FACILITY PLAN

for

WASTEWATER TREATMENT FACILITIES

INDIANOLA, IOWA

April 2016

(Revised April 2018)

CITY OF INDIANOLA EST. 1849

HRG

HRGreen

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1. EXECUTIVE SUMMARY

1.1. SCOPE AND BACKGROUND

This Facility Plan is required by the Iowa Department of Natural Resources (IDNR) as the official document to evaluate and recommend improvements to Indianola's wastewater treatment system infrastructure. The report projects the wastewater produced by the City's residential, commercial and industrial wastewater contributors and presents a wastewater treatment plan to meet the treatment needs and environmental protection for the 20 year planning period and beyond.

The City's North Wastewater Treatment Facility (NWWTF) has served the community since the 1970s. The NWWTF was designed to support a population of 11,000. A couple of rounds of modifications in the 1990s and early 2000s expanded the wastewater treatment plant's capacity to meet the City's needs, however; the current condition of the treatment plant is poor. The plant is currently unable to treat the original NWWTF's design flow due to failed equipment, one of the main original process units is near collapse, and there are numerous other treatment processes units beyond their useful life.

The wastewater collection system (sanitary sewers, lift stations and force mains) in Indianola has recently undergone major improvements to repair and replace approximately one fourth of the sanitary sewer conveyance system. Although these improvements were necessary to reduce Sanitary Sewer Overflows (SSOs), there continues to be a significant volume of clean water entering the sanitary sewer system. Most communities have a 5 to 1 ratio of peak (hourly) flows to average wastewater flows that reach the wastewater treatment plant. Indianola's ratio of peak wastewater flows to average wastewater flows is around 8 to 1. It will take years of public education, City ordinance enforcement, systematic sewer inspection and repairs and construction projects to get the sanitary sewer collection system closer to a more typical peak hourly to average flow ratio.

In 2014 a Siting Study was completed to evaluate and recommend modifications to the existing wastewater treatment versus build new wastewater treatment facilities at a new site. The study concluded to build a new wastewater treatment facility at the Farm Site. The Farm Site includes approximately 360 acres of property about 1.5 miles north and west of the existing NWWTF. In addition to the condition of the existing NWWTF there are many drivers for a new WWTP at the Farm Site. The most significant drivers are explained below:

- **Replacement of the existing NWWTF**. The existing wastewater treatment plant needs major modifications to make it a reliable plant at the current and future flows. Making a major investment to upgrade the plant still leaves the City relying on some old infrastructure that will need additional investment in ten years or so.
- **The Iowa Nutrient Strategy applies to Indianola**. The State has adopted the Iowa Nutrient Strategy which will require Grade IV WWTPs to meet more stringent effluent requirements for Total Nitrogen and Phosphorus removal. The existing NWWTF would need major modifications to meet these requirements. A new WWTP could be

much more efficient to meet the requirements as well as additional future requirements.

- **Treatment capacity for growth**. For years the City has lacked wastewater treatment capacity for growth of the community as well as economic development. A new WWTP would have some capacity for industrial contributors. The City's Economic Development group could actively market businesses and industries that would be beneficial to the City of Indianola.
- **Treating Peak Wastewater Flows**. Most of the current wastewater treatment problems in Indianola relate to not being able to handle the high flows that correspond to a peak event. As wastewater treatment moves towards higher levels of treatment to meet more stringent nutrient removal requirements, new concepts for *wet weather side stream* treatment will be important to process those dilute flows quickly so as not to upset the nutrient removal portions of the treatment process.
- **Encroachment on the existing NWWTF site**. The existing NWWTF on Hoover Street is a relatively small footprint with potential for homes on the east and north. In addition, there is planning for further development of Hoover Street as an arterial which would open the area for further development. The existing NWWTF site will definitely receive more scrutiny and more provisions to eliminate odors will need to be added in the future. The site separation is much better at the Farm Site and because the City owns much more land this will not be a problem in the future.

1.2. EVALUATIONS

The Facility Plan was developed based on the requirements of the IDNR Design Standards. The existing loads and flows were reviewed and the design flows and loads were established for the future residential projected population and an allotment for industrial growth. A Waste Load Allocation (WLA) was developed for the Middle River as a proposed receiving stream adjacent to the Farm Site. The WLA along with the Iowa Nutrient Strategy was used to evaluate wastewater treatment technologies considered in this report. A condition evaluation was completed for the collection system and the existing NWWTF. The Hydraulic Study completed in 2014 covers a detailed summary of the sanitary sewer collection system.

Two preliminary treatment options were developed for further evaluation. One preliminary treatment alternative continued to use some of the preliminary treatment processes at the existing NWWTF and then convey the flows to the Farm Site for some additional preliminary treatment followed by secondary treatment. The second alternative for preliminary treatment eliminated all the existing processes at the NWWTF and provided all the preliminary and further wastewater treatment at the Farm Site.

Three secondary treatment alternatives were reviewed to treat up to average wet weather flows at the Farm Site. A Process Workshop was used to present and provide an understanding of the potential secondary treatment options. The selected secondary treatment process was a two stage oxidation ditch followed by chemical phosphorus removal. The oxidation ditch process will remove BOD, total solids, ammonia and total nitrogen ahead of the phosphorus removal. The three secondary treatment alternatives evaluated were: activated sludge, oxidation ditch process, and a sequencing batch reactor (SBR). Each of these secondary treatment process alternatives are reliable and flexible alternatives. Ultraviolet (UV) disinfection was planned to follow each secondary treatment alternative.

Aerobic digestion was the solids treatment process selected at the Process Workshop and evaluated. Two alternatives of aerobic digestion and biosolids storage were evaluated.

The project schedule has been planned to best align with the City's funding of the project. The City is aggressively paying down debt from the recent collection system projects to make debt room for a major wastewater treatment project. The project is planned to start construction of the proposed wastewater treatment plant at the Farm Site in spring of 2020. The biggest challenge for a deferred start of the project will be to keep the existing NWWTF in reliable operation for the next several years without huge replacement costs.

1.3. RECOMMENDATIONS

The recommended wastewater treatment facility for the City of Indianola is covered in detail in Chapter 12 of this Facility Plan. The treatment plant processes selected for the City in this report result in a flexible, reliable, easily operating wastewater treatment system that will meet the required nutrient removal strategy for the next 20 years and the foreseeable future. The selected treatment process includes an established technology known for its ease of operation for the secondary treatment system and an innovative economical *wet weather side stream* treatment process to help the plant meet the discharge permit and eliminate sanitary sewer overflows (SSOs) in the community.

The opinion of probable construction cost for the proposed wastewater treatment plant improvements at the Farm Site is $31,723,000.

1.4. REVISIONS TO THE FACILITY PLAN

HR Green and the City of Indianola have had several rounds of discussion and technical meetings with the IDNR since the original issue of this Facility Plan. The meetings have been intended for enhancing the IDNR's understanding of the project and design of nutrient removal facilities. Several key elements/issues originally presented in this report have been changed/modified or need additional clarification through those discussions with the IDNR. The original report has been revised to add or further clarify significant additional detail. The paragraphs shown in italics font show the information that has been revised or added. Generally these revisions include:

- *Modifications based on new Waste Load Allocation (WLA) for the Middle River by the IDNR*
- *Revised design wastewater Flows and Loads (approved by IDNR)*
- *Additional detail including a summary of Biowin model information for expected design operating conditions*
- *Discussion of Store and Treat vs. Wet Weather Split Flow Treatment*

- *Additional detail regarding technical alternatives for Wet Weather Split Flow Treatment*

At the time of the revised Facility Plan (March 2018) the following items supporting this revised Facility Plan have been submitted to the IDNR:

- *Revised Wastewater Flows and Loads (approved by IDNR)*
- *Letter to IDNR discussing potential variances needed for the proposed improvements*
- *Completed Antidegradation Alternatives Analysis for Indianola Wastewater Treatment Plant (submitted March 2018)*

2. INTRODUCTION

2.1. BACKGROUND

The City of Indianola has provided the community with appropriate wastewater conveyance and wastewater treatment infrastructure to serve the community to meet the requirements of Iowa Department of Natural Resources (IDNR) and to protect the local environment. As the wastewater treatment facilities are nearing the end of their useful life, significant planning is necessary to continue to meet this commitment.

