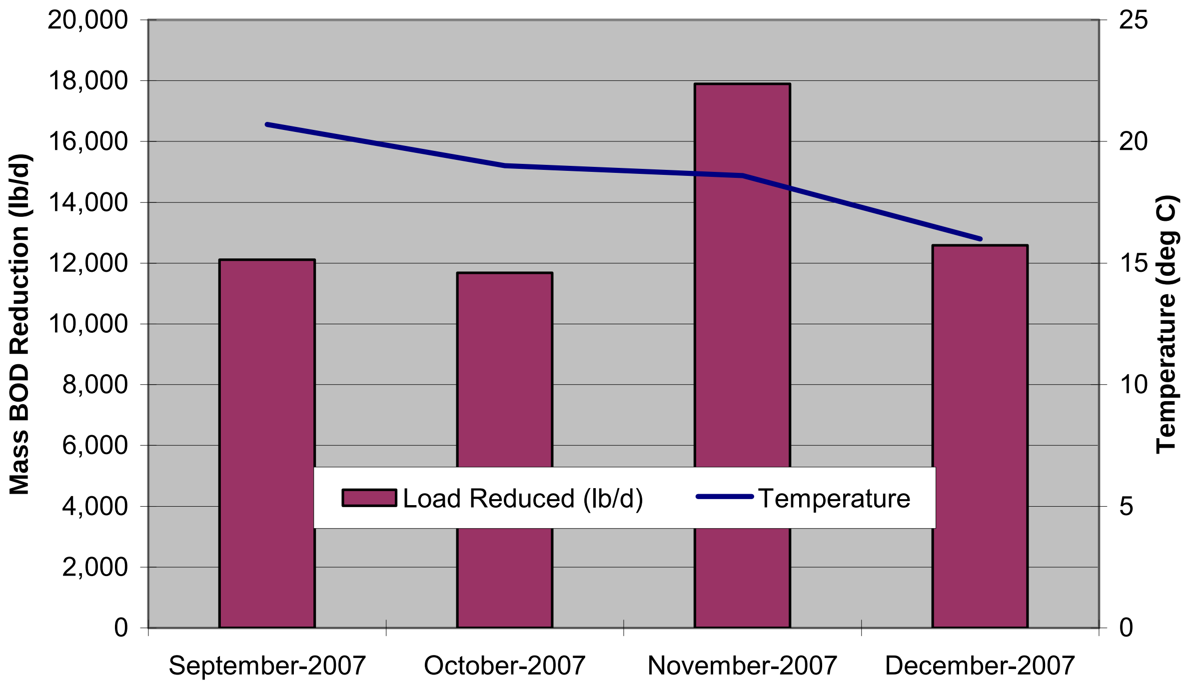


Figure 20 shows that temperature does not appear to have much affect on BOD mass removal at the Sioux Falls WRF. This can be explained by the covers on the trickling filters and heat retained by the biomass. However, with the significant change in the BOD load at the same time the temperature was decreasing, it is hard to separate one from the other. At this time a conclusive relationship between temperature and mass reduction can not be established.

Based on Figure 19 and Figure 20, the performance of the FSTF appears to be more dependent on mass applied as opposed to other variables at this time. Although the historical data and original design specifications show organic treatment capacity is available, to evaluate the performance of the secondary process at the WRF, an organic model has been established. By establishing a reliable simulation of performance, the model will aid in determining future organic capacity of the secondary treatment process.

6.3 Models Considered

Several attached growth models have been developed by others for process simulation of rock trickling filters. The models reviewed include the Velz (1948) model and two additional models that are hybrids of the original Velz model, the NRC model, Eckenfelder model and a model of regressed plant data. These models were subsequently used to predict the effluent BOD from September through December of 2007 and thus compared graphically to show the reliability of the final prediction of FSTF effluent.

Other attached growth models such as those by Schulze, Germain, Galler and Gottas have been suggested primarily for plastic media and have not been included in this evaluation.

6.3.1 Velz and Modified Velz Models

Velz used the theory that reduction of BOD was a first order reaction and proposed the BOD remaining at depth D as follows:

- ***Velz Equation:*** $LN(S_e/S_o) = -KD$
  - S_o = *Influent BOD*
  - S_e = *Effluent BOD*
  - K = *Treatability factor*
  - D = *Depth of filter*
  - LN = *Natural log*

In this case, the depth, D, was used to represent the time of contact with the microorganisms. However, this relationship does not consider the effects of hydraulic and/or organic loading. But is easy to use and has been found to give generally satisfactory results in predicting the treatment effectiveness. Temperature must also be considered in the kinetic relationship or treatability factor and is generally reported at 20 degrees C.

In the 1993 HDR report to the City of Sioux Falls, a variation of the Velz equation was used where time of contact or depth of media was replaced with the filter volume divided by the influent flow as follows:

- ***Modified Velz Equation:*** $LN(S_e/S_o) = -K(V/Q)^n$
  - V = *Filter volume*
  - Q = *Influent flow rate*
  - n = *Empirical media characteristic*

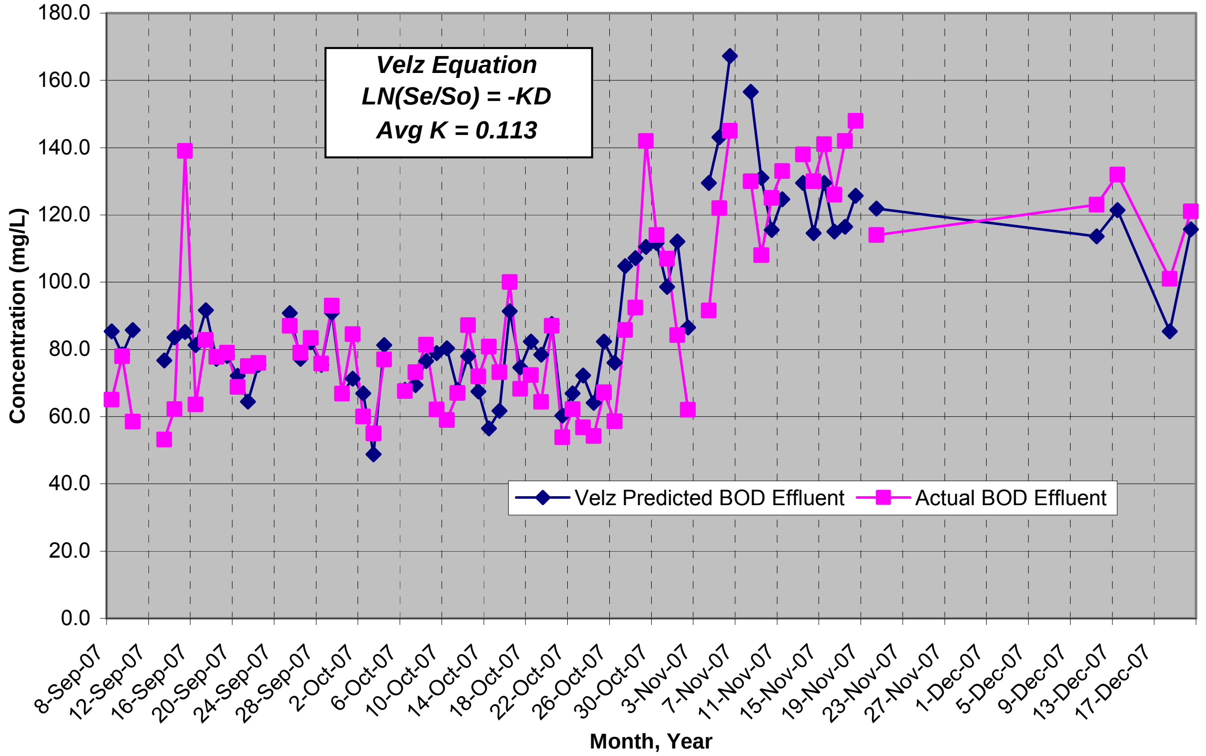
Whereas Velz reported the treatability factor to be equal to about 0.1505 to 0.175, the HDR report indicated a result that varied between 0.20 and 0.27. HDR further suggested that their K value predicted the filter performance well except during summer months where the difficulty in prediction of BOD reduction was attributed to nitrification. Further, due to regular occurrence of nitrification measured in the SSTF, treatability analysis for BOD reduction could not be performed.

Another modification of the Velz model was reported by Metcalf and Eddy and others around 1990, where the volume of the filter was represented by the specific media surface area and the depth of the filter. Again the hydraulic loading was included. The relationship suggested is as follows:

- ***Modified Velz Equation:*** $LN(S_e/S_o) = -KA_vD/Q^n$
  A_v = *Media surface area*

This relationship gives very close results to the earlier modification of the Velz formulation. All three of the models presented above have been utilized to predict the effluent from the FSTF. Figure 21 shows the Velz predicted BOD effluent from the First Stage Trickling Filter compared to the actual BOD effluent utilizing data from September to December 2007.

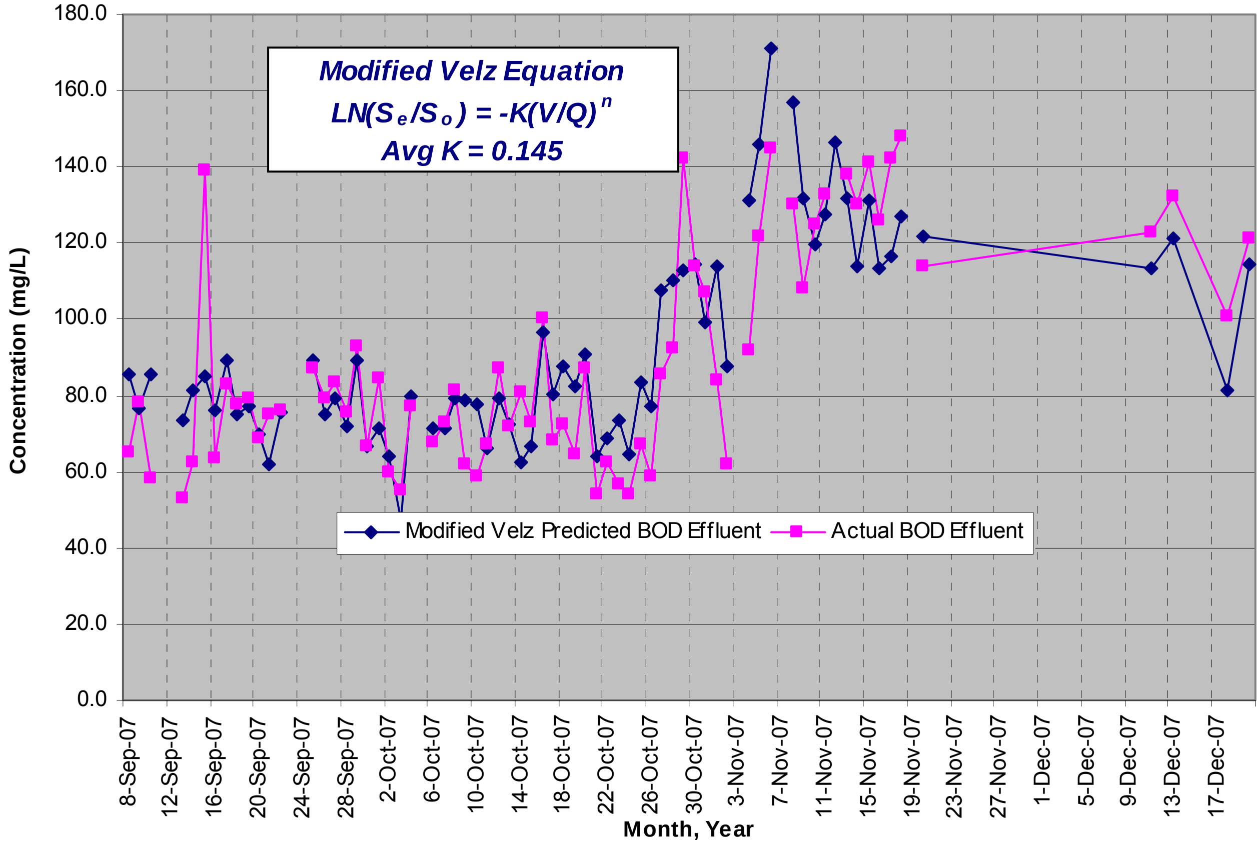
Figure 21 Velz Predicted BOD Effluent and Actual BOD Effluent Daily Data September – December 2007



For September through December 2007 data, the Velz equation yielded a K_{20} average value of 0.113.

Figure 22 shows the Modified Velz Equation predicted BOD effluent from the First Stage Trickling Filter process compared to the actual BOD effluent utilizing data from September to December 2007.