The City's North Wastewater Treatment Facility (NWWTF) has served the City well but is also near the end of its life. The facility treats the residential, commercial and industrial wastewater flows that are collected and conveyed through the City's sanitary sewer collection system. The existing NWWTF is not suitable for the additional nutrient removal requirements currently proposed by the IDNR.

The City of Indianola purchased approximately 360 acres approximately one-half mile west and one mile north of the existing North Wastewater Treatment Facility. The new property (Farm Site) was proposed to be the home for the future wastewater treatment plant. HR Green completed a Siting Study in 2014 to evaluate the options of 1) Upgrade the existing wastewater treatment plant at the existing facility, 2) Abandon the existing treatment plant and construct a new wastewater treatment plant at the Farm Site, or 3) Upgrade part of the existing wastewater plant at the existing site and construct the back half of the treatment system at the Farm Site. Through this study the recommended plan for wastewater treatment plant improvements was agreed to construct new wastewater treatment facilities at the Farm Site.

The existing collection system consists of approximately 83 miles of sanitary sewer, 1,560 manholes, 10 lift stations, and two equalization basins. Since 2008 the City has been working to improve the collection system and eliminate sanitary sewer overflows (SSOs). Four phases of collection system repair and lining projects have been recently completed to reduce I/I in the collection system. These projects have had a significant impact on reducing I/I and eliminating SSOs. The City has also spent significant time and effort to inspect and repair private sanitary sewer service connections across the community.

HR Green completed an assessment and hydraulic model of the sanitary system in 2013. The GIS based hydraulic model is a tool that can be used by the City to evaluate and predict specific problems in the collection system. The hydraulic model together with flow monitoring information gathered at specific locations can be used to help the City focus on specific areas of the collection system instead of major sections of repair or replacement.

The IDNR has recently implemented the Iowa Nutrient Strategy to reduce nutrients discharged from the largest wastewater treatment plants in the state. The Iowa Nutrient Strategy will have a huge impact on the wastewater treatment requirements for the City of Indianola. The strategy over time will reduce discharge of total nitrogen to 10 mg/l and total phosphorus to 1.0 mg/l. This Facility Plan includes planning for treatment at the proposed Indianola

Wastewater Treatment Plant to these effluent discharge levels. Information about the Iowa Nutrient Strategy is included in Appendix A.

The City of Indianola has experienced an extremely high peak flow to average wastewater flow ratio up to 8:1. This high peak flow is problematic both for the collection system and for wastewater treatment facilities. The City has recently completed collection system projects to reduce I/I with some success (reduced peak to average ratio to 7:1) but at a cost around $18M. The wastewater treatment plant is now faced with treating those high flows. This Facility Plan proposes Wet Weather Side Stream Treatment as a cost effective alternative to sizing the new secondary treatment facilities to treat the entire peak flow while meeting the proposed discharge permit. Wet Weather Side Stream treatment is a treatment concept to help protect the secondary treatment biology and plant stability during high flows.

2.2. PURPOSE AND SCOPE

The purpose of this Facility Plan is two-fold. First, the City of Indianola will use it as a guide to planning and designing wastewater treatment facilities to meet the City's wastewater treatment needs for the near and extended future. Second, the Facility Plan will be used by IDNR to review the proposed technologies and wastewater treatment infrastructure proposed to meet the environmental requirements required by the state and federal requirements. The Facility Plan must develop a flexible solution to meet the wastewater treatment requirements for the 20-year planning period and also more of a long-term vision for Indianola for beyond 50 years.

This Facility Plan is unique because its implementation isn't planned to be started for several years. The City expects to continue to treat wastewater at the existing North WWTF for the next five years or so. This is important for the City so they can continue to save for the project as they pay down other sewer debt. A second part of deferring the improvements is that the existing NWWTF continues to function in a somewhat reliable manner to meet the discharge permit. For now, the City is planning the construction of the new wastewater treatment plant at the Farm Site to start in the spring of 2020.

This Facility Plan was developed to provide a reliable wastewater treatment system to meet the next and future NPDES discharge permits in the most cost effective manner. The Facility Plan was developed around a reliable and flexible secondary treatment system and then a cost effective preliminary treatment system, solids processing system and operations infrastructure to support the plant operation. Several innovative concepts have been included to help reduce overall construction costs but yet handle all the flow and load conditions expected.

Although a sewer rate analysis was not part of this work, the project construction cost estimates will help to define increases in sewer rates to fund the project.

3. EXISTING CONDITIONS AND PROJECTIONS (Entire Section 3 Revised)

3.1. EXISTING SERVICE AREA

The Indianola North WWTF treats wastewater from the incorporated areas of town. Residential, commercial and industrial sources make up the wastewater flow. The plant is located on Cavitt Creek on the northwest side of town. There are approximately 83 miles of sanitary sewer in the city. The collection system has historically received significant Inflow and Infiltration (I/I) to the sanitary system. In 2014, the City completed construction of a four phased program to reduce I/I to eliminate overflows and bypassing that is associated with the heavy I/I. Since this program has been completed, the City has noticed a reduction in sanitary sewer flows. The new WWTF needs to be designed to accommodate and/or handle reasonable peak flows during wet weather.

3.2. POPULATION

The population serviced by the Indianola North WWTF is assumed based on census information. The current population of Indianola is estimated at 15,310.

Census population data for the years 1860-present is shown in Figure 3-1 below. A comprehensive plan had been completed for the City in October 2011. The comprehensive plan forecasted population trends through 2030 using up-to-date growth trends and extrapolated population projections. The same increasing rate used in the comprehensive plan has been used to estimate future population through the end of the facility planning period (2040). The projected values are also plotted in Figure 3-1.

In 2007, Central Iowa Regional Transportation Planning Alliance (CIRTPA) released its Long Range Transportation Plan. A more aggressive growth rate was used in the 2011 comprehensive plan and in this facility plan to estimate the 2040 design population.

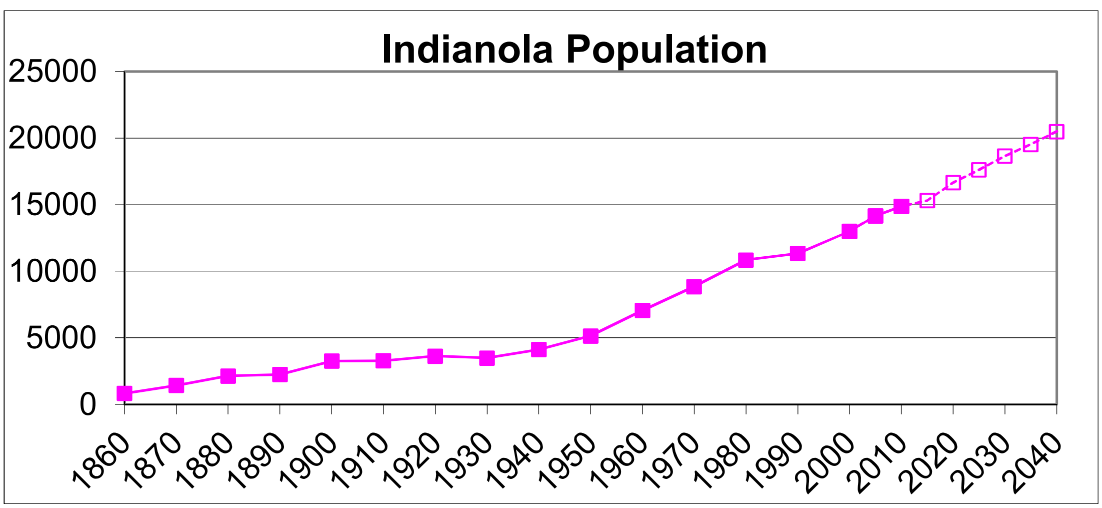


Figure 3-1 Indianola Population

The population for the future is assumed to follow the same general progression as in the past. See Table 3-1 for population projections.

Table 3-1 Population Projection Estimates

Year	Population
2020	16,657
2030	18,655
2040	20,491

3.3. EXISTING WASTEWATER FLOWS AND CHARACTERISTICS

Flow

Table 3-2 is a summary of the total influent wastewater flows discharged to the North WWTF for the period from 2010 through 2015. Total annual, daily average, and maximum day wastewater flows are shown. Also shown in Table 3-2 is the calculated ratio of maximum day flows to daily average flows.

Table 3-2 Influent Wastewater Flow Data for 2002 thru 2007

Year	Total Annual flow, MG	Daily Average Flow, MGD	Maximum Day Flow, MGD	Ratio of Max/Ave day
2010	1000	2.87	11.40	3.97
2011	799	2.19	11.58	5.28
2012	511	1.40	4.76	3.40
2013	623	1.70	11.21	6.58
2014	753	2.06	8.82	4.28
Average	737	2.04	9.55	4.70
Maximum	1000	2.87	11.58	6.58

(2015 data not shown in this table.)

The monthly average data from January 2010 thru March 2015 is charted in Figure 3-2. There are two sets of data plotted on this chart and several of the subsequent North WWTF flow charts. The data range titled “Total Flow (Includes EQ)” represents the entire wastewater flow that is conveyed to the North WWTF and is measured before excess flows are diverted to the equalization basin. The other data range titled “Thru Plant” only measures the flow that gets pumped through the plant after the diversion takes place.

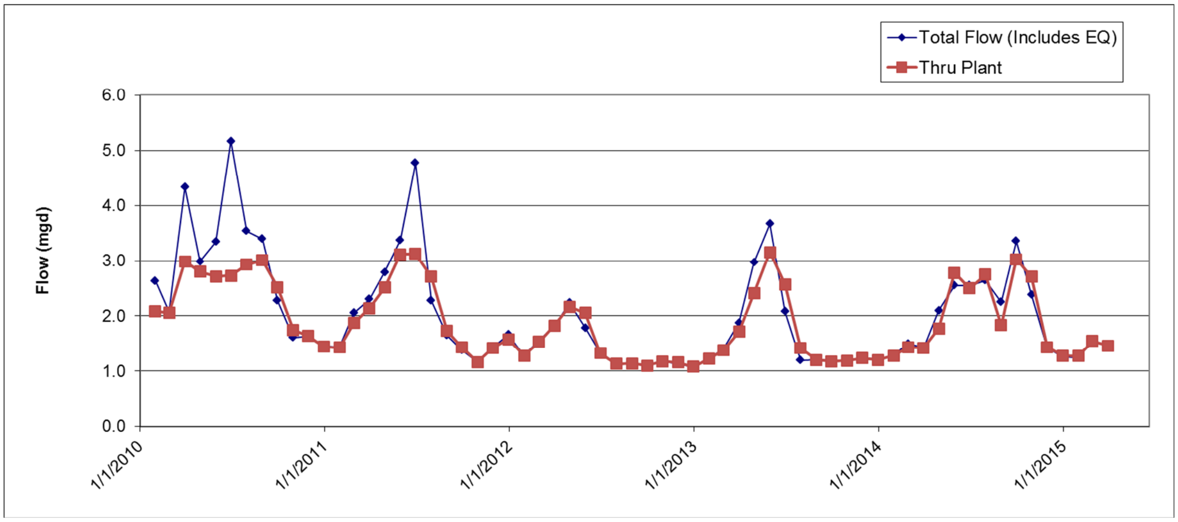


Figure 3-2 Monthly Averages (2010-2015)

The monthly data from January 2010 thru March 2015 was reviewed for max daily flows and is charted in Figure 3-3.

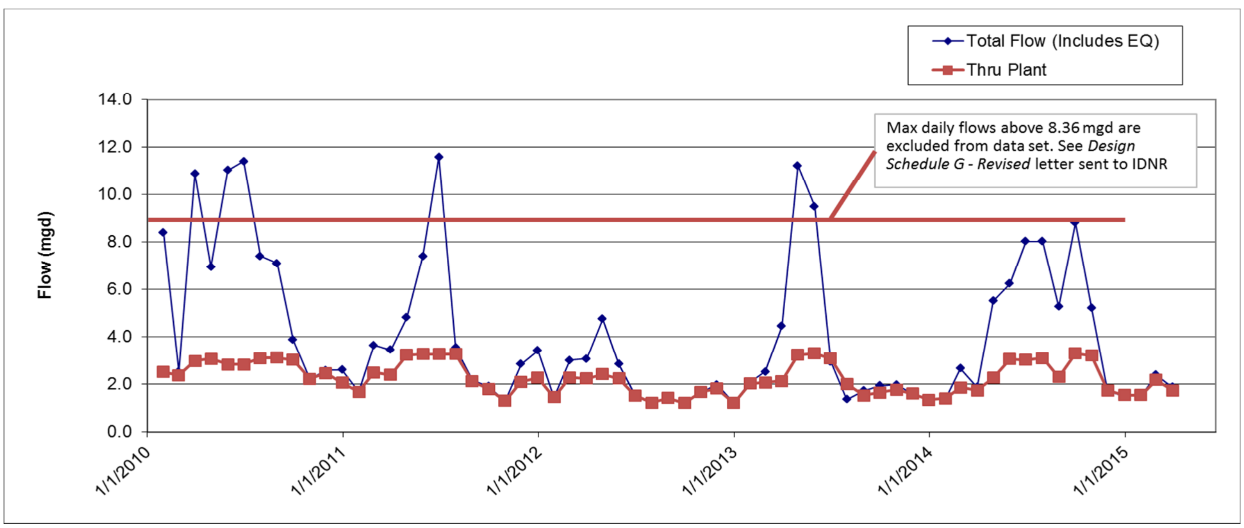


Figure 3-3 Maximum Daily Flows (2010-2015)

Average dry weather (ADW) is the daily average flow when the groundwater is at or near normal and runoff is not occurring. Average wet weather (AWW) is the daily average flow for the wettest thirty (30) consecutive days for mechanical plants. The maximum wet weather (MWW) is the total maximum flow received during any 24 hour period when groundwater is high and runoff is occurring. Peak hourly wet weather (PHWW) is the total maximum flow received during one hour when the groundwater is high, runoff is occurring, and the domestic, commercial and industrial flows are at their peak. Table 3-3 summarizes the ADW, AWW, MWW, and PHWW flows (through March 2015).

Table 3-3 Current Flows

Parameter	Value
ADW	1.56 MGD
Daily Average	2.02 MGD
AWW	5.17 MGD
MWW	8.36 MGD
PHWW (est.*)	13.67 MGD

* PHWW flow estimated from sanitary sewer model. This flow was based on the maximum flow received during one hour when the groundwater is high, runoff is occurring, and domestic, commercial, and industrial flows are at their peak.

Since the initial submittal of flows and loads report to the IDNR, the MWW and PHWW flows have been revised. See the *Design Schedule G – Revised* letter and the corresponding IDNR concurrence letter dated July 13, 2017 in Appendix B of this report for additional discussion and justification for these revisions.

Biochemical Oxygen Demand

Biochemical oxygen demand (BOD) is a measure of the strength of pollutants or oxygen reduction potential of the waste stream. Since effluent regulations have required nitrification, regulators have allowed carbonaceous biochemical oxygen demand (cBOD) tests to be used. These tests inhibit the effects of nitrifying biomass in the sample. The nitrifying biomass can give false readings in the BOD test. Therefore, cBOD tests have been completed. This test is also allowed on the influent samples for simplicity. The cBOD test has been shown to underestimate BOD strength of the influent wastewater by 15% or even more. The relationship between cBOD and BOD is plant specific, and possibly seasonal. This should be confirmed on a case-by-case basis. Through a range of plant testing in which BOD5 tests have been run alongside cBOD tests at the existing Indianola North WWTF, a ratio of 0.78 to 1.0 has been established for the relationship between CBOD and BOD, respectively. These results are also in line with a Study of Raw Wastewater BOD and cBOD Relationship article that was published by the Water Environment Foundation in 2006. The City has run these cBOD/BOD tests as 24 hour composite samples at multiple times during this year, in an attempt to establish the most representative and accurate ratio between the two tests.

The cBOD data was reviewed for period from 2010-2015 and is shown in Figure 3-4. The cBOD concentration is typical of low to medium strength wastewater. It should be noted that data from June 2014 through February 2015 was thrown out since it is believed the deionized water used in the cBOD test was contaminated with copper from the copper still used. The contamination of residual copper can inhibit bacterial activity and skew results from the cBOD test. The Figure 3-4 compares the 30-day cBOD concentration averages and maximums.