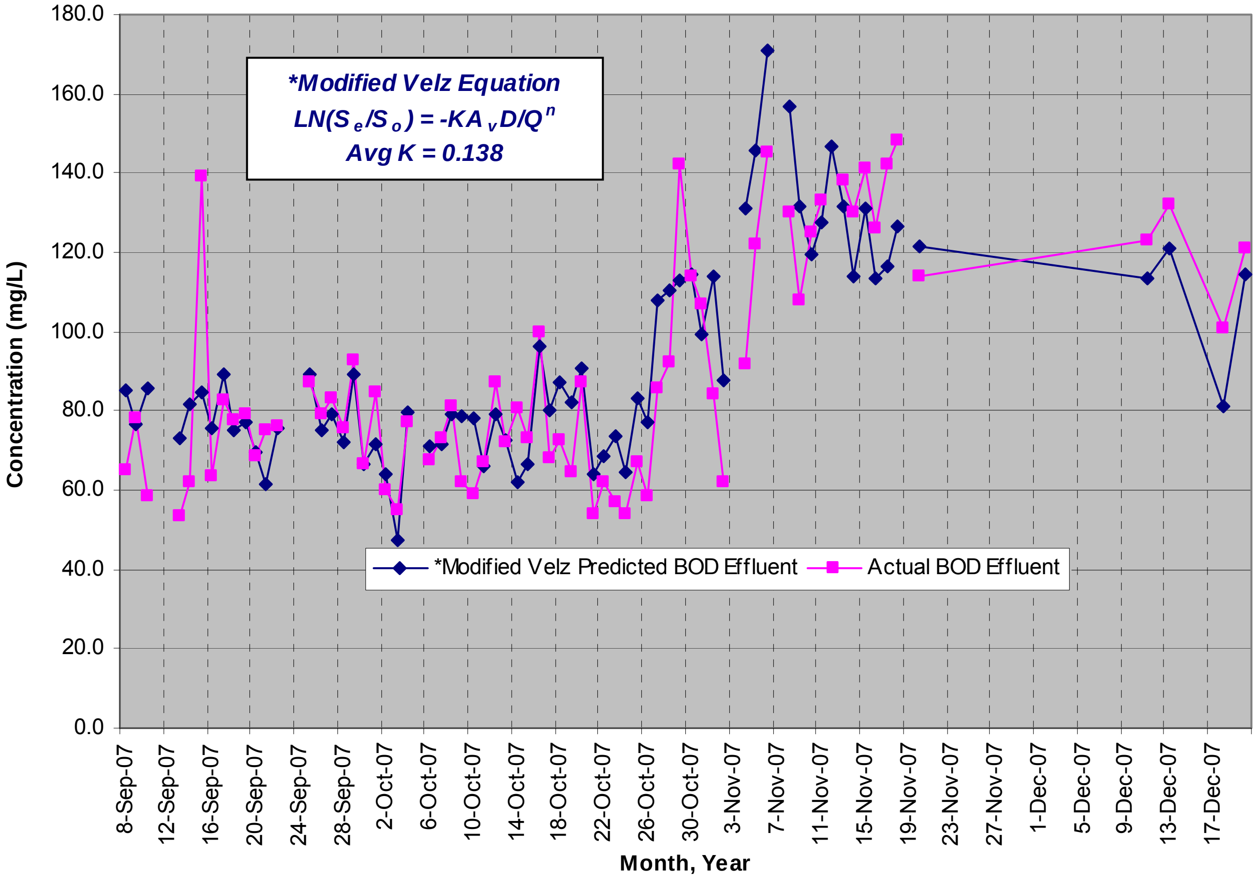
Figure 22 Modified Velz Predicted BOD Effluent and Actual BOD Effluent Daily Data September – December 2007



For September through December 2007 data, the Modified Velz equation yielded a K_{20} average value of 0.145.

Figure 23 shows the *Modified Velz Equation predicted BOD effluent from the First Stage Trickling Filter process compared to the actual BOD effluent utilizing data from September to December 2007.

Figure 23 *Modified Velz Equation Predicted BOD Effluent and Actual BOD Effluent Daily Data September – December 2007



For September through December 2007 data, the *Modified Velz equation yielded a K_{20} average value of 0.138.

6.3.2 NRC Model

The original design model for rock trickling filters was developed by the National Research Council (NRC) in the early 1940s. It was based on field data for BOD removal efficiency and organic loading rates. The equation for predicting removal efficiency across the first stage trickling filter is as follows:

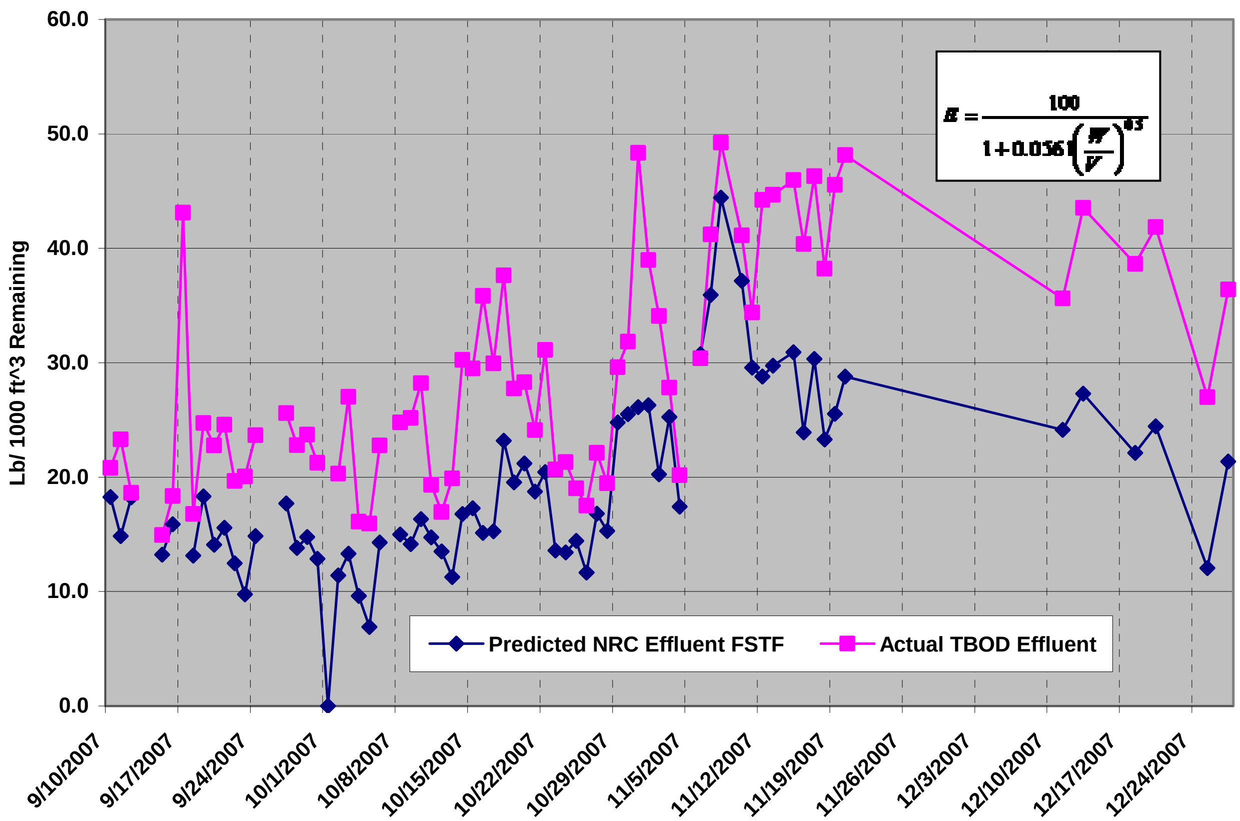
➢ ***NRC Equation:***

$$E_1 = \frac{100}{1 + 0.0561\left(\frac{W_1}{V_1 F}\right)^{0.5}}$$

W=BOD Loading, pounds per day
V = Volume, cubic feet x 1,000
F = Recirculation Factor (F=1 when R=0)
E_1 = Removal Efficiency FSTF

The NRC equation was used to predict the effluent from the first stage trickling filters. Figure 24 presents the NRC predicted effluent compared to the actual BOD effluent for 2007 data.

Figure 24 NRC Predicted BOD Effluent and Actual BOD Effluent Daily Data September – December 2007



For the second stage trickling filters, the NRC predicted removal efficiency is as follows:

- ***NRC Equation:*** $$E_2 = \frac{100}{1 + \frac{0.0561}{1 - E_1}\left(\frac{W_2}{V_2 F}\right)^{0.5}}$$

W_2 = BOD Loading, pounds per day
V_2 = Volume, cubic feet x 1,000
F = Recirculation Factor (F=1 when R=0)
E_2 = Removal Efficiency SSTF

The removal efficiency predicted by the NRC Equation for the second stage trickling filter process was evaluated for the SF WRF. However, the predicted removal was much higher than actual plant data and was eliminated as a suitable model.

6.3.3 Eckenfelder Model

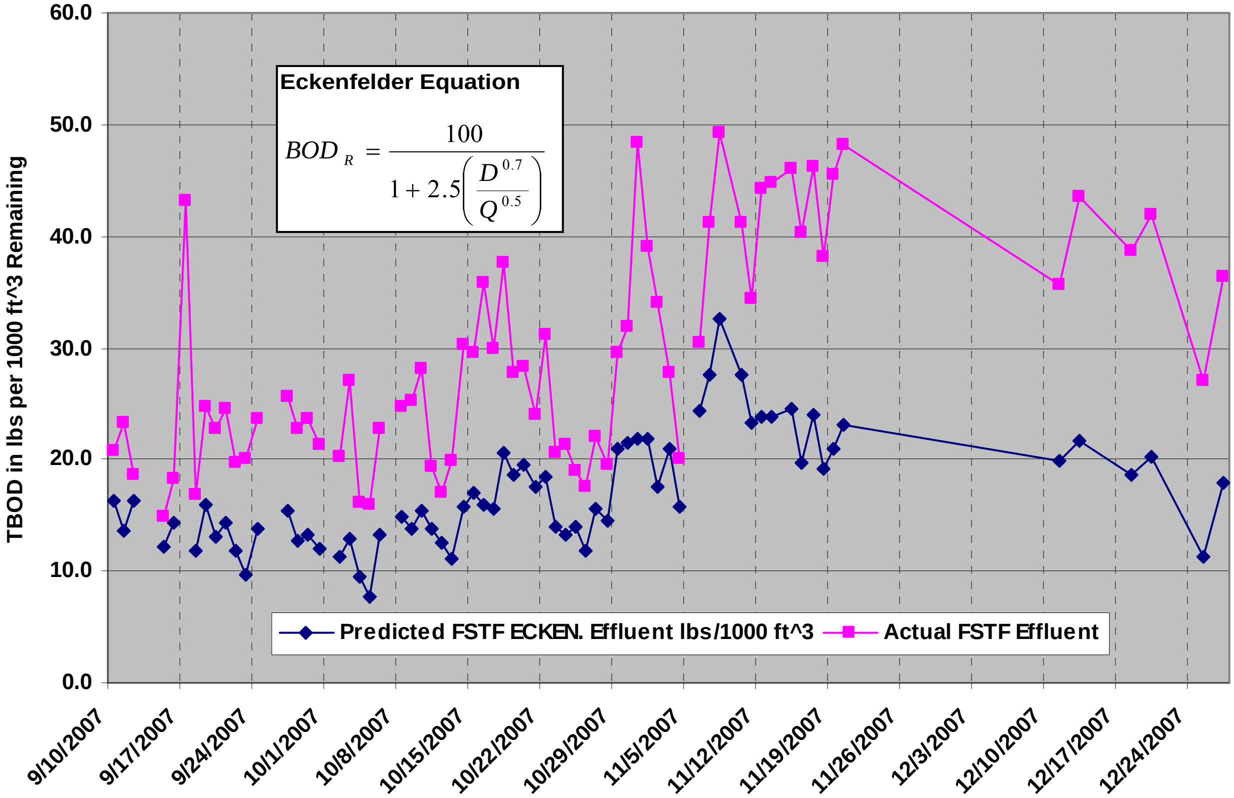
A version of the NRC equation by Eckenfelder was also used to predict BOD reduction in the first stage trickling filters. The Eckenfelder equation is as follows:

- ***Eckenfelder Equation:*** $$BOD_R = \frac{100}{1 + 2.5\left(\frac{D^{0.7}}{Q^{0.5}}\right)}$$

D = Depth
Q = Flow (mgad)

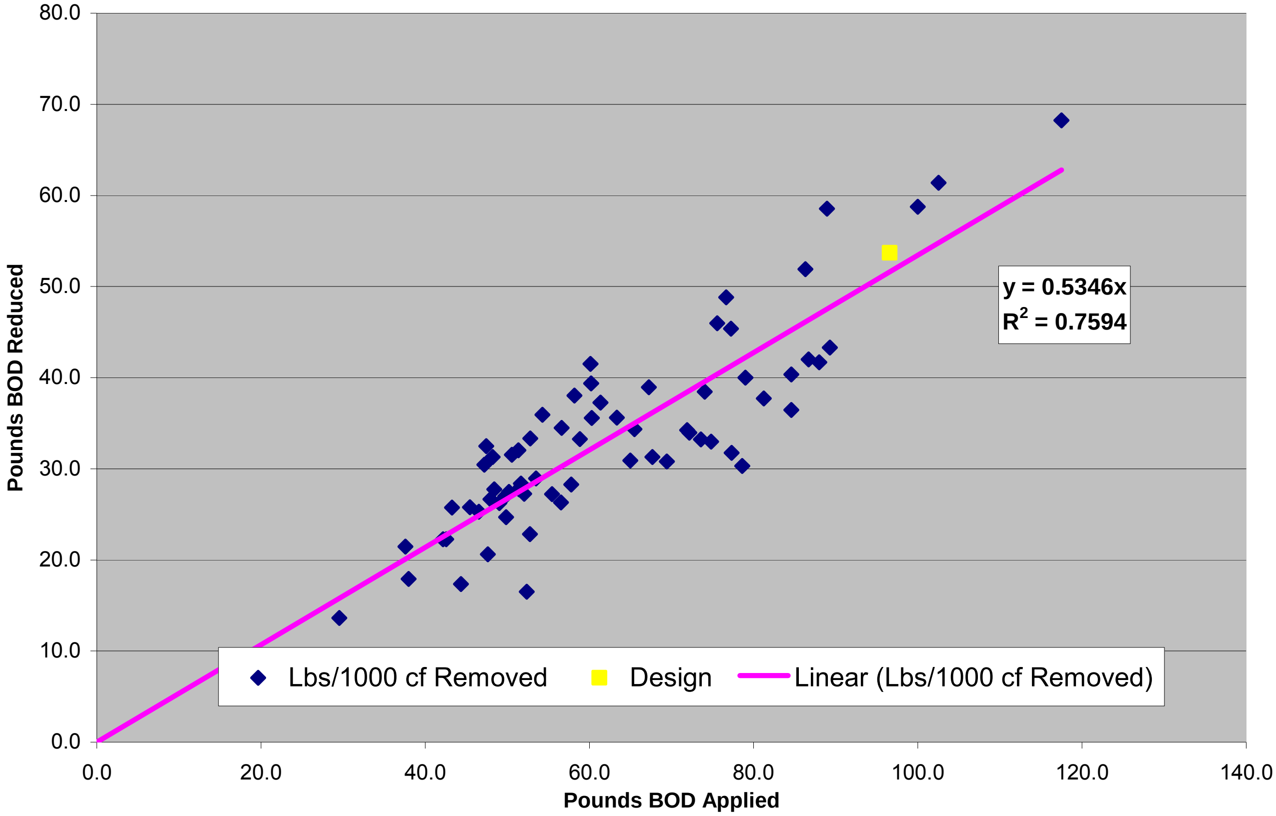
Figure 25 presents the Eckenfelder predicted daily effluent compared to the actual BOD effluent for September – December 2007 data.