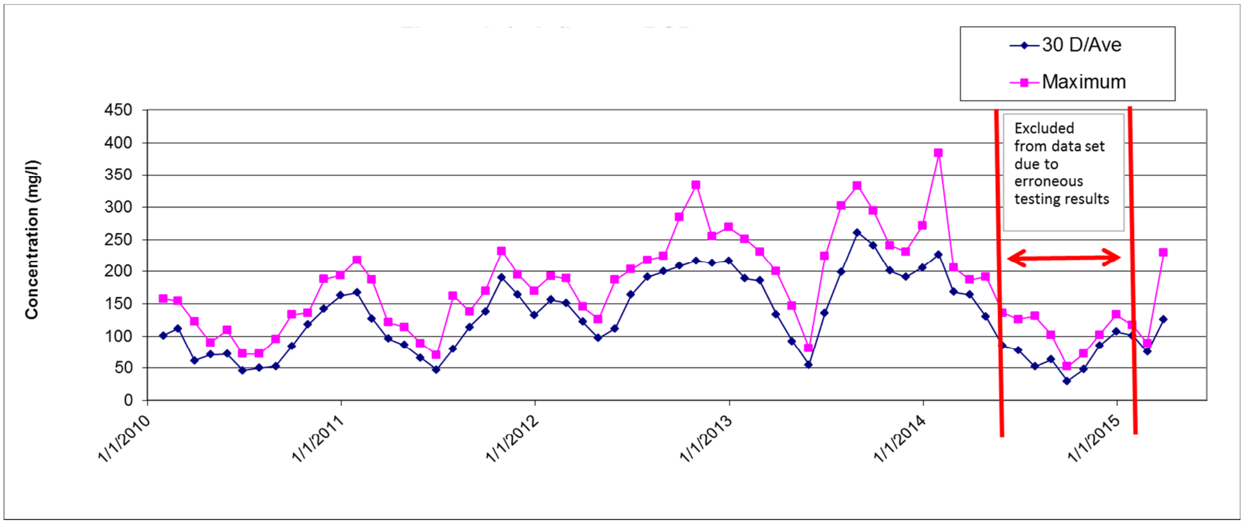


Figure 3-4 Influent cBOD

cBOD mass loading is shown in Figure 3-5. The seasonal fluctuation has no clear pattern. This chart again compares the 30-day averages with the maximum daily loading. The cBOD has been relatively steady throughout the data set that was evaluated, although there has been some slight increase in cBOD concentrations. This could be due to some of the improvements that the City has done to eliminate overflows and bypasses in the collection system. These improvements are intended to help reduce the infiltration and inflow to the sanitary system during peak flow events. Another effect is the waste concentrations in sanitary flows will be higher than those with higher contributions of I/I, and the organic loading to the sanitary system will be increased.

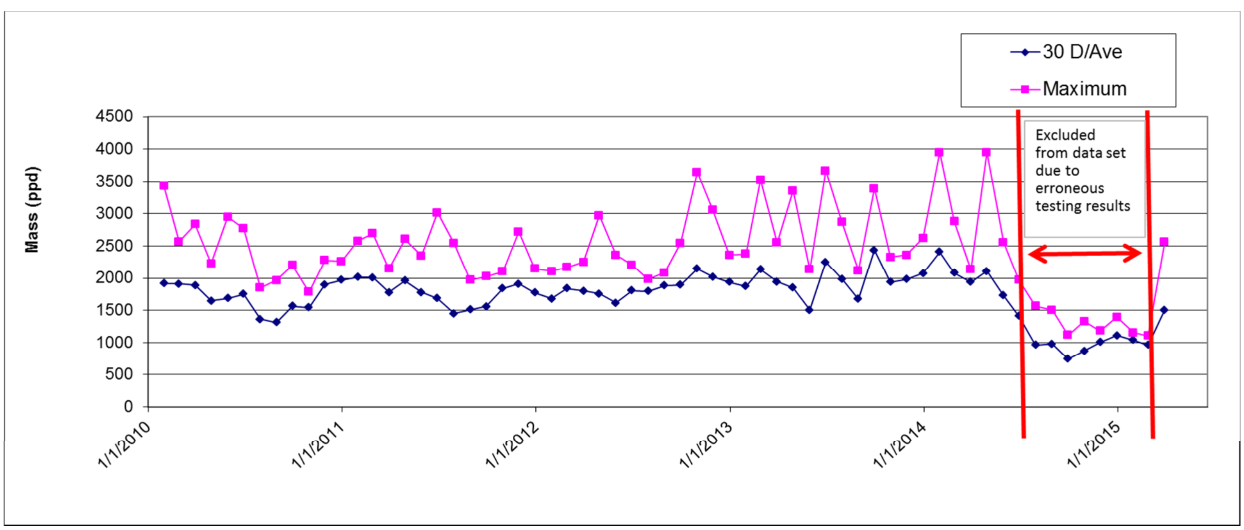


Figure 3-5 Influent cBOD Mass Loading

Organic loading data is summarized in Table 3-4.

Table 3-4 Current cBOD Loading (through 3/15)

Parameter	Value (ppd)
Average Month	1,840
Max Month	2,437
Max Day	3,952

Total Suspended Solids

Total suspended solids (TSS) data was reviewed from 2010 -2015. Figure 3-6 shows TSS loading of wastewater from January 2010 to March 2015. This chart compares the 30-day averages with the maximum daily loading. The January and June 2010 values are outliers.

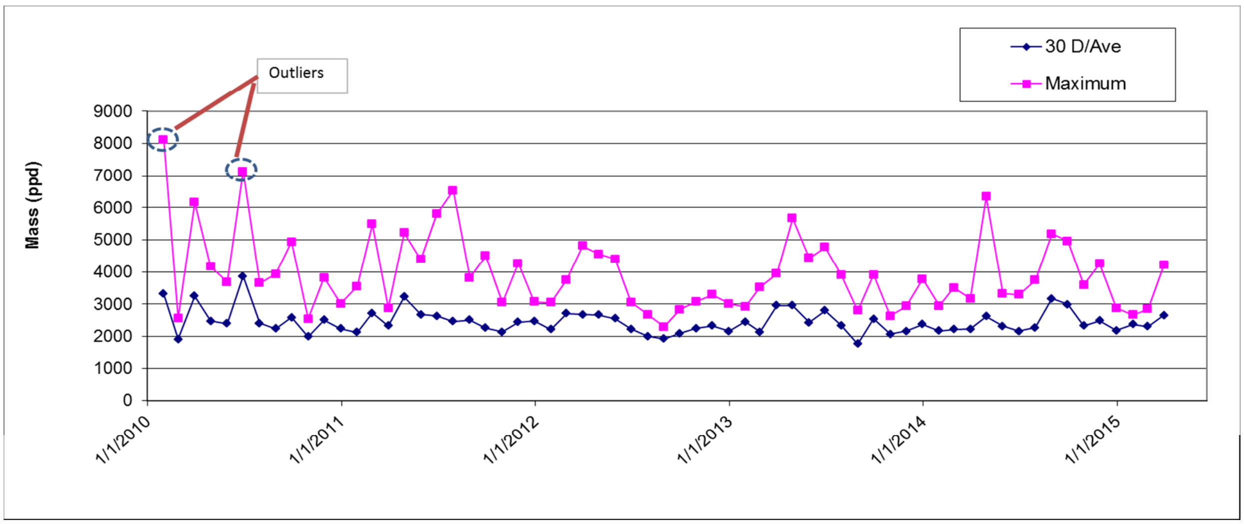


Figure 3-6 Influent TSS Mass Loading

TSS loading data is summarized in Table 3-5.

Table 3-5
Indianola North WWTF Historical TSS Loading 2010-2015

Parameter	Value (ppd)
Average Month	2,453
Max Month	3,859
Max Day	6,529*

* Outliers: 8118 and 7130

Ammonia-Nitrogen and Total Kjeldahl Nitrogen

The influent ammonia-N data was reviewed from 2010 -2015. Figure 3-7 shows influent ammonia-N loading of wastewater from January 2010 to March 2015. This chart compares the 30-day averages with the maximum daily loading. The high ammonia-N maximum loadings from April – June of 2013 are uncharacteristic and nominally 50% higher than other data reported for the evaluation period. After further evaluation of these spikes, it was discovered that the likely cause of these spikes was false readings from an ammonia-selective electrode used to determine ammonia content of samples. These three spike readings should be discarded from the evaluation results. See the *Revised Ammonia Loadings* memo Appendix C for more discussion on this topic.

It was also discovered that influent ammonia readings have generally been artificially high due to the influence of a supernatant flow stream from the plant's biosolids storage tank. Testing has shown that the ammonia results are higher when decanting than without decanting. In addition to the supernatant from the biosolids storage tank, there is also a supernatant line from the anaerobic digesters that decants less frequently but also contributes to superficial ammonia readings. To establish the max month and max day ammonia loadings, typical peaking factors can be assumed. Metcalf and Eddy, 2003, *Wastewater Engineering, Treatment and Reuse, 4th Edition. Metcalf and Eddy* gives typical information on the ratio of averaged peak and low-constituent mass loadings to average mass loadings. Typical ammonia peaking ratios for max day to average and for max 30 day to average are 2.0 to 1 and 1.5 to 1, respectively. Therefore, the current max day ammonia load can be taken as 490 ppd and 368 ppd.

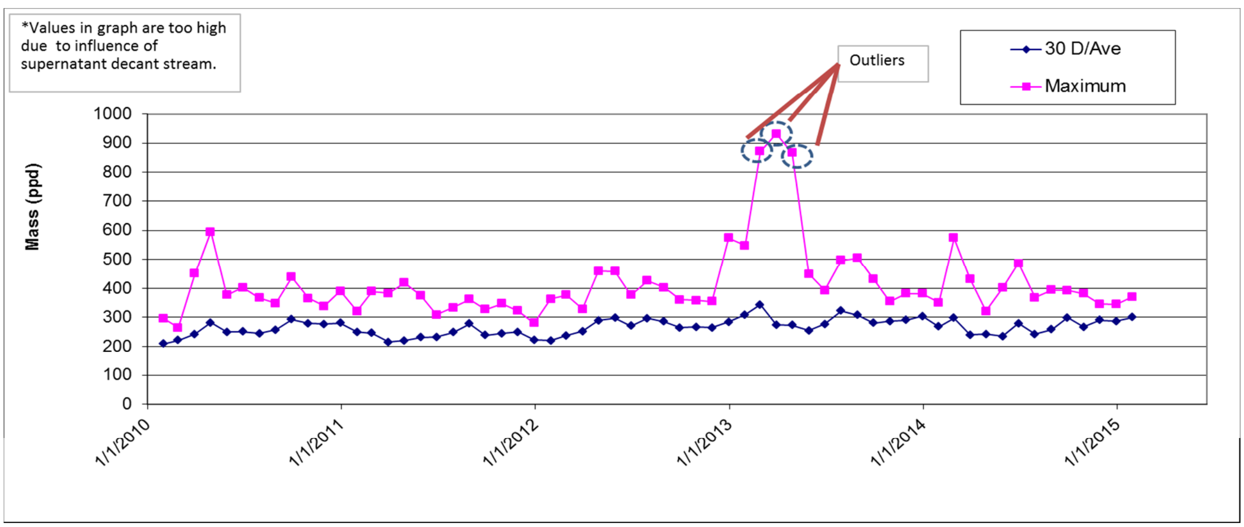


Figure 3-7 Influent Ammonia

Total kjeldahl nitrogen (TKN) data was not regularly monitored in history. For facility planning purposes, TKN was estimated based off the typical relationship between ammonia-N and TKN. This relationship was estimated using Metcalf and Eddy, 2003, *Wastewater Engineering, Treatment and Reuse, 4th Edition*. Ammonia loading data is summarized in Table 3-6.

Table 3-6
Indianola North WWTF Historical Ammonia Loading 2010 - 2015

Parameter	Value (ppd)
Average Month	240
Max Month	368
Max Day	490

Population Equivalent Analysis

The flows and pollutant loadings were reviewed for data spanning January 2010 through March 2015. The monthly flows were reviewed for each year, and the months (typically November through February) where the groundwater table was historically near normal with little or no runoff occurring were selected for each year and averaged to find the ADW. The ADW from 2010 to 2015 is 1.56 MGD. This flow per capita (15,310 persons) is 102 gal/capita/day which is close to typical (typical value is 100 gal/capita/day for domestic wastewater flow). The cBOD loading during the same time period is 1,840 lbs/day and 2,437 lbs/day for average and max month conditions, respectively. The BOD loading during the same time period is 2,359 lbs/day and 3,124 lbs/day for average and max month conditions, respectively.The ratio is 1.32 max month/average. The average loading per capita is 0.15 lb/capita/day, which is on the low side of the typical value (0.17 lb/capita/day of BOD). The TSS loading during this time period is 2,453 lbs/day and 3,859 lbs/day for average and max month conditions respectively. This ratio is 1.57 max month/average. The average loading per capita is 0.16 lb/capita/day, which is slightly low but within the typical range (0.13-0.33 lb/capita/day). The ammonia-N loading during this time period is 240 lbs/day and 368 lbs/day for average and max month conditions respectively. This ratio is 1.53 max month/average. The average loading per capita is 0.016 lb/capita/day, which is within the typical range (0.011-0.026 lb/capita/day). See Table 3-7 for a summary of the historic Flow, cBOD, BOD, TSS, and Ammonia loadings during the indicated time period.

Table 3-7
Indianola North WWTF Historical Flows and Loads 2010-2015

Parameter	Value	Per Capita (Est)
Flow		
ADW	1.56 MGD	102 gal/cap/day
AWW	5.17 MGD	
MWW	8.36 MGD	
PHWW	13.67 MGD	
cBOD		
Average	1840 lbs/day	0.12 lbs/cap/day
Max Month	2437 lbs/day	
Max Day	3952 lbs/day	
BOD (calculated from cBOD influent data)		
Average	2359 lbs/day	0.15 lbs/cap/day
Max Month	3124 lbs/day	
Max Day	5067 lbs/day	
TSS		
Average	2453 lbs/day	0.16 lbs/cap/day
Max Month	3859 lbs/day	
Max Day	6529 lbs/day	
Ammonia-N		
Average	240 lbs/day	0.016 lbs/cap/day
Max Month	368 lbs/day	
Max Day	490 lbs/day	

Total Phosphorous

The Iowa Nutrient Strategy applies to Indianola. The State has adopted the Iowa Nutrient Strategy which will require Grade IV WWTPs to meet more stringent effluent requirements for Total Nitrogen and Phosphorus removal. In anticipation for these effluent limits, the City of Indianola has performed testing of their raw influent total phosphorous (TP). The testing to date has been performed in the spring of 2015 and the fall of 2017. Generally, the testing has shown that influent TP is within the range of 4.4 – 6.3 mg/L with an average value of 5.3 mg/L. These results are typical of domestic wastewater.

The average TP loading during the testing is 69 ppd. To establish max month and max day TP loadings, typical peaking factors can be assumed. Metcalf and Eddy, 2003, *Wastewater Engineering, Treatment and Reuse, 4th Edition. Metcalf and Eddy* gives typical information on the ratio of averaged peak and low-constituent mass loadings to average mass loadings. Typical TP peaking ratios for max day to average and for max 30 day to average are 2.2 to 1 and 1.5 to 1, respectively. Therefore, the current max day total phosphorus load can be taken as 152 ppd and 103 ppd.

TP loading data is summarized in Table 3-8.

Table 3-8
Indianola North WWTF TP Historical Loading 2015 & 2017

Parameter	Value (ppd)
Average Month	69
Max Month	103
Max Day	152

4. EXISTING FACILITIES EVALUATION

4.1. EXISTING COLLECTION SYSTEM

The existing collection system consists of approximately 83 miles of sanitary sewer, 1,560 manholes, 10 lift stations, and two equalization basins. The sanitary sewer piping ranges from 6 to 36-inch of varying material types. All flow is directed to the wastewater treatment plant located at the north west corner of town. A map of the system is shown in Figure 4-1. The map also includes the lift station catchment boundaries. There are ten (10) lift stations within the collection system and eight (8) catchment areas. Two (2) of the lift stations (North Plant Lagoon Lift Station and South Plant EQ Lift Station) are required for pumping flow into the equalization basins

The McCord Catchment is pumped by the McCord lift station into the South Plant Catchment. The South Plant Catchment is then pumped into a force main that runs parallel with a force main from the Plainview Lift Station. These two parallel force mains convey flow to the Morlock Catchment Area. The Morlock Catchment area is then pumped by the Morlock lift station to the North Plant Catchment. The wastewater then flows by gravity to the North Plant Lift Station. The Wesley, N 65/69 Catchment and Quail Meadows Catchment are pumped into the North Plant catchment and then flow by gravity to the North Plant Lift Station. Once the flow gets to the North Plant Lift Station it is pumped into the treatment processes at the North WWTF. A flow diagram of the lift stations is included in Figure 4-2.