Figure 25 Eckenfelder Predicted BOD Effluent and Actual BOD Effluent Daily Data September – December 2007



Regression of Plant Data in Figure 26 shows the mass reduction as a function of the BOD load applied on a daily basis from September 8 – December 31, 2007.

Figure 26 Regressed Model
Daily Data September – December 2007



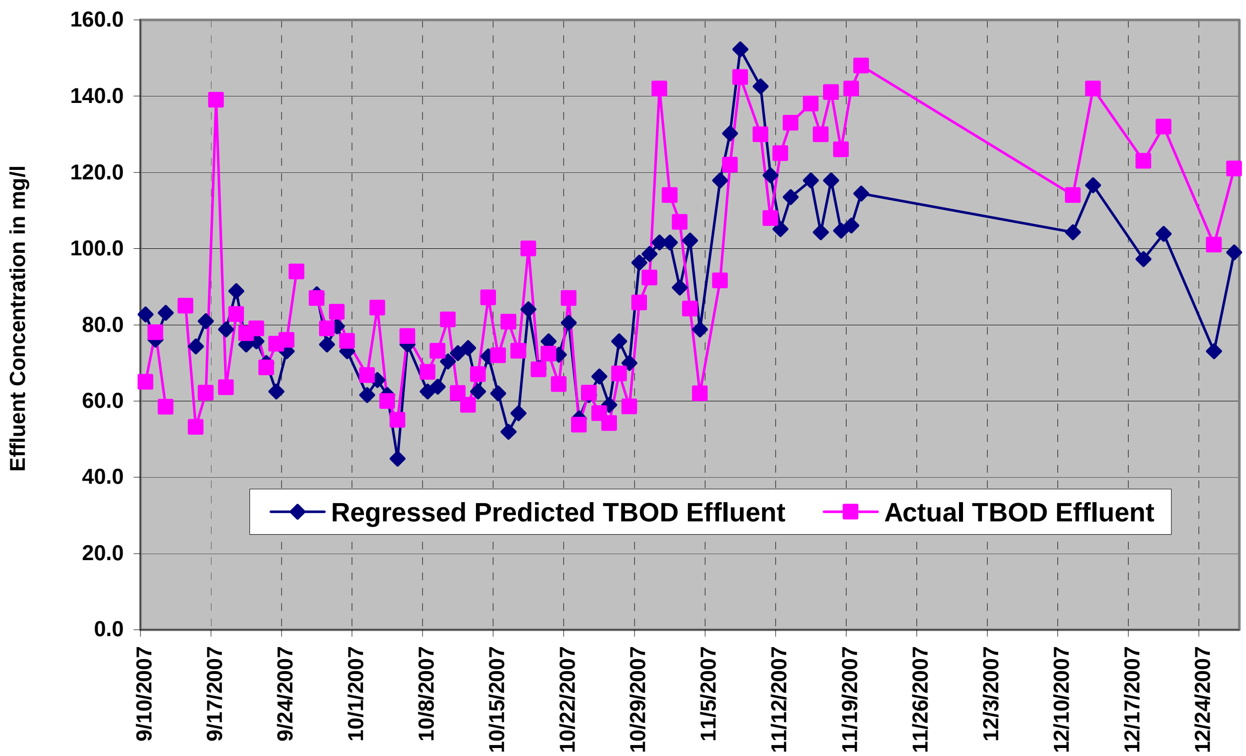
The regression equation generated from Figure 26 is as follows:

- ***Regressed Equation:*** $y = 0.5346x$

y = Mass BOD Reduced (lbs/1,000 cubic feet)
x = Mass BOD Applied (lbs/1,000 cubic feet)

The regression model was used to generate a graph showing the predicted BOD effluent and actual BOD effluent from the first stage trickling filter process and is shown in Figure 27.

Figure 27 Regressed Predicted BOD Effluent and Actual BOD Effluent

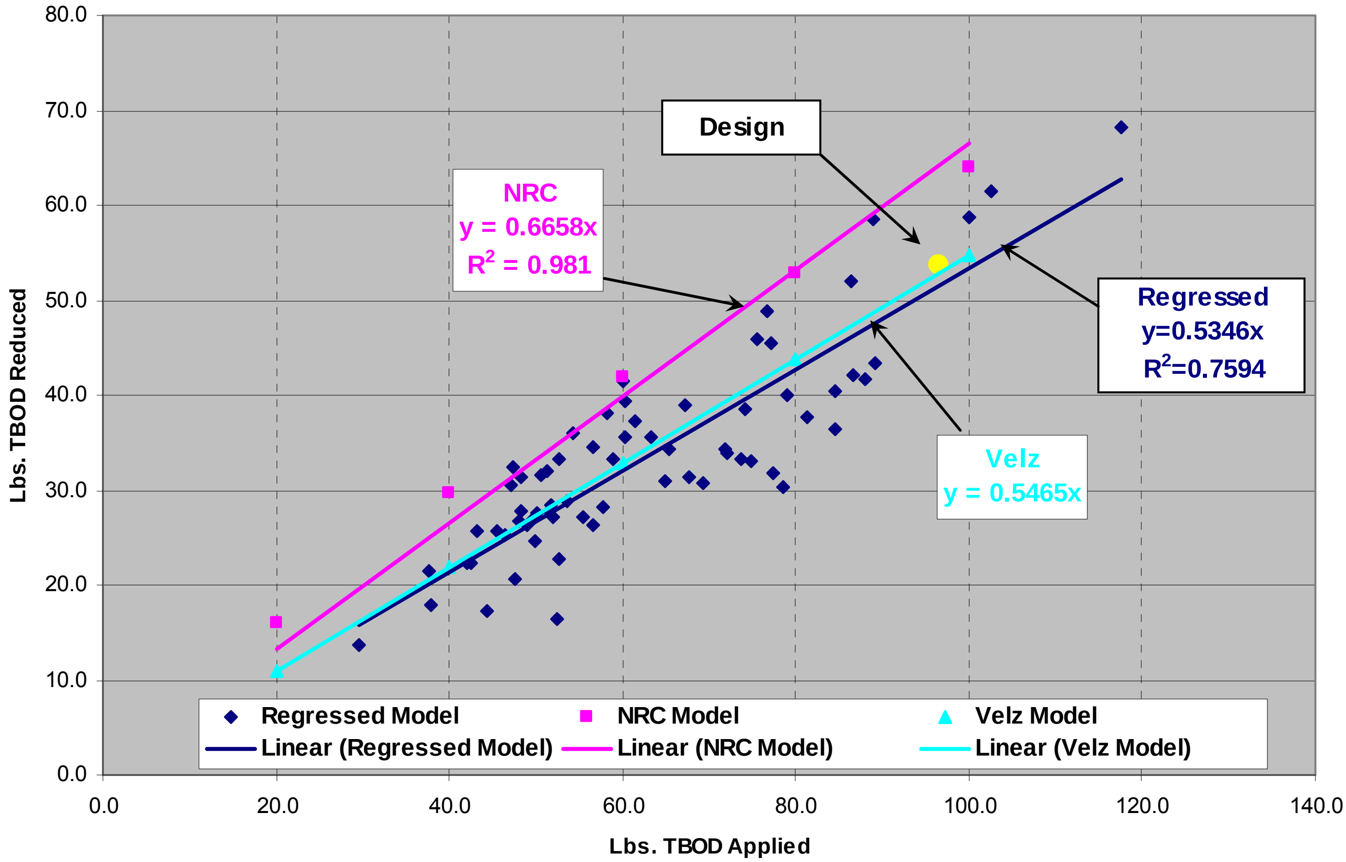


6.4 Discussion of Models Considered

All models show variation in predicting the actual effluent, sometimes greater than and sometimes less than the actual measured effluent concentration. This is expected as the final predictions are based upon the mean value of the treatability factor or the mean efficiency predicted by the NRC or Eckenfelder equation. This is true even with compensation for temperature, where necessary.

Figure 28 presents a compilation of the two best fit models (NRC and Velz) with the regressed data, and show good correlation but with slightly higher multipliers. The linear regressed line for actual data, forced to pass through zero, predicts about a 53.5% reduction of mass BOD applied with a regression coefficient of about 76%. The figure further shows the application of the NRC and Velz equation imposed on the actual data and compared to the regressed relationship.

Figure 28 NRC, Velz and Regressed Model BOD Reduction Prediction



The equations of best fit show:

- Y = 0.5346 X, Regressed plant data
- Y = 0.6658 X, NRC equation, and
- Y = 0.5465 X, Velz equation.

Where: Y = Mass BOD Reduced, and

X = Mass BOD Applied in lbs/1000 ft^3

Based upon the high removal approximation estimated by the NRC equation, it was rejected as a suitable model for the FSTF process. While the Velz model offered a good fit to the data, the regressed equation provided a better fit and presented a more conservative predictor for removal efficiency, when compared to the Velz equation. Therefore, the regressed equation is the recommended organic model for simulating BOD removal across the FSTF process at the Sioux Falls WRF.

6.5 Allowable Headworks Loading

Based on the performance data gathered from September through December of 2007 and the recommended Organic Model, the allowable headworks loadings for TBOD, TSS and TKN were determined.

6.5.1 TBOD Loading

An organic model of the first stage trickling filter was developed to predict BOD removal. Based on the sampling data to date, the organic model shows a BOD removal efficiency of 53.46% for historical organic loading conditions. This regressed model can be used in conjunction with the following parameters to determine an allowable loading to each unit process:

- Influent Activated Sludge Concentration of 50 mg/l or less
- Average Daily Influent Flow of 21 mgd
- Plant Effluent TBOD Concentration of 10 mg/l or less
- Unit Process Performance from September through December of 2007 (listed in Table 4)

Table 4 Unit Process TBOD Removal Efficiency

Process	Removal Efficiency
Primary Clarifier	48.2%
First Stage Trickling Filter	53.5%
Second Stage Trickling Filter	29.0%
Activated Sludge	85.6%
Effluent Filters (Discharge)	57.7%

At an influent average 2030 design flow of 21.0 mgd and an allowable Activated Sludge Influent of approximately 50 mg/l, the allowable BOD loading to the Activated Sludge system is 8,770 pounds per day. Working back to the headworks of the WRF, and expecting 29% reduction in the SSTF (as per the sampling analysis), this equates to a SSTF loading of 12,353 lb/d. Based on the organic model, a 53.5% reduction is predicted in the FSTF, equating to an allowable FSTF loading of 26,542 lb/d. Knowing the typical BOD reduction across the Primary Clarifiers is about 48.2%, this results in a calculated headworks loading of approximately 51,240 lb/d.

Upstream of the primary clarifiers, in-plant waste is added increasing the TBOD loading substantially. The in-plant waste is high in particulate matter and low in organic strength, as shown by the increase in TBOD and TSS and decrease in CBOD and SBOD in Figure 29. The organic removal processes (tricking filters and activated sludge basins) are intended to remove carbonaceous and soluble organics, not particulate matter. Clarifiers at the WRF are responsible for removing particulate matter. Because the clarifiers remove the particulate matter, organic removal process capacity should not be decreased to account for in-plant waste solids. Therefore, because of its high particulate strength and low organic strength, the in-plant waste will be accounted for in the allowable headworks loading calculations for TSS and not TBOD.