The two equalization basins are located at the South Plant Lift Station and at the North WWTF. The South Plant Equalization Basin has an approximate volume of 13 Million Gallons (MG). There is a splitter box at this site that allows high flows to be redirected into the South Plant EQ Lift Station before being pumped into the equalization basin. When high flows subside, wastewater in the equalization basin is metered and brought back to the South Plant Lift Station. The North WWTF Equalization Basin has an approximate volume of 27 MG. Flows above the setpoint of the North Plant Lift Station are split in the Influent Control Structure and flow into the North Plant Lagoon Lift Station. When high flows subside, the wastewater from the equalization basin is drained back by gravity to the Influent Control Structure and measured in a flume before dumping into the North Plant Lift Station.

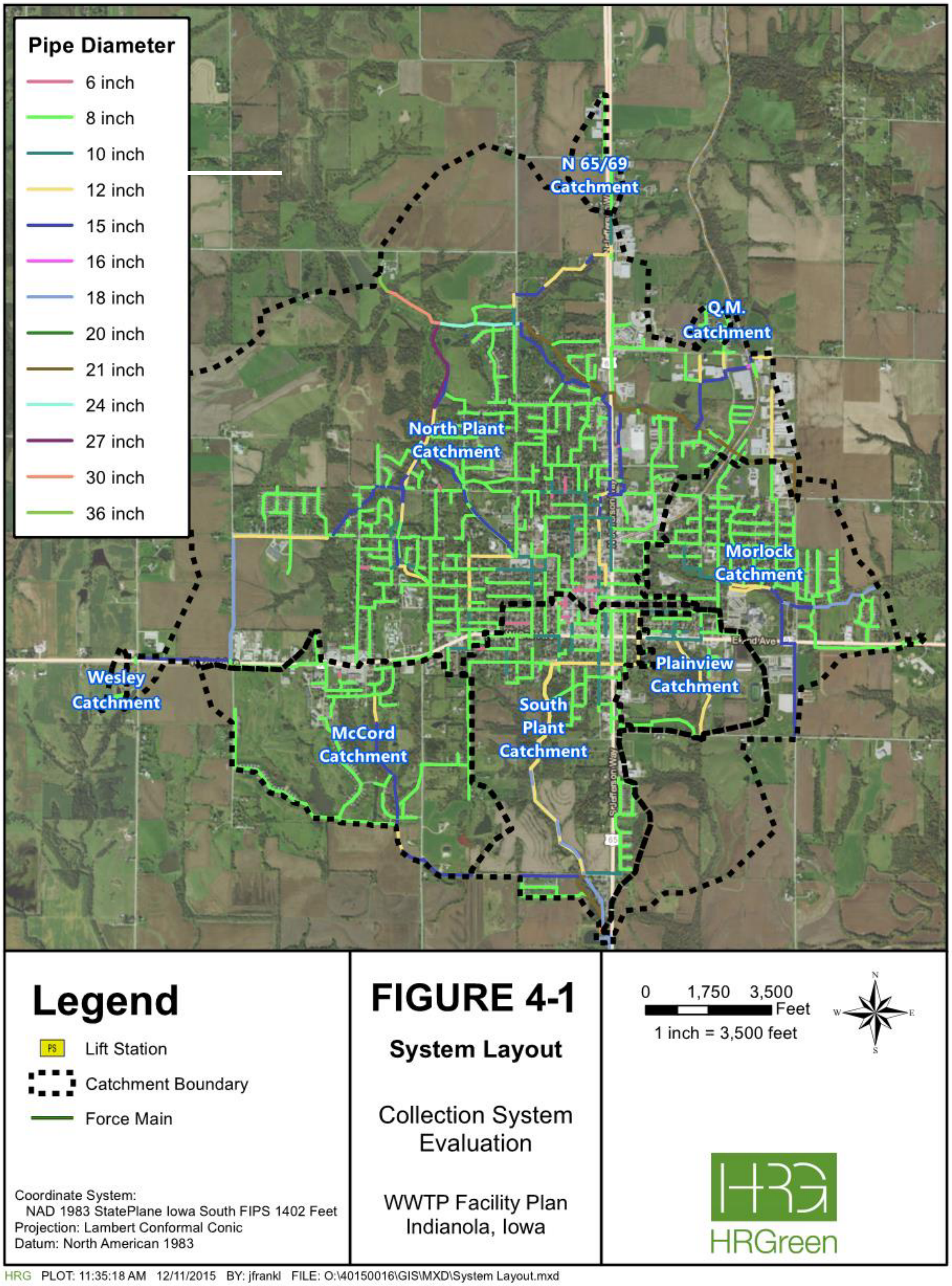


Figure 4-1 System Layout

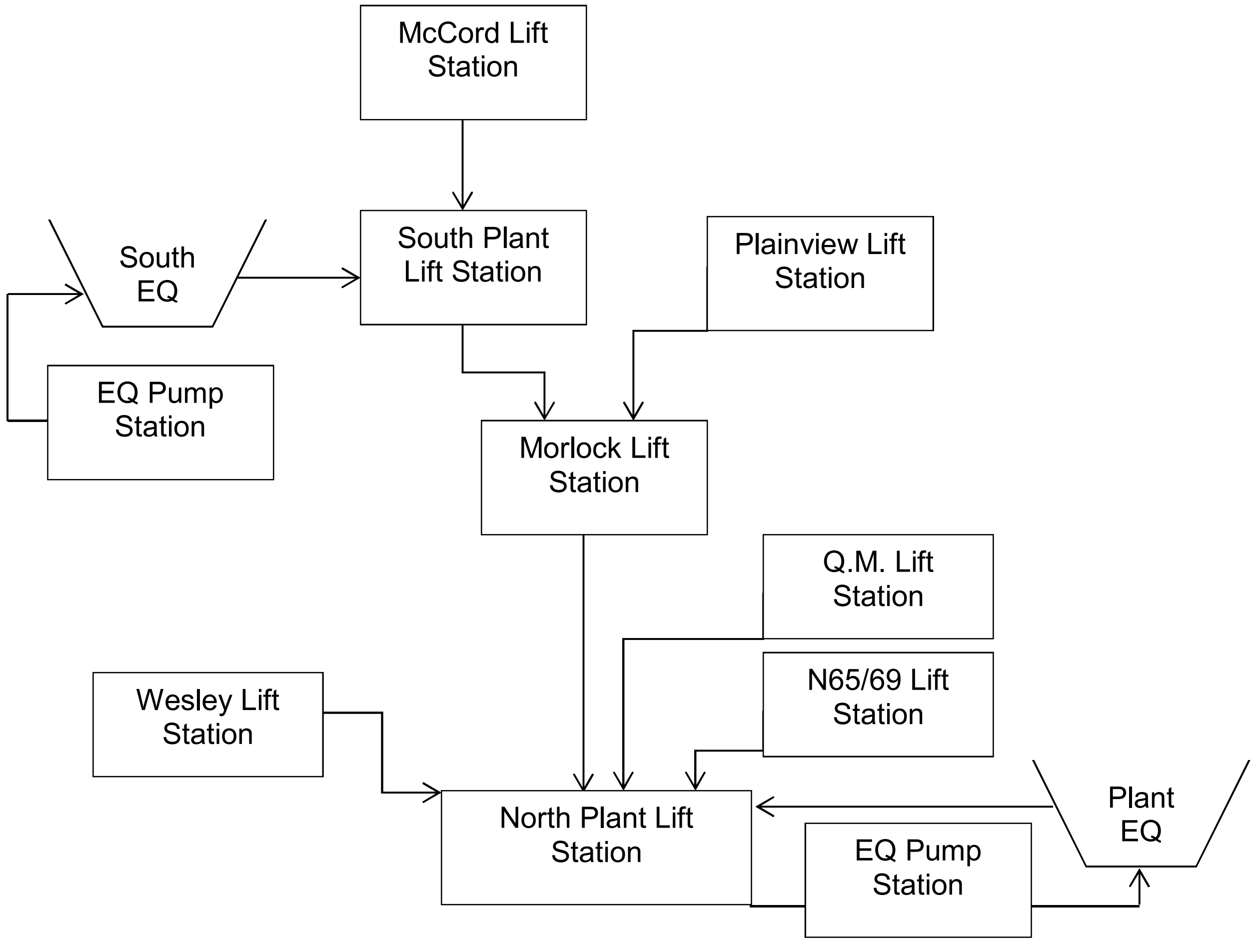


Figure 4-2 Lift Station Flow Diagram

The gravity sewers experience a large amount of excess flow (i.e. inflow and infiltration) during wet weather events and a high peaking factor compared to the average dry weather flows. The excessive wet weather flow was causing surcharging of the gravity system and sanitary sewer overflows (SSO's) at various locations in the sanitary sewer system. Due to the high peaking factor and excessive wet weather flows in the sanitary sewer system, the City implemented a phased program to reduce the inflow and infiltration (I&I) in the system and eliminate surcharging and SSO's. The program that was implemented was divided into four phases and became an Administrative Consent Order authorized by the Iowa Department of Natural Resources in 2009. The improvements that were implemented as part of this program included manhole inspections, sewer main televising, flow metering, sewer lining, residential inspections, sewer point repairs, manhole sealing, manhole replacement, sewer service lining, external sewer point repairs, replacement of sanitary sewer mains, expansion of the South Plant Equalization basin, conversion of polishing pond into equalization basin, and other miscellaneous improvements.

The Administrative Consent Order was satisfied in 2014. With the four-phased project complete the City has replaced or lined approximately 25% of their collection system sewers and replaced or repaired approximately 35% of their sewer manholes since 2008 along with the improvements listed above. The City has seen a significant decrease in excessive I&I and SSO's since these improvements were made. Even though the City is not under Administrative Consent Order, they are still committed to televising, inspecting, flow monitoring, and repairing the sanitary sewer system as a systematic approach.

4.2. MORLOCK LIFT STATION IMPROVEMENTS

During the time that this report was revised, the City of Indianola is partially complete with the construction of the Morlock Lift Station and Sanitary Sewer Improvements. The construction increases the capacity of the Morlock Lift Station, the sanitary force main, and a section of undersized gravity sewer downstream from where the force main connects. At the Morlock Lift Station itself one new pump will be installed, a valve vault is constructed to accurately meter the flows, new electrical pump controls and emergency engine generator will be installed, improvements to the HVAC system and much of the pump discharge piping is being replaced.

This Morlock Lift Station improvements project will allow the Morlock Lift Station to continue to operate reliably as the largest sanitary lift station in Indianola's sanitary sewer collection system. In addition the modifications at the lift station, force main, and gravity sanitary sewer allow the Morlock Lift Station to pump the sanitary flows projected from the Sanitary Sewer Hydraulic Model from the 25 year design event. These improvements should eliminate all known SSO's in the Morlock drainage basin. The construction projects for the Morlock Lift Station, force main, and gravity sanitary sewer should be completed by July of 2018.

4.3. SOUTH PLANT SYSTEM

The South WWTP in Indianola was taken out of service in the 1990's and converted to earthen basin equalization and a sanitary lift station. The earthen equalization basin capacity was approximately 9.0 million gallons to equalize peak flows from the south collection system. The South Lift Station pumped flows to the Morlock Lift Station and the Morlock Lift Station pumped the sanitary sewer flows on to the NWWTP.

As part of the Administrative Order in 2009 to make improvements to the collection system, the South Lift Station equalization basins were expanded in 2013 to approximately 13.0 million gallons. The additional equalization basin volume was intended to eliminate SSO's from the South Lift Station.

As the City and HR Green developed the Morlock Lift Station Improvements project, it was determined that during peak flow events when Morlock was surcharged, the South Lift Station was actually shut off from continuing to send flow to the Morlock catchment. Obviously, this operational configuration put more stress on the equalization volume at the South Lift Station and there was a higher risk of SSO's at that lift station and equalization basin.

When the current Morlock Lift Station Improvements are complete (planned for July of 2018) the new Morlock Lift Station will be able to handle a 25 year peak flow event without shutting flows off from the South Lift Station and overloading the 13.0 million gallon equalization basins. A Technical Memorandum is included in Appendix D of this report analyzing the storage capacity at the South Lift Station combined with its continuous pumping capacity.

4.4. EXISTING TREATMENT PLANT SITE

In 1978, the City of Indianola constructed the North Wastewater Treatment Facility (NWWTF) to serve the north part of the City and upgraded the south plant which served the southern area of the City. In 1992 the City abandoned the south plant and constructed collection system facilities to convey all wastewater flows to the NWWTF. Various improvements projects have been completed at the NWWTF over the years to increase the treatment capacity.

The NWWTF was designed for a 4.32 mgd maximum capacity through the treatment plant with any excess flows being pumped to the 27 MG equalization basin for treatment later. The treatment plant and equalization were designed to handle peak flows of 8.35 mgd. The existing NWWTF is located on approximately 32 acres on Hoover Street on the north edge of Indianola. The surrounding area to the north and west is mostly rural. A few houses are located just to the east of the existing plant site and the golf course owns property just to the south. Figure 4-3 shows an aerial map of the existing plant site.

The existing NWWTF discharges treated wastewater to Cavitt Creek. Cavitt Creek flows north to the Middle River.

Figure 4-3 Existing NWWTF Site Plan

4.5. EXISTING TREATMENT FACILITIES

The existing North WWTF includes much of the original 1978 construction and is mostly currently operating. An upgrade to the plant in 1994 added the Screening Building and made modifications to the Primary Pumping Station. Many of the process units are at the end or nearing the end of their useful life. The original plant was designed to treat 4.32 mgd with higher flows diverted to the equalization basin and then later brought back thru the wastewater treatment process. The current treatment capacity for the NWWTF is less than 4.0 mgd due to some of the equipment being inoperable. The reduction in capacity of the NWWTF results in difficulty operating the treatment facilities during wet weather flows.

The reliability of the secondary treatment process to remove ammonia during winter months is questionable. In the last few winters the plant has encountered upsets that have interrupted the nitrification process and stopped ammonia removal. During these times the Indianola wastewater treatment plant has violated its discharge permit for ammonia removal. With the low wastewater temperatures, it becomes difficult to get nitrification restarted.

A more comprehensive summary of existing wastewater treatment plant condition is as follows:

Preliminary Treatment: The preliminary treatment at the existing wastewater treatment plant includes the following process units: Screening Building, junction chamber, primary pumping station, 27 million gallon earthen equalization basin and grit removal system. The Screening Building includes one mechanical screen capable of passing 12 mgd at high flows. However, during high flows the flow runs out of the channel and much of it bypasses the screen. The Primary Pump Station includes treatment plant pumps and lagoon pumps. Several of these pumps are not operational and need replacement. Additionally the flow meters for each of these pumping systems need replacement. Also, the electrical and mechanical systems are badly corroded and are in need of wholesale replacement. The existing earthen equalization basin capacity has been reduced over the years by sludge and grit that has deposited in the basin. A lagoon cleaning project needs to occur to restore the equalization basin capacity back to 27 million gallons. The grit removal system needs a replacement of equipment to effectively remove grit at the flows anticipated. Overall, the existing preliminary treatment system needs some fixes and replacement but generally if some of these repairs are made, it can continue in service for several more years.

Primary Treatment: Primary treatment includes the primary clarifiers, primary sludge pumping, secondary pumping station and fixed film reactor. This equipment was mostly part of the original plant construction. Generally, these process units and equipment are corroded and near the end of their useful life. The fixed film reactor system is nearing collapse and needs to be replaced if the process is continued. The secondary pump station needs major improvements and equipment replacement. The primary clarifiers have some remaining life with general equipment replacement but some major structural rehab needed also. Major investment is needed here if any of this equipment is to remain in service past only a few years.

Secondary Treatment: The secondary treatment system at the existing NWWTF includes aeration tanks with a medium bubble diffused aeration system, aeration blowers, final clarifiers with covers, waste activated sludge (WAS) and return activated sludge (RAS) pumping facilities. This equipment was mostly part of the original plant construction (except for the recent south clarifier equipment replacement and the RAS pump replacement). Generally, the secondary treatment system will not be adequate for future nutrient removal without major improvements and expansion. However, with the recent modifications to the equipment, the secondary treatment process should be reliable for ammonia removal for flows up to 3.0 mgd for the next few years.