Figure 29 TBOD, CBOD, SBOD and TSS through WRF
September – December 2007

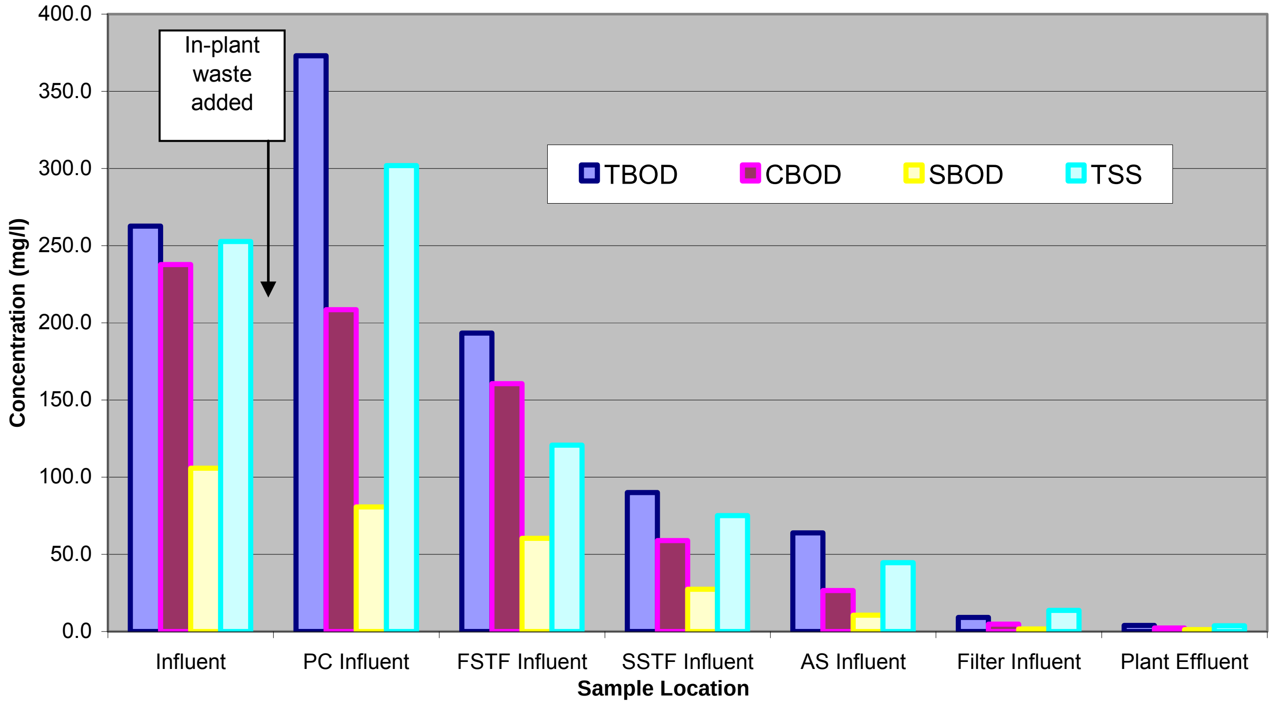


Table 5 summarizes the allowable TBOD loading to the various processes at the WRF.

Table 5 Allowable TBOD Loading

Process	Allowable Loading (lb/d)	Concentration at 21 mgd (mg/l)	Design Constraints (mg/l)
WRF Influent	**51,240**	**293**	
Primary Clarifier	26,542	152	
First Stage Trickling Filter	12,353	71	
Second Stage Trickling Filter	8,770	50	≤ 50
Activated Sludge	1,263	7	
Effluent Filters (Discharge)	535	3	< 10

Based on the above analysis, the recommended allowable TBOD loading is 51,240 lb/d (293 mg/l at 21.0 mgd).

6.5.2 TSS Loading

The performance data from September through December, 2007 was utilized to calculate the allowable headworks TSS loading and are shown in Table 6. The following parameters were used in the determination.

- Influent Effluent Filter Concentration of 20 mg/l or less
- Average Daily Influent Flow of 21 mgd
- Plant Effluent TBOD Concentration of 10 mg/l or less
- Primary Clarifier Design Removal Efficiency of 55% was used instead of Actual Efficiency (60%) to be conservative

Table 6 Unit Process Performance

Process	Removal Efficiency
Primary Clarifier	55%
First Stage Trickling Filter	37.8%
Second Stage Trickling Filter	40.7%
Activated Sludge	69.2%
Effluent Filters (Discharge)	72.5%

At an influent average 2030 design flow of 21.0 mgd and an allowable Effluent Filter Influent of approximately 18 mg/l (less than 20 mg/l to be conservative), the allowable TSS loading to the Effluent Filters is 3,167 pounds per day. Working back to the headworks of the facility, and expecting 69.2% reduction in the Activated Sludge system (as per the sampling analysis), this equates to a loading of 10,283 lb/d. Based on a 40.7% reduction in the SSTF the predicted loading to the SSTF is 17,338 lb/d. The TSS reduction in the FSTF system is approximately 37.8%, which equates to a FSTF influent loading of 27,855 lb/d. TSS reduction in the Primary Clarifiers of about 55% equates to a Primary Clarifier loading of 61,900 lb/d.

As mentioned in the previous section and shown in Figure 29, the in-plant waste is high in solids. The in-plant waste cannot be neglected in the allowable TSS loading calculations because the clarifiers must have capacity to remove the particulate matter returned to the head of the plant. The allowable headworks loading must be decreased to accommodate the solids returned in the in-plant waste. The projected 2030 in-plant waste TSS loading is estimated at 18,000 lb/d. This equates to an allowable influent TSS loading of 43,900 lb/d.

Table 7 summarizes the allowable TSS loading to the various processes at the WRF.

Table 7 Allowable TSS Loading

Process	Allowable Loading (lb/d)	Concentration at 21 mgd (mg/l)	Design Constraints
WRF Influent	**43,900**	**251**	
Primary Clarifier Influent	61,900	353	+ 18,000 lb/d
Primary Clarifier Effluent	27,855	159	
First Stage Trickling Filter Effluent	17,338	99	
Second Stage Trickling Filter Effluent	10,283	59	
Activated Sludge Effluent	3,167	18	≤ 20 mg/l
Effluent Filters (Discharge)	871	5	< 10 mg/l

Based on the above analysis, the recommended allowable TSS loading is 43,900 lb/d (251 mg/l at 21.0 mgd).

6.5.3 TKN Loading

The allowable TKN headworks loading is dependent on the blower capacity and diffuser characteristics of the activated sludge system. The following parameters were used in the determination.

- Influent Activated Sludge Concentration of 50 mg/l or less
- Average Daily Influent Flow of 21 mgd
- Plant Effluent TBOD Concentration of 10 mg/l or less
- Based on the design SRT of the Activated Sludge Basin, a BOD:O_2 Ratio of 1 was used.
- Based on the theoretical oxygen demand for nitrification, a 4.6 lb O_2 per lb of TKN was used.

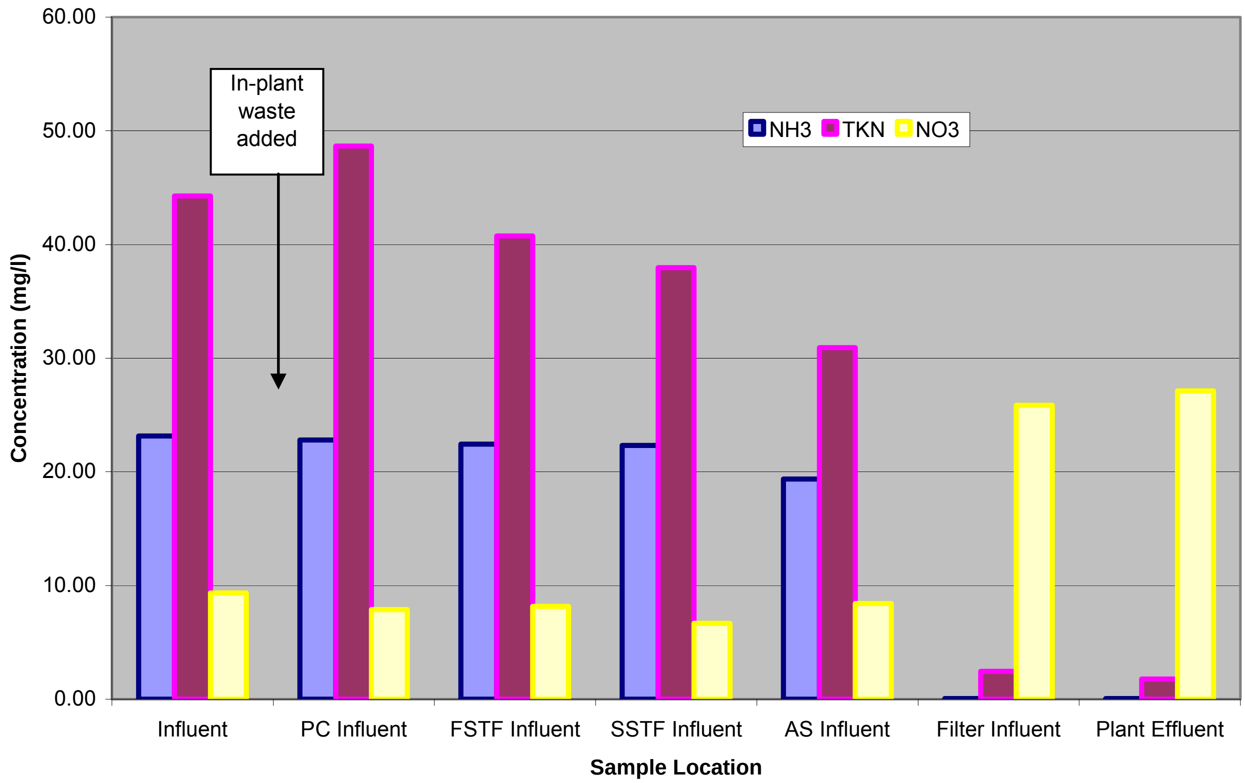
- Three blowers in service with a total oxygen capacity of 52,200 pounds per day were used.

The activated sludge basins were designed as the nitrification process at the WRF. Therefore, it was assumed no nitrification will occur upstream of the activated sludge process at 2030 design conditions. Because the primary and secondary processes were designed to achieve an effluent concentration of 50 mg/l, the BOD loading must be considered in the total oxygen demand calculations in the activated sludge system.

As previously mentioned, in-plant waste is added upstream of the primary clarifiers, increasing the TKN loading. The in-plant waste is high in TKN but low in ammonia, as shown by the increase in TKN and the unchanged ammonia concentration in Figure 30. Therefore, the TKN in the in-plant waste is mostly comprised of organic nitrogen and not ammonia. The particulate matter in the in-plant waste is high in organic nitrogen because it consists of facultative sludge basin supernatant and tertiary nitrification waste activated sludge (WAS). The biomass responsible for treatment in the supernatant and WAS metabolize ammonia. The product of metabolism results in biomass comprised of organic nitrogen, which explains the high organic nitrogen content present in the in-plant waste.

After the primary clarification process, the TKN concentration is below the influent TKN concentration and the ammonia concentration remains unchanged. The organic nitrogen is removed with the particulate matter in the primary clarifiers. Because the in-plant waste TKN is primarily organic nitrogen removed in the primary clarifiers, the TKN capacity of the activated sludge system should not be reduced.

Figure 30 TKN, Ammonia and Nitrate Concentration at WRF
September – December 2007



The flow (21 mgd), blower capacity (52,200 lb/d) and influent BOD concentration (50 mg/l) were kept constant while varying the TKN concentrations to determine the maximum allowable TKN capacity. When the total oxygen demand for nitrification is greater than the blower capacity, the allowable TKN loading is exceeded. Table 8 shows the WRF allowable TKN capacity determination.

Table 8 WRF Allowable TKN Capacity

Flow (mgd)	Blower Capacity (Lb O_2)	TKN (mg/l)	TKN (Lbs)	BOD (mg/l)	BOD (Lbs.)	O_2 Demand TKN	O_2 Demand BOD	Total O_2 for Nitrification
21	52,200	20	3,502.8	50	8,757	16,113	8,757	24,870
21	52,200	25	4,378.5	50	8,757	20,141	8,757	28,898
21	52,200	30	5,254.2	50	8,757	24,169	8,757	32,926
21	52,200	35	6,129.9	50	8,757	28,198	8,757	36,955
21	52,200	40	7,005.6	50	8,757	32,226	8,757	40,983
21	52,200	45	7,881.3	50	8,757	36,254	8,757	45,011
21	52,200	50	8,757.0	50	8,757	40,282	8,757	49,039
21	52,200	52	9,107.3	50	8,757	41,893	8,757	50,650
21	52,200	53	9,282.4	50	8,757	42,699	8,757	51,456
21	52,200	53.5	9,369.9	50	8,757	43,102	8,757	51,859
21	52,200	53.7	9,405.0	50	8,757	43,263	8,757	52,020
21	52,200	53.9	**9,440.1**	50	8,757	43,424	8,757	52,181
21	52,200	54	9,457.6	50	8,757	43,505	8,757	52,262

Based on the above analysis, the recommended allowable TKN loading is 9,440 lb/d (53.9 mg/l at 21.0 mgd).

7.0 UNIT PROCESS DESIGN, REGULATORY AND PERFORMANCE ANALYSIS

The facility design, facility performance and recommended standards were compared in determining if the WRF can handle increased flow and/or load. A number of design standards were referenced to determine performance guidelines for the WRF unit process evaluation.

7.1 Standards Used

The Recommended Standards for Wastewater Facilities (10 States Standards) is commonly used in the planning and design of wastewater treatment facilities across the country. The state of South Dakota also has Design Criteria for use in planning and design.

7.1.1 Recommended Standards for Wastewater Facilities, 2004 Edition

The Recommended Standards for Wastewater Facilities, commonly referred to as the Ten States Standards, is a widely accepted resource for planning and design of wastewater treatment facilities. The first edition of the Ten States Standards was published in 1951 to provide a guide in the design for the average conventional wastewater treatment facility. The Standards have be subsequently revised and published again throughout the years to the current edition, 2004.

The Ten State Standards suggest limiting values for unit process design and to attempt to establish uniformity of design practices. It should be noted that that the design criteria are intended for more conventional collection and treatment systems. Innovative approaches to treatment are not included.

7.1.2 South Dakota Department of Environment and Natural Resources (SD DENR) Design Criteria

The SDDENR has published Design Criteria similar to the Ten States Standards to establish uniformity of practice and as a guide in the design and preparation of plans and specifications. The Design Criteria set the minimum requirements for compliance with the reviewing authority, the SD DENR.

7.1.3 Other Reference Materials Used

Other wastewater treatment facility reference documents were consulted during the analysis of the facility.

- ✓ Metcalf & Eddy, Wastewater Engineering Treatment and Reuse 4th Edition (2003).
- ✓ US EPA, Nitrogen Control Manual of Practice (1993).
- ✓ Water Environment Federation, Design of Municipal Wastewater Treatment Plants, Manual of Practice No. 8 (1992).

7.2 Design and Performance Analysis

The Sioux Falls Water Reclamation Facility was designed to treat the parameters shown in Table 1 on Page 3. The Facility was constructed in two phases, Phase 1 began operation in 1981 and Phase 2 was completed in 1986. Since operation began, a large amount of data has been collected on the performance of individual treatment components. However, due to non representative sampler locations upstream of the activated sludge system, the performance of the affected processes will be evaluated based on data from September – December of 2007. This data will be used to evaluate the original design capacity and the facility's ability to treat additional flow and loading. The individual unit processes were listed and described in Section 2.0.

The following series of tables show the design, performance and recommended standards of the individual treatment processes. The performance data that fall outside of the recommended standards are highlighted in blue. Process performance upstream of Activated Sludge was analyzed using September to December 2007 data only. The underlined values are the controlling values for unit process capacity. The yellow highlight shows the average flow capacity and the green highlight shows the peak hourly flow capacity.

7.2.1 Brandon Road Pump Station

The original design capacity of the Brandon Road Pump Station was 40.5 mgd. In the recent headworks upgrade project the forcemain was re-routed, increasing the headloss in the forcemain, thereby reducing the peak capacity to approximately 35.0 MGD.

7.2.2 Screens

Rotary fine screens were installed at the WRF in 2007. The manufacturer's rated peak flow capacity of the new screens is 57.9 mgd.

7.2.3 Grit Removal

Table 9 shows the facility design, performance and recommended standards for the grit removal process at the Sioux Falls WRF.

Table 9 - Aerated Grit Chamber Performance Data

Aerated Grit Chamber	Facility Design	Facility Performance	Recommended Standard
Aeration Rate	11 cfm/ft	3.3 – 6.7 cfm/ft	3 – 8 cfm/ft[1][2]
Grit Removal Rate	1.5 – 7.5 ft^3/MGAL	-	0.5 – 27 ft^3/MGAL[1]
Peak Hourly Detention Time	8.0 minutes	7.5 minutes	3 – 5 minutes minimum[2] 3 minutes minimum[3]

[1] Metcalf & Eddy Wastewater Engineering Treatment and Reuse
[2] Great Lakes-Upper Mississippi River Recommended Standards for Wastewater Facilities
[3] South Dakota Department of Environment and Natural Resource Design Criteria

The aerated grit chamber at the Sioux Falls WRF appears to be performing admirably. Based on the table above, the performance of the grit chamber meets or exceeds all recommended performance standards.

The aerated grit chamber standards and calculated capacity are shown in Table 10.

Table 10 Aerated Grit Chamber

Aerated Grit Chamber	Ten State Standards	SD DENR Design Criteria	Capacity
Detention Time (Peak Hourly Flow)	3 – 5 minutes	3 minutes	72 mgd

The recommended aeration rate for aerated grit chambers is between 3 and 8 cubic feet per minute per foot. The controlling detention time under peak hourly flow conditions is 3 minutes per the South Dakota DENR Design Criteria. This equates to approximately 72 mgd capacity. The original design was for 5 minutes detention time, which equates to a flow of 43.2 mgd. The expected grit removal rate is approximately 1 to 4 cubic feet of grit per million gallons of flow. The grit removed at the Sioux Falls WRF is not quantified; however, according to the criteria between 14 and 60 cubic feet of grit per day would be expected.

7.2.4 Primary Clarifiers

The following table shows the primary clarifier design, performance and recommended standards for the Sioux Falls WRF. The flows used below are the applied flows to the primary clarifiers, which consists of influent flow, WAS flow, in-plant waste flow and back wash return.

Table 11 – Primary Clarifier Performance Data

Primary Clarifiers	Facility Design	Facility Performance	Recommended Standard
BOD Removal(3)	33%	62.03%	25-35%(1) 33%(2),(4)
TSS Removal(3)	74%	61.64%	50-65%(1) 55%(4)
TKN Removal(3)		21.78%%	
Side Water Depth	8 ft	8 ft	10 ft minimum(2)
Weir Loading Average Daily Peak Hourly		 15,227 gpd/ft 28,574 gpd/ft	20,000 gpd/ft maximum(4) 30,000 gpd/ft maximum(2)
Surface Overflow Rate Average Daily Peak Hourly	 560 gpd/sf 1,061 gpd/sf	 676.7 gpd/sf 1,270.0 gpd/sf	1,200 gpd/sf(2) 1,500(4)
Detention Time Average Daily Peak Hourly	 2.7 hrs 1.4 hrs	 2.12 hrs 1.13 hrs	 1.5-2.5 hrs(1) -

(1) Metcalf & Eddy Wastewater Engineering Treatment and Reuse
(2) Great Lakes-Upper Mississippi River Recommended Standards for Wastewater Facilities
(3) Percent Removal data based on 3 tanks in service
(4) South Dakota Department of Environment and Natural Resource Design Criteria

Although the side water depth of the primary clarifiers were designed based upon outdated, more shallow depths, and they no longer meet the minimum depth recommendation as outlined in the 10-State Standards, the clarifiers are performing well within the recommended ranges for removal, surface overflow rate (SOR) and detention time. In fact, the BOD removal experienced in the primary clarifiers is well above expected values. However, there is a discrepancy between 10-State Standards and the SD DENR Design Criteria; according to SD DENR criteria the weir loading is exceeded. Taking into account the above removal rates were recorded based upon one unit out of service, the primary clarifiers appear to have excess capacity at this time. Though the current SOR of the primary clarifiers is higher than the original design, the performance of the clarifiers has not suffered and the SOR experienced at the WRF is well within the industry's recommended standards.

The primary clarifier standards and calculated capacity are shown in Table 12.

Table 12 Primary Clarifiers

Primary Clarifiers	Ten State Standards	SD DENR Design Criteria	Capacity
Weir Loading (Peak Hourly Flow)	30,000 gpd/sf	20,000 gpd/ft	33.912 mgd
Surface Overflow (Average)	None	1,000 gpd/sf	25.434 mgd
Surface Overflow (Peak Hourly Flow)	1,200 gpd/sf	1,500 gpd/sf	30.521 mgd
Detention Time (Average)	None	1.4 hours	26.104 mgd
Detention Time (Peak Hourly Flow)	None	0.95 hours	38.469 mgd

The design criteria for weir loading is 20,000 gpd/ft for the SD DENR Design Criteria and 30,000 gpd/sf for the 10 State Standards. The SD DENR Criteria are based on an outdated version of the 10 State Standards. The common current design practice for primary clarifiers of 30,000 gpd/sf will be used for this study. The controlling standard is the surface overflow rate, which calculates to a capacity of 25.434 mgd average flow and 30.521 mgd peak hourly flow.

7.2.5 First Stage Trickling Filters

Table 13Table 13 shows the Sioux Falls WRF First Stage Trickling Filter design, performance and recommended standards comparison.

Table 13 - 1st Stage Trickling Filter Performance Data

First Stage Trickling Filters	Facility Design	Facility Performance	Recommended Standard
Hydraulic loading			
Average Daily	0.163 gpm/sf	0.205 gpm/sf	0.068 - 0.682 gpm/sf(1)
Maximum Daily	0.328 gpm/sf	0.392 gpm/sf	0.159 - 0.638 gpm/sf(2)
Organic Loading	96.6 lb BOD/10^3 cf	67.04 lb BOD/10^3 cf	15-150 lb/10^3 cf(1) 30-100 lb/10^3 cf(2)
BOD Removal Efficiency	55.56%	53.42%	50-90%(1)
NH_3-N Removal Efficiency	None expected	<1 %	None expected(1)

(1) Metcalf & Eddy Wastewater Engineering Treatment and Reuse
(2) South Dakota Department of Environment and Natural Resource Design Criteria

The first stage trickling filters appear to be under loaded organically. The above chart does show they are slightly over loaded hydraulically. They are performing slightly under the original design. All facility performance data is within the recommended standards for the first stage trickling filters. The actual capacity of the trickling filters was examined in previous sections of this report through the process of organic and hydraulic simulations.

The design standards and calculated capacity for the first stage trickling filters is shown in Table 14.

Table 14 First Stage Trickling Filters

First Stage Trickling Filters	Ten State Standards	SD DENR Design Criteria	Capacity
Hydraulic Loading (including recirculation)	None	0.159 – 0.638 gpm/sf	52.575 mgd
Organic Loading	None	30 – 100 lb BOD/ 10^3 cf/day	40,058 lb BOD/d

The SDDENR Criteria lists ranges for hydraulic and organic loading of trickling filters. These Criteria should be used with discretion, since trickling filter performance is highly dependent upon many variables including media type, media configuration, distributor rate and influent waste characteristics. The Criteria is presented for reference; however, more confidence should be given to actual performance data and modeling efforts. The hydraulic loading range for the FSTF is 13.102 – 52.575 mgd and the organic loading range is 12,018 – 40,058 lb BOD/day.

7.2.6 First Stage Intermediate Clarifiers

The following table shows the First Stage Intermediate Clarifier design, performance and recommended standards comparison.

Table 15 – 1st Stage Intermediate Clarifier Performance Data

First Stage Intermediate Clarifiers	Facility Design	Facility Performance	Recommended Standard
Side Water Depth	10 ft	10 ft	12 ft minimum(1)
Surface Overflow Rate			
Average Daily	780 gpd/sf	973.9 gpd/sf	400-700 gpd/sf(2)
Peak Hourly	1,559 gpd/sf	1,866 gpd/sf	1,500 gpd/sf(1) (3)
Detention Time			
Average Daily	2.32 hrs	1.84 hrs	-
Peak Hourly	1.15 hrs	0.96 hrs	-

(1) Great Lakes-Upper Mississippi River Recommended Standards for Wastewater Facilities
(2) Metcalf & Eddy Wastewater Engineering Treatment and Reuse
(3) South Dakota Department of Environment and Natural Resource Design Criteria

The first stage intermediate clarifiers do raise some concern as they were also (similar to the primary clarifiers) designed around outdated, shallower standards. Because the clarifiers are hydraulically loaded beyond their design, the SOR of the clarifiers falls outside the current recommended performance standards. As the hydraulic loading to the first stage intermediate clarifiers expands because of increased population, the performance of the intermediate clarifiers may suffer because of the shallow depth and increased SOR. As a result of the WRF design, there is excess treatment capacity downstream of the first stage intermediate clarifiers and therefore the shallow depth and higher than expected SOR appear to not affect the performance of downstream treatment units at this time.

The standards for intermediate clarifiers and calculated capacity for the Sioux Falls WRF are shown in Table 16.

Table 16 First Stage Intermediate Clarifiers

First Stage Intermediate Clarifiers	Ten State Standards	SD DENR Design Criteria	Capacity
Weir Loading (Peak Hourly Flow)	30,000 gpd/ft	20,000 gpd/ft	32.028 mgd
Surface Overflow (Average)	None	1,000 gpd/sf	17.309 mgd
Surface Overflow (Peak Hourly Flow)	1,500 gpd/sf	1,500 gpd/sf	25.964 mgd
Detention Time (Average)	None	1.8 hours	23.903 mgd
Detention Time (Peak Hourly Flow)	None	1.2 hours	25.895 mgd

The design criteria for weir loading is shown as 20,000 gpd/ft for the SD DENR Design Criteria and 30,000 gpd/sf for the 10 State Standards. The SD DENR Criteria are based on an outdated version of the 10 State Standards. Common current design practice for intermediate clarifiers of 30,000 gpd/sf will be used for this study. The controlling capacity for average flow is 17.309 mgd and for peak hourly flow is 25.895 mgd.

7.2.7 Second Stage Trickling Filters

The following table shows the Second Stage Trickling Filter design, performance and recommended standard comparisons.

Table 17 – Second Stage Trickling Filter Performance Data

Second Stage Trickling Filters	Facility Design	Facility Performance	Recommended Standard
Hydraulic loading Average Daily Maximum Daily	 0.141 gpm/sf 0.284 gpm/sf	 0.169 gpm/sf 0.340 gpm/sf	 0.068 - 0.682 gpm/sf(1) 0.159 - 0.638 gpm/sf(2)
Organic Loading	42 lb BOD/10^3 cf	27.0 lb BOD/10^3 cf	15-150 lb/10^3 cf(1) 30-100 lb/10^3 cf(2)
BOD Removal Efficiency	50%	29.1%	50-90%(1)
NH_3-N Removal Efficiency	None expected	13.2%	None expected(1)
(1) Metcalf & Eddy Wastewater Engineering Treatment and Reuse			
(2) South Dakota Department of Environment and Natural Resource Design Criteria			

The second stage trickling filters at the WRF are under loaded organically. The filters are currently being loaded at approximately 65% their design capacity. On the other hand, they are over their design hydraulic loading. The BOD removal efficiency is less than the original design and likely due to the under loaded condition. The SSTF achieves a small amount of ammonia reduction. This nitrification is attributed to the low level of soluble BOD (≈ 25 mg/l) in the SSTF influent. When soluble BOD is minimized (in under loaded BOD conditions) microorganisms will consume soluble nutrients (i.e, ammonia) because the remaining BOD is not as easily degraded. Based on this analysis, it appears the SSTFs may have available BOD removal capacity.

The standards for trickling filters and calculated capacity for the Sioux Falls WRF are shown in Table 18.

Table 18 Second Stage Trickling Filters

Second Stage Trickling Filters	Ten State Standards	SD DENR Design Criteria	Capacity
Hydraulic Loading (including recirculation)	None	0.159 – 0.638 gpm/sf	60.652 mgd
Organic Loading	None	30 – 100 lb BOD/ 10^3 cf/day	46,213 lb BOD/d

The SD DENR Criteria lists ranges for hydraulic and organic loading of trickling filters. These Criteria should be used with discretion, since trickling filter performance is highly dependent upon many variables including media type, media configuration, distributor rate and influent waste characteristics. The Criteria is presented for reference; however, more confidence should be given to actual performance data and modeling efforts. The hydraulic loading range for the SSTF is 15.116 – 60.652 mgd and the organic loading range is 13,864 – 46,213 lb BOD/day.

7.2.8 Second Stage Intermediate Clarifiers

The following table shows the Second Stage Intermediate Clarifier design, performance and recommended standard comparison.

Table 19 – Second Stage Intermediate Clarifier Performance Data

Second Stage Intermediate Clarifiers	Facility Design	Facility Performance	Recommended Standard
Side Water Depth	10 ft	10 ft	12 ft minimum(1)
Surface Overflow Rate			
Average Daily	780 gpd/sf	930.45 gpd/sf	400-700 gpd/sf(2)
Peak Hourly	1,559.1 gpd/sf	1,866.05 gpd/sf	1,500 gpd/sf(1) (3)
Detention Time			
Average Daily	2.32 hrs	1.93 hrs	-
Peak Hourly	1.15 hrs	0.96 hrs	-

(1) Great Lakes-Upper Mississippi River Recommended Standards for Wastewater Facilities
(2) Metcalf & Eddy Wastewater Engineering Treatment and Reuse
(3) South Dakota Department of Environment and Natural Resource Design Criteria

The second stage intermediate clarifiers, similar to the first stage intermediate clarifiers do raise some hydraulic concerns as they were also designed around outdated, shallower standards. Because the clarifiers are hydraulically loaded beyond their design, the SOR of the clarifiers falls outside the current recommended performance standards. As the hydraulic loading to the second stage intermediate clarifiers expands because of increased population, the performance of the intermediate clarifiers may suffer because of the shallow depth and increased SOR. As a result of the design of the WRF and the lack of organic loading applied to the second stage trickling filters, there is excess treatment capacity downstream of the second stage intermediate clarifiers and therefore the shallow depth and higher than expected SOR appears to not affect the ultimate performance of downstream treatment units at this time.

The standards for intermediate clarifiers and calculated capacity for the Sioux Falls WRF are shown in Table 20.

Table 20 Second Stage Intermediate Clarifiers

Second Stage Intermediate Clarifiers	Ten State Standards	SD DENR Design Criteria	Capacity
Weir Loading (Peak Hourly Flow)	30,000 gpd/ft	20,000 gpd/ft	32.028 mgd
Surface Overflow (Average)	None	1,000 gpd/sf	17.309 mgd
Surface Overflow (Peak Hourly Flow)	1,500 gpd/sf	1,500 gpd/sf	25.964 mgd
Detention Time (Average)	None	1.8 hours (min)	23.903 mgd
Detention Time (Peak Hourly Flow)	None	1.2 hours (min)	25.895 mgd

The design criteria for weir loading is shown as 20,000 gpd/ft for the SD DENR Design Criteria and 30,000 gpd/sf for the 10 State Standards. The SD DENR Criteria are based on an outdated version of the 10 State Standards. Common current design practice for intermediate clarifiers of 30,000 gpd/sf will be used for this study. The controlling capacity for average flow is 17.309 mgd and for peak hourly flow is 25.895 mgd.

7.2.9 Activated Sludge System

Table 21 shows the Activated Sludge design, performance and recommended standard comparison.

Table 21 – Activated Sludge System Performance Data

Activated Sludge	Facility Design	Facility Performance	Recommended Standard
BOD Removal(2)	76.60%	87.89%	
TBOD Concentration	50 mg/l	63.8 mg/l	
SBOD Concentration	NA	11.0 mg/l	≤ 12.0 mg/l(3)
TKN Loading	5,419 lb TKN/d	4,297 lb TKN/d	
CBOD:TKN Ratio	NA	0.87	≤ 1.0(3)
NH^3-N Removal	4,000 lb NH^3-N/d	2,290 lb NH^3-N/d	
NH^3-N Removal(2)		99.73%	

(1) Great Lakes-Upper Mississippi River Recommended Standards for Wastewater Facilities
(2) Performance based on 3 tanks in service
(3) Water Environment Federation

The activated sludge basin was designed for separate stage (tertiary) nitrification at the WRF. Although the influent TBOD concentration to the activated sludge basin is greater than design, the nitrification capacity is unaffected. Because it is difficult to evaluate nitrification efficiency based on influent TBOD concentrations, current design standards look to soluble and carbonaceous organics to evaluate nitrification growth inhibition. To maintain a suitable population of nitrifying organisms in tertiary systems, the Water Environment Federation recommends maintaining an influent SBOD concentration ≤ 12.0 mg/l and a CBOD:TKN ratio ≤ 1.0. The activated sludge system at the Sioux Falls WRF is well within these standards, even while utilizing only half the capacity (only three of the six basins are normally operated). The activated sludge basin is under loaded in terms of TKN, and the ammonia oxidation is greater than 99%. At this time, the activated sludge basin seems to be performing excellently. Currently, there are no operational concerns with the activated sludge basin.

The SD DENR Design Criteria and Ten State Standards suggest limiting values for unit process design and are intended for more conventional wastewater treatment systems. Innovative approaches to treatment such as activated sludge basins for tertiary nitrification are not included in the standards. Therefore, no capacity calculation based on SD DENR and Ten State Standards can be presented.

7.2.10 Final Clarifiers

The following table shows the Final Clarifier design, performance and recommended standard comparison. It should be noted that only three of the four final clarifiers were in operation during this evaluation.

Table 22 – Final Clarifier Performance Data

Final Clarifier	Facility Design	Facility Performance	Recommended Standard
Side Water Depth	14 ft	14 ft	12 ft minimum(1)
Solids Loading Rate			
Average Daily	14.2 lb/sf/d	10.7 lb/sf/day	-
Peak Hourly		15.2 lb/sf/day	35 lb/sf/day(1) 25 lb/sf/day(3)
Surface Overflow Rate			
Average Daily	427 gpd/sf	525.8 gpd/sf	400-700 gpd/sf(2)
Peak Hourly	860 gpd/sf	998.8 gpd/sf	800/1000 gpd/sf(1)(3)
Detention Time			
Average Daily	5.89 hrs	4.78 hrs	-
Peak Hourly	2.92 hrs	2.52 hrs	-

(1) Great Lakes-Upper Mississippi River Recommended Standards for Wastewater Facilities
(2) Metcalf & Eddy Wastewater Engineering Treatment and Reuse
(3) South Dakota Department of Environment and Natural Resource Design Criteria

The original facility design of the final clarifiers was based on achieving a 20 mg/l TSS effluent concentration before entering the effluent filters. The purpose of the 20 mg/l limitation was to ensure a low effluent TSS and to reduce the risk of ammonification (organic nitrogen converting to ammonia nitrogen) prior to discharge. As shown in the previous table, the facility performance does not meet the recommended standards for final clarifiers. In addition, the facility performance criteria of surface overflow rate and detention time exceed the original facility design conditions. The original design intent of preventing effluent ammonia permit violations needs to be accounted for in the evaluation of the final clarifiers in conjunction with published final clarifier recommended standards. This will be further examined in the final section of this report.

The standards for final clarifiers and calculated capacity for the Sioux Falls WRF are shown in Table 23.

Table 23 Final Clarifiers

Final Clarifiers	Ten State Standards	SD DENR Design Criteria	Capacity
Weir Loading (Peak Hourly Flow)	30,000 gpd/ft	15,000 gpd/ft	28.3 mgd
Surface Overflow (Peak Hourly Flow)	800/1000 gpd/sf	800/1000 gpd/sf	25.1/31.4 mgd
Peak Solids Loading Rate	35 lb/day/sf	25 lb/day/sf	785,400 lb/d
Detention Time (Peak Hourly Flow)	None	2.7 hours	29.2 mgd

The design criteria for final clarifiers following a 2 stage nitrification process in an activated sludge basin is 800 gallons per day per square foot, which equates to 25.133 mgd at peak hourly flow. However, the City of Sioux Falls has operational conditions similar to a single stage nitrification process, which correlates to a surface overflow rate of 1,000 gpd/sf and a peak hourly flow of 31.4 mgd. Further analysis of the Final Clarifier performance will be discussed in the final section of this report.

7.2.11 Effluent Filters

The following table shows the Effluent Sand Filter design, performance and recommended standard comparisons.

Table 24 – Effluent Sand Filter Performance Data

Effluent Sand Filters	Facility Design	Facility Performance	Recommended Standard
Hydraulic Loading Average Daily Maximum Daily	2.01 gpm/sf 4.05 gpm/sf	2.24 gpm/sf 4.40 gpm/sf	1.96-9.82 gpm/sf[1] 1.96-9.82 gpm/sf[1] 5.0 gpm/sf [3]
TSS Average Influent	20 mg/l	2.5 mg/l	-
TSS Removal[2]	53.72%	27.3%	-

[1] Metcalf & Eddy Wastewater Engineering Treatment and Reuse
[2] Performance based on 8 tanks in service
[3] Great Lakes-Upper Mississippi River Recommended Standards for Wastewater Facilities

The effluent sand filters are slightly over loaded hydraulically compared to the original facility design; however, they are well within industry design standards. The TSS in the influent is below the design concentration and the removal efficiency is much higher than design. Although the filters are currently performing satisfactorily, at higher influent flow rates, the effluent sand filters may present a hydraulic bottle neck in the WRF.

The standard for effluent filters and calculated capacity for the Sioux Falls WRF are shown in Table 25.

Table 25 Effluent Filters

Effluent Filters	Ten State Standards	SD DENR Design Criteria	Capacity
Hydraulic Loading (Peak Hourly Flow)	5 gpm/sf	None	33.293 mgd

The controlling standard for effluent filters is a hydraulic loading of 5 gallons per minute per square foot, which equates to a capacity of 33.293 mgd at peak hourly flow.

7.2.12 Chlorine Contact Basin

The following table shows the Chlorine Contact Basin design, performance and recommended standard comparisons.

Table 26 – Chlorine Contact Basin Performance Data

Chlorine Contact Basin	Facility Design	Facility Performance	Recommended Standard
Hydraulic Loading			
Average Daily	13.4 MGD	14.9 MGD	-
Maximum Daily	26.9 MGD	29.3 MGD	-
Average Daily Contact Time	> 30 minutes	27.3 minutes	30 – 120 minutes(1)
Peak Hourly Contact Time	> 15 minutes	13.9 minutes	15 minutes minimum(2)

(1) Metcalf & Eddy Wastewater Engineering Treatment and Reuse
(2) Great Lakes-Upper Mississippi River Recommended Standards for Wastewater Facilities

The chlorine contact basin is hydraulically over loaded. At current WRF influent flow rates, both average daily and peak hourly, the chlorine contact basin falls outside the industry's recommended standards for contact time to ensure proper disinfection. The City injects chlorine upstream of the effluent filters to increase contact time to meet disinfection standards.

The standard for chlorine contact tanks and calculated capacity for the Sioux Falls WRF are shown in Table 27.

Table 27 Chlorine Contact Basin

Chlorine Contact Basin	Ten State Standards	SD DENR Design Criteria	Capacity
Contact Time (Peak Hourly)	15 minutes minimum	None	27 mgd

The recommended standard is 15 minutes minimum contact time for disinfection, which equates to 27 mgd at peak hourly flows. However, the Sioux Falls WRF injects chlorine upstream of the effluent filters, which increases the contact time for disinfection. The additional volume in the effluent filters used in conjunction with the chlorine contact basin calculates to a peak hourly flow capacity of approximately 64 mgd.

8.0 HYDRAULIC UNIT PROCESS SUMMARY

Section 5.0 discussed the hydraulic model development and calibration. Process capacities were determined based on the limitations predicted by the calibrated model. Section 7.0 presented the regulatory standards for each unit process and associated calculated capacities. This section summarizes the hydraulic capacities from Sections 5.0 and 7.0 and presents the controlling process capacity of each unit.

8.1 Brandon Road Pump Station

The original design capacity of the Brandon Road Pump Station was 40.5 mgd. In the recent headworks upgrade project the forcemain was re-routed, increasing the headloss in the forcemain, thereby reducing the peak capacity to approximately 35.0 MGD.

8.2 Screens

Rotary fine screens were installed at the WRF in 2007. The manufacturer's rated peak flow capacity of the new screens is 57.9 mgd.

8.3 Grit Removal

The constraints and capacity of the aerated grit removal system are shown in Table 28.

Table 28 Aerated Grit Chamber

Aerated Grit Chamber	Hydraulic Model	Regulatory
Peak hourly flow	38 mgd	72 mgd

The capacity of the aerated grit chamber, according to the SD DENR standards is 72 mgd peak hourly flow. A hydraulic constraint occurs in the aerated grit chamber, according to the hydraulic model, at approximately 38 mgd. At 38 mgd, the effluent weir of the aerated grit chamber begins to flood. However, this is likely due to restrictions in the trickling filter distributor arms, not a limitation of the aerated grit process.

8.4 Primary Clarifiers

The constraints and capacity of the primary clarifiers are shown in Table 29.

Table 29 Primary Clarifiers

Primary Clarifiers	Hydraulic Model	Regulatory
Average Flow	30.0 mgd	25.4 mgd
Peak Hourly Flow	N/A	30.5 mgd

The controlling average flow capacity of the primary clarifiers is 25.4 mgd. The controlling peak hourly flow capacity of the primary clarifiers is 30.5 mgd. A hydraulic deficiency occurs in the primary clarifiers, according to the hydraulic model. At 30.0 mgd, the weirs of the primary clarifiers begin to flood. The deficiency is directly related to the surcharging experienced in splitter manhole No. 4 and not a limitation of the primary clarifier.

8.5 First Stage Trickling Filters

The constraints and capacity of the first stage trickling filters are shown in Table 30.

Table 30 First Stage Trickling Filters

First Stage Trickling Filters	Hydraulic Model	Regulatory
Hydraulic	27.0 mgd	52.6 mgd
Organic	N/A	40,058 lb BOD/d

The SD DENR Design Criteria lists wide ranges for hydraulic and organic load capacity of trickling filters. As previously mentioned, these ranges will be included for reference. The process capacity should be based on site-specific performance data and modeling of existing facility. The hydraulic capacity according to standards is 52.6 mgd; the organic loading capacity according to standards is 40,058 pounds of BOD per day. A hydraulic constraint occurs in the first stage trickling filters at approximately 27.0 mgd according to the hydraulic model. Based on field testing conducted in October 2007, corrosion in the distributor arms is suspected as the cause for the hydraulic constraint. At 27.0 mgd, flow back-up occurs in the upstream splitter manhole 4. The manufacturer's rated flow capacity of the distributor arms is 42.0 mgd.

8.6 First Stage Intermediate Clarifiers

The constraints and capacity of the first stage intermediate clarifiers are shown in Table 31.

Table 31 First Stage Intermediate Clarifiers

First Stage Intermediate Clarifiers	Hydraulic Model	Regulatory
Average Flow	46.0 mgd	17.3 mgd
Peak Hourly Flow	N/A	25.9 mgd

The limiting average flow rate for the first stage intermediate clarifiers according to the SD DENR Criteria is 17.3 mgd. The limiting peak hourly flow rate for the intermediate clarifiers is 25.9 mgd. At 46.0 mgd, a hydraulic deficiency occurs in the first stage intermediate clarifiers, according to the hydraulic model. At 46.0 mgd the weirs begin to submerge in the clarifiers.

8.7 Second Stage Trickling Filters

The constraints and capacity of the second stage trickling filters are shown in Table 32.

Table 32 Second Stage Trickling FIlters

Second Stage Trickling Filters	Hydraulic Model	Regulatory
Hydraulic	30-35 mgd	60.6 mgd
Organic	N/A	46,213 lb BOD/d

The SD DENR Design Criteria lists wide ranges for hydraulic and organic load capacity of trickling filters. As previously mentioned, these ranges will be included for reference. The process capacity should be based on site-specific performance data and modeling of existing facility. The hydraulic capacity limit is 60.6 mgd and the organic loading limit is 46,213 pounds of BOD per day according to standards. The initial hydraulic model did not detect any flow constraints in the second stage trickling filters up to the peak flow simulated, 48.6 mgd. The manufacturer's original rated capacity of the distributor arms was 46.0 mgd. However, additional field testing at flow rates greater than 28.0 mgd, it appears the SSTF distributor arms may be a hydraulic constraint. In their present condition, it is doubtful whether the SSTF distributor arms can convey the manufacturer's rated flow capacity of 46.0 mgd without causing weir submergence in Splitter Manhole No. 6.

8.8 Second Stage Intermediate Clarifiers

The constraints and capacity of the second stage intermediate clarifiers are shown in Table 33.

Table 33 Second Stage Intermediate Clarifiers

Second Stage Intermediate Clarifiers	Hydraulic Model	Regulatory
Average Flow	26.5 mgd	17.3 mgd
Peak Hourly Flow	N/A	25.9 mgd

The limiting average flow rate for the second stage intermediate clarifiers according to the SD DENR Criteria is 17.309 mgd. The limiting peak hourly flow rate for the intermediate clarifiers is 25.895 mgd. At 26.5 mgd, a flow constraint occurs according to the hydraulic model. At 26.5 mgd the weirs in the second stage intermediate clarifiers begin to submerge. The cause of weir submergence is from backup occurring in the process pump station, due to pumping capacity limitations, and unlikely due to the design of the clarifiers. Flooding of the second stage intermediate clarifiers at 27.125 mgd is illustrated in Figure 31. Notice the weirs are completely submerged.

Figure 31 Second Stage Intermediate Clarifiers at 27.125 mgd

8.9 Activated Sludge Basins

The SD DENR Design Criteria and Ten State Standards are intended for more conventional wastewater treatment systems. Innovative approaches to treatment such as activated sludge basins for tertiary nitrification are not included in the standards. The hydraulic model did not show any constraints in the activated sludge system up to the simulated flow of 48.6 mgd.

8.10 Final Clarifiers

The constraints and capacity of the final clarifiers are shown in Table 34.

Table 34 Final Clarifiers

Final Clarifiers	Hydraulic Model	Regulatory
Peak Hourly Flow	> 48.6 mgd	31.4 mgd
Peak Solids Loading Rate	N/A	785,400 lb/d

The limiting peak hourly flow rate for the final clarifiers according to the 10 States Standards is 31.4 mgd based on a surface overflow rate of 1,000 gpd/sf. The peak solids loading rate to the final clarifers is 785,400 lb/d. The hydraulic model did not detect any flow constraints in the final clarifiers up to the peak flow simulated, 48.6 mgd.

8.11 Effluent Filters

The constraints and capacity of the effluent filters are shown in Table 35.

Table 35 Effluent Filters

Effluent Filters	Hydraulic Model	Regulatory
Average Flow	22.0 mgd	N/A
Peak Hourly Flow	N/A	33.3 mgd

The peak hourly flow rate to the effluent filters according to the 10 State Standards is 33.3 mgd. At 22.0 mgd a hydraulic constraint was indicated by the hydraulic model. At approximately 22.0 mgd, bypass to the chlorine contact basin occurs.

8.12 Chlorine Contact Tank

The constraints and capacity of the chlorine contact basin are shown in Table 36.

Table 36 Chlorine Contact Basin

Chlorine Contact Basin	Hydraulic Model	Regulatory
Average Flow	28.0 mgd	N/A
Peak Hourly Flow	N/A	27.0 mgd

The peak hourly flow rate to the effluent filters according to the 10 State Standards is 27.0 mgd. However, the Sioux Falls WRF injects chlorine upstream of the effluent filters, which increases the contact time for disinfection. The additional volume in the effluent filters used in conjunction with the chlorine contact basin calculates to a peak hourly flow capacity of approximately 64 mgd. At 28.0 mgd a hydraulic constraint was indicated by the hydraulic model. At 28.0 mgd, the effluent flume begins to submerge. Submerged flume conditions require upstream and downstream level measurements to accurately calculate flow, which was not included in the original facility design. As evident by Figure 32, at 27.125 mgd the effluent flume is nearly submerged.

Figure 32 Effluent Flume at 27.125 mgd

8.13 Cascade Aeration

The hydraulic model did not indicate a hydraulic constraint at the cascade aeration structure up to the simulated flow of 48.6 mgd. There are no regulatory criteria to evaluate capacity of the effluent aeration system.

9.0 RECOMMENDED FORMAL CAPACITY OF WRF

The capacity available for each unit process is shown in Table 37 below. The yellow highlights correspond to average daily capacities; the green highlights correspond to peak hourly flow capacities; the orange highlights correspond to equipment limitations and the white rows correspond to hydraulic model and field testing.

Table 37 Unit Process Constraints

Unit Process	Capacity/Constraint	Flow
FS Intermediate Clarifiers	Average	17.31 mgd
SS Intermediate Clarifiers	Average	17.31 mgd
Effluent Filters	Bypass begins to Cl_2 Contact	22.00 mgd
Primary Clarifiers	Average	25.43 mgd
FS Intermediate Clarifiers	Peak Hourly	25.90 mgd
SS Intermediate Clarifiers	Peak Hourly	25.90 mgd
Second Stage IC	Weirs submerged	26.50 mgd
MH 11	Surcharging begins	26.50 mgd
SP MH 4	Surcharging begins	27.00 mgd
FS Trickling Filter	Distributor Capacity	27.00 mgd
SS Trickling Filter	Distributor Capacity	28.00 mgd
Chlorine Contact	Effluent flume surcharges	28.00 mgd
Process Pump Station	Pumping Capacity	30.00 mgd
Primary Clarifiers	Weirs submerged	30.00 mgd
Primary Clarifiers	Peak Hourly	30.52 mgd
SP MH 7	Weirs submerged	31.00 mgd
SP MH 3	Weirs submerged	31.00 mgd
Final Clarifiers	Peak Hourly	31.40 mgd
Effluent Filters	Peak Hourly	33.29 mgd
Brandon Road Pump Station	Pumping Capacity	35.00 mgd
SP MH 4	Structure freeboard limited	35.00 mgd
SP MH 1	Weirs submerged	36.00 mgd
Primary Clarifiers	Structure freeboard limited	37.00 mgd
Aerated Grit Removal	Effluent weir submerged	38.00 mgd
Chlorine Contact	Structure freeboard limited	41.00 mgd
Effluent Filters	Effluent weir submerged	41.00 mgd
First Stage IC	Weirs submerged	46.00 mgd
SP MH 5	Weirs submerged	48.00 mgd

9.1 Average Daily Flow Capacity

The recommended average daily flow capacity for the Sioux Falls WRF is 21.0 mgd. This recommendation is based on:

- Hydraulic model simulations indicating effluent filter by-pass flow occurs at approximately 22.0 mgd.
- Field observations at varying flows indicating effluent filter by-pass conditions at high flows.
- No hydraulic constraints were simulated or observed during field testing below 22.0 mgd.

Table 37 shows the recommended average re-rate capacity of 21.0 mgd exceeds the regulatory capacity of 17.31 mgd for the first and second stage intermediate clarifiers. However, the treatment efficiency of the intermediate clarifiers is not critical to achieve effluent permit requirements. Average capacity is not typically the design parameter for clarifiers. In fact, the 2004 edition of the Ten State Standards does not include average recommended design values. There are numerous downstream processes that ensure limits are not exceeded. Historical influent flows and effluent quality are shown in Figure 33.

Figure 33 Influent Flow vs. Effluent TSS
(May 24-June 25, 2004, March – July 2006)

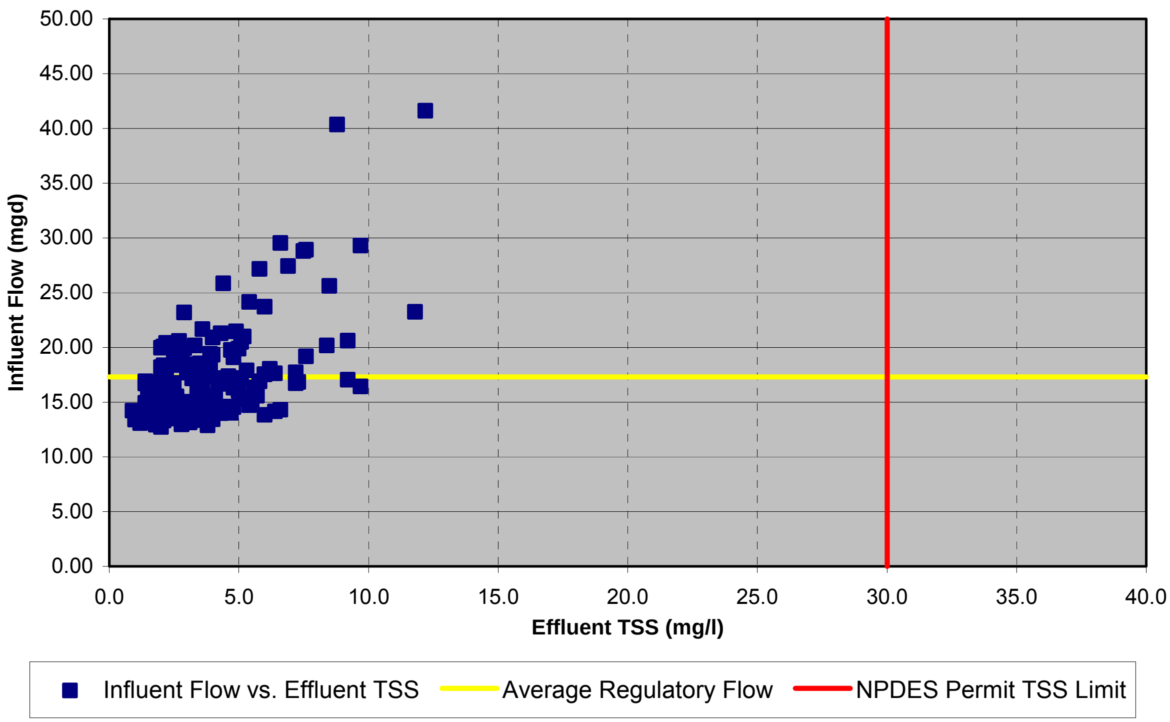


Figure 33 shows average daily flows from a high influent flow period in 2004 and March through July of 2006. At the highest daily influent flow of 41.6 mgd, the effluent TSS was about 12 mg/l. This is much lower than the NPDES Permit limit of 30 mg/l, which is shown on the figure as the vertical red line. Historical data shown in Figure 33 indicates no variation in effluent quality at flows of 21.0 mgd. Therefore, due to the limited use of average design values for clarifiers, the number of downstream processes and historical data shown on Figure 33, exceeding the regulatory average daily flow capacity of the intermediate clarifiers to a capacity of 21.0 mgd will not impact the final effluent quality.

9.2 Peak Hourly Flow Capacity

The recommended peak hourly flow capacity for the Sioux Falls WRF is 35.0 mgd. This recommendation is based on:

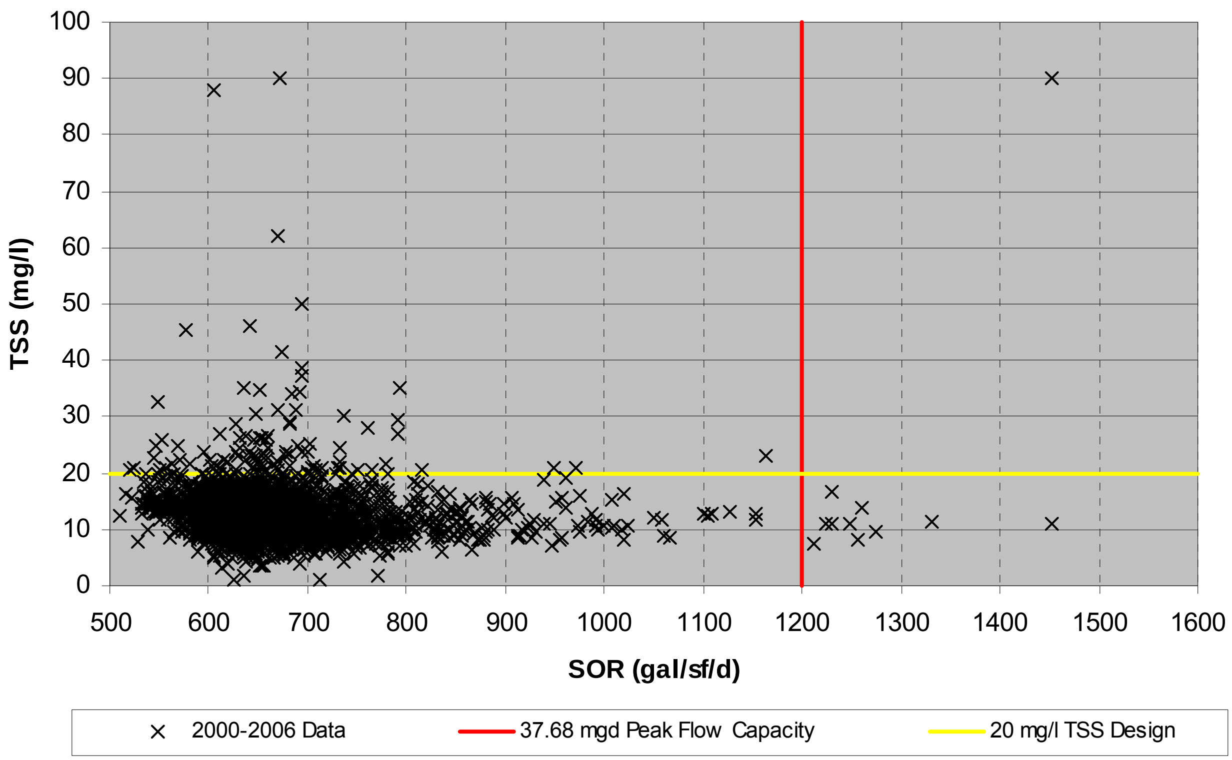
- Hydraulic model simulations indicating freeboard limitations at 35.0 mgd, however
- Process pump station limits flow to the tertiary and final treatment process to 26.0 - 30.0 mgd,

➢ And final clarifiers' surface overflow rates.

Table 37 shows the regulatory capacities for the Final Clarifiers (31.4 mgd) and the First and Second Stage Intermediate Clarifiers (25.90 mgd) are exceeded at 35.0 mgd peak hourly flow. The regulatory capacity listed in the SD DENR criteria and the 10 States Standards is based on conventional treatment processes as a general recommendation for design. However, the Sioux Falls WRF is not a conventional treatment plant. Non-conventional facilities require more emphasis on pilot testing or actual facility performance data to determine facility capacity.

The recommended peak hourly re-rate capacity of 35.0 mgd exceeds the regulatory capacity of 31.4 mgd for the final clarifiers. The peak hourly regulatory capacity is based on a surface overflow rate of 1,000 gpd for final clarifiers following nitrification. Figure 34 shows the surface overflow rate from 2000-2006 at the WRF and the surface overflow rate corresponding to surface overflow rate of 1,200 gpd/sf and a flow of 37.68 mgd.

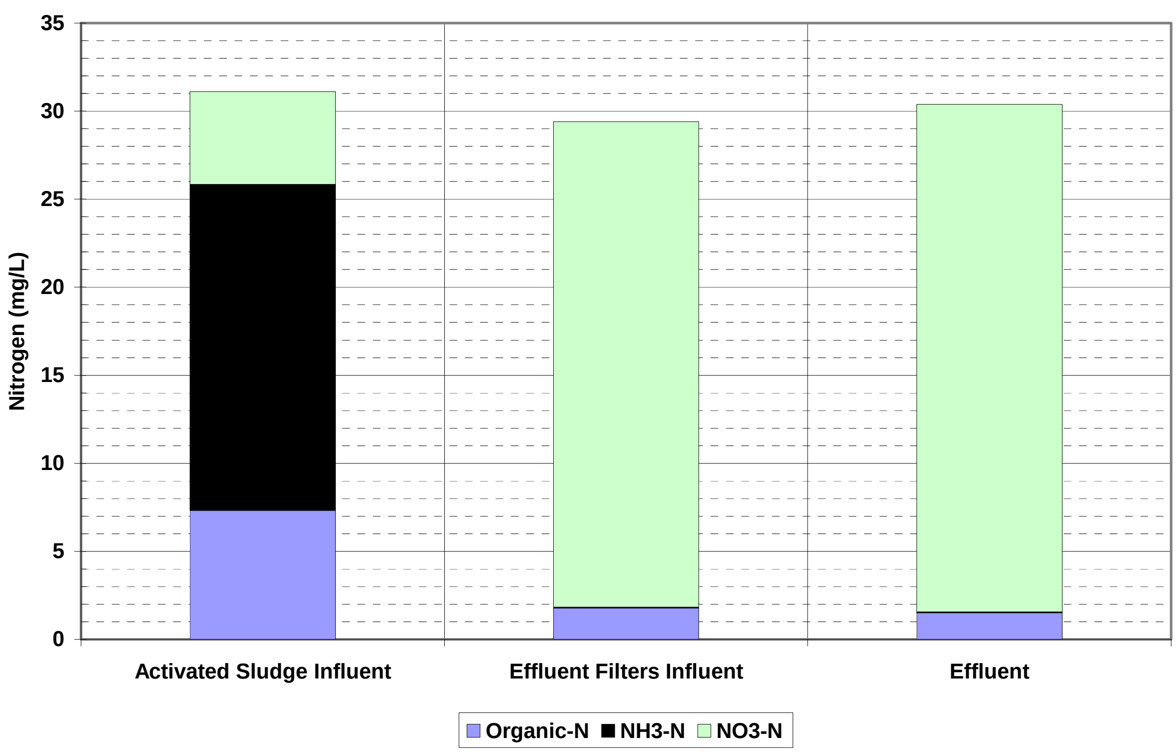
Figure 34 SOR vs. Final Clarifier TSS Effluent



The historical data shows no correlation between SOR and TSS effluent achieved. The SOR at the recommended peak hourly flow of 37.68 mgd is equivalent to 1,200 gal/day/sf, as illustrated by the vertical red line. The original influent design to the effluent filters was 20 mg/l (the horizontal yellow line). Based on the historical data presented in Figure 34, at a SOR less than 1,200 the effluent TSS concentration was below 20 mg/l the majority of the time. During the limited times when the final clarifier TSS effluent concentration exceeds 20 mg/l, the effluent filters will still adequately remove TSS; however, they may need to be backwashed more often. The effluent TSS permit limit at the WRF is 30 mg/l, which can reliably be met at 35.0 mgd.

Another consideration in the design of the final clarifiers and effluent filters is the ammonia-nitrogen limit. Concern was given to the possibility of converting organic nitrogen from the final clarifier effluent to ammonia-nitrogen before discharge. Based on sampling data to date, it appears there is not enough organic nitrogen available following the effluent filters to cause effluent ammonia violations due to ammonification. This is illustrated by a nitrogen balance from the Activated Sludge influent to the Plant effluent in Figure 35.

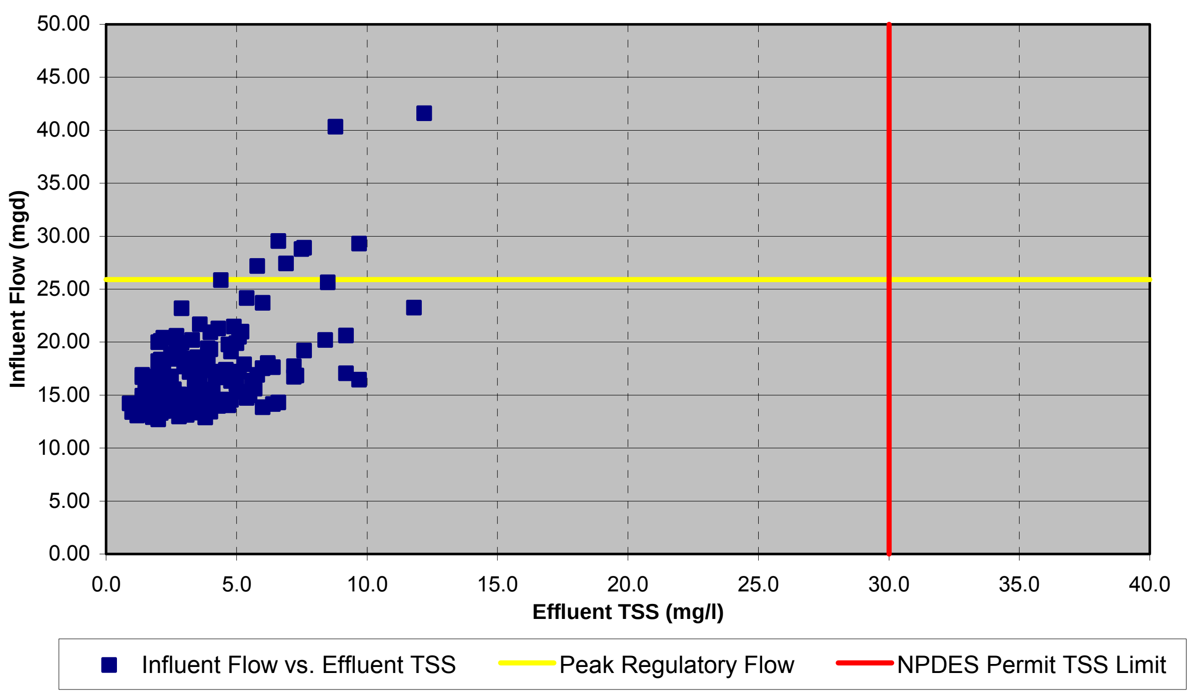
Figure 35 Nitrogen Balance – Activated Sludge to Plant Effluent



The data in Figure 35 shows the activated sludge basin oxidizes ammonia very well. Seeing as the basin also converts a significant amount of organic nitrogen, TKN should continue to be used as the parameter for system control. Figure 35 also reveals the amount of organic nitrogen and NH_3-N in the plant effluent is approximately 1.5 mg/l and 0.1 mg/l, respectively. Even if the entire amount of effluent organic nitrogen was converted to ammonia (which is not possible), the facility would meet the effluent ammonia limit as the sum of NH_3-N and organic nitrogen is less than 2.0 mg/L. Therefore, the ammonia-nitrogen limit will not be compromised due to the ammonification of organic nitrogen and is not a concern in the design of the effluent filters.

Table 37 shows the recommended peak hourly flow re-rate capacity of 35.0 mgd exceeds the regulatory capacity of 25.90 mgd for the first and second stage intermediate clarifiers. However, historical data shows the treatment efficiency of the intermediate clarifiers is not critical to achieve effluent permit requirements. There are numerous downstream processes that ensure limits are not exceeded. Figure 36 shows historical performance data for influent flows in May and June of 2004 (high flow period) and March – July of 2006.

Figure 36 Average Daily Influent Flow vs. Effluent TSS
May - June 2004, March – July 2006



The average daily influent flows are shown in Figure 36. Actual peak hourly flows were higher – up to 48.60 mgd in June of 2004. Peak hourly flow data is not kept by plant personnel; therefore average daily flows are shown. Even at average daily flows above 35.0 mgd (peak hourly flows were actually higher), effluent TSS values were well under the permitted limit of 30.0 mg/l. Due to the number of downstream processes and historical data shown on Figure 36, exceeding the regulatory peak hourly flow capacity of the intermediate clarifiers to a capacity of 35.0 mgd will not impact the final effluent quality.

9.3 Loading Capacity

A number of organic models were evaluated to select an appropriate predictor of TBOD removal in the first stage trickling filters. The models evaluated include Velz, Modified Velz, *Modified Velz, NRC, Eckenfelder, and Regression of plant data. While the Velz model offered a good fit to the data, the regressed equation provided a better fit and presented a more conservative predictor for removal efficiency, when compared to the Velz equation. Therefore, the regressed equation is the recommended model for simulating BOD removal across the FSTF process at the Sioux Falls WRF.

Based on the performance data gathered from September through December of 2007 and the recommended Organic Model, the allowable headworks loadings for TBOD, TSS and TKN were determined in Section 6.0 of this Report. Table 38 shows the recommended allowable headworks loadings for TBOD, TSS and TKN.

Table 38 Recommended Allowable Headworks TBOD, TSS and TKN Loadings

Parameter	Loading (lb/d)	Concentration at 21 mgd (mg/l)
TBOD	51,240	293
TSS	43,900	251
TKN	9,440	53.9