Disinfection: An existing chlorine contact tank does exist at the plant, but plant effluent is not currently disinfected. Major improvements would be needed to retrofit the existing tank to meet disinfection requirements.

Solids Processing: The existing solids processing facilities at the NWWTF include anaerobic digestion with one primary digester and one secondary digester with ancillary systems. Much of the equipment in the anaerobic digestion process needs replacement, but generally these systems have some remaining life. In addition to the solids treatment process, the 2.0 million gallon biosolids storage tank is in adequate condition for some continued use.

Ancillary Facilities: Many of the ancillary buildings, building systems and employee spaces are in need of repair or replacement. These buildings and spaces do not generally meet current design codes and recommendations for employee spaces. The entire wastewater treatment plant is backed up by a stand-by engine generator that is in good condition.

In summary, the overall condition of the existing wastewater treatment facilities at the NWWTF is poor. Additionally, the reduced capacity of the treatment plant due to failing equipment creates problems with handling peak flows during prolonged wet weather conditions. The plant deficiencies and general manual operation have significantly increased the attention needed by operations staff. The existing NWWTF should not be considered a reliable wastewater treatment facility beyond only a few years.

5. DESIGN CONDITIONS

5.1. GENERAL

This chapter discusses the water quality standards and effluent limitations which impact the proposed improvements to the Indianola, Iowa wastewater treatment facilities. Point discharges of pollution in Iowa are regulated by permits issued by IDNR. Because the permits limit the quantity of certain parameters and pollutants in the effluent from point sources, the limitations which apply to a given effluent are essential for proper planning and design of wastewater treatment facilities. These effluent limitations are also, in turn, directly related to the water quality standards which apply to the river or stream receiving the discharge and must be appropriately modified to suit local conditions.

5.1.1. RECEIVING STREAMS

The City of Indianola currently discharges its treated wastewater into the Cavitt Creek a tributary to the Middle River. Cavitt Creek is classified as primary contact recreation use (Class 1 A) ·and a warm water fisheries - Type 2 (Class B(WW-2). The Middle River is classified as primary contact recreation use (Class 1 A) and a warm water fisheries -Type 1 (Class B(WW-1). *The wastewater treatment plant constructed at the Farm Site would have the option to discharge to either Cavitt Creek or the Middle River. Discharge to the Middle River is slightly more advantageous due to the higher low flow in the river. For this reason, the new wastewater treatment plant to be constructed at the Farm Site will discharge into the Middle River. A Waste Load Allocation for the Middle River receiving stream has been developed by IDNR and is attached in Appendix E of this report.*

5.1.2. WATER QUALITY STANDARDS

Water quality standards for the State of Iowa are regulated by IDNR and presented in Section 567 - Environmental Protection Commission of the Iowa Administrative Code under Chapter 61 - Water Quality Standards. IDNR has developed a classification system for all surface waters in the State of Iowa to define water quality according to use and for the protection of beneficial uses. This classification system establishes general use and designated use river and stream segments.

General use segments are watercourses with intermittent flow or typically flow only for short periods of time following precipitation or as a result of discharges from wastewater treatment facilities. These waters do not support a viable aquatic community of significance during low flow, and do not maintain pooled conditions during periods of no flow. However, during low periods when sufficient flow exists in the intermittent watercourses to support various uses, the general use segments are to be protected in accordance with the "General Water Quality Criteria" which are discussed later in this chapter. Also, aquatic life existing within these watercourses during elevated flows are to be protected from acutely toxic conditions.

Designated use segments are bodies of water which maintain flow throughout the year, or contain sufficient pooled areas during intermittent flow periods to maintain a viable aquatic community of significance.

Designated use waters are to be protected for all uses of general use segments in addition to the specific uses assigned. Designated use segments include;

Class A1 - Primary Contact Recreation Use: Waters in which recreational or other uses may result in prolonged and direct contact with the water, involving considerable risk of ingesting water in quantities sufficient to pose a health hazard. Such activities would include, but not be limited to, swimming, diving, water skiing, and water contact recreational canoeing.

Class A2 - Secondary Contact Recreational Use: Waters in which recreational or other uses may result in contact with the water that is either incidental or accidental. During the recreational use, the probability of ingesting appreciable quantities of water is minimal. Class A2 uses include fishing, commercial and recreational boating, any limited contact incidental to shoreline activities and activities in which users do not swim or float in the water body while on a boating activity.

Class A3 - Children's Recreational Use: Waters in which recreational uses by children are common. Class A3 waters are water bodies having definite banks and bed with visible evidence of the flow or occurrence of water. This type of use would primarily occur in urban or residential areas.

Class B(WW-1) Warm Water - Type 1: Waters in which temperature, flow and other habitat characteristics are suitable to maintain warm water game fish populations along with a resident aquatic community that includes a variety of native nongame fish and invertebrate species. These waters generally include border rivers, large interior rivers, and the lower segments of medium-size tributary streams.

Class B(WW-2) Warm Water - Type 2: Waters in which flow or other physical characteristics are capable of supporting a resident aquatic community that includes a variety of native nongame fish and invertebrate species. The flow and other physical characteristics limit the maintenance of warm water game fish populations. These waters generally consist of small perennially flowing streams.

IDNR has also established "General Water Quality Criteria" which are applicable to all surface waters including those which are designated use segments. As stated in Chapter 61, the "General Water Quality Criteria" are applicable at all places and at all times to protect livestock and wildlife watering, aquatic life, non-contact recreation, crop irrigation, and industrial, domestic, agricultural and other incidental water withdrawal uses not protected by specific numerical criteria. The "General Water Quality Criteria" are as follows:

1. Such waters shall be free from substances attributable to point source waste discharges that will settle to form sludge deposits.

2. Such waters shall be free from floating debris, oil, grease, scum, and other floating materials attributable to wastewater discharges or agricultural practices in amounts sufficient to create a nuisance.
3. Such waters shall be free from materials attributable to wastewater discharges or agricultural practices producing objectionable color, odor, or other aesthetically objectionable conditions.
4. Such waters shall be free from substances attributable to wastewater discharges or agricultural practices in concentrations or combinations which or toxic to human, animal, or plant life.
5. Such waters shall be free from substances attributable to wastewater discharges or agricultural practices, in quantities which would produce undesirable or nuisance aquatic life.
6. The turbidity of the receiving water shall not be increased by more than 25 Nephelometric turbidity units by any point source discharge.
7. Cations and anions guideline values to protect livestock watering may be found in the "Supporting Document for Iowa Water Quality Management Plans," Chapter IV, July 1976, as revised on November 11, 2009.
8. The Escherichia coli (E. coli) content of water which enters a sinkhole or losing stream segment, regardless of the water body's designated use, shall not exceed a Geometric Mean value of 126 organisms/100 ml or a sample maximum value of 235 organisms/100 ml. No new wastewater discharges will be allowed on watercourses which directly or indirectly enter sinkholes or losing stream segments.

5.2. EFFLUENT LIMITATIONS

The Federal Water Pollution Control Act Amendment of 1972 (PL92-500) increased the role each state plays in control of the discharge of pollutants into its waterways. Under this amendment, the National Pollutant Discharge Elimination System (NPDES) permit program was established which is administered by the Environmental protection Agency (EPA). Monitoring and surveillance of water quality is conducted by IDNR through its operation permit program. IDNR has assumed the responsibility of the NPDES program for the State and the program is now operated through the state operating permit system. The NPDES permit establishes effluent limitations for all wastewater treatment systems discharging or planning to discharge effluent to rivers and streams within the state of Iowa.

5.2.1. Existing Effluent Limitations

The Indianola, Iowa sewage treatment plant is currently operating under Iowa NPDES permit Number 91-33-001. The NPDES permit was issued January 2, 2002, and expired on January 1, 2007. *A copy of the permit is available online at the IDNR website.*

Table 5-1 presents the current effluent limitations for the Indianola wastewater treatment plant as stated in the NPDES permit. The effluent limitations are based on effluent discharge to the Cavitt Creek.

Table 5-1 NPDES Permit No. 91-33-001

Parameter	**Permit Limit**			
	30 Day Average		7 Day Average	
	mg/l	ppd	mg/l	ppd
$CBOD_5$	25	521	40	834
Total Suspended Solids	30	626	45	938
	30 Day Average		Daily Maximum	
Ammonia-Nitrogen	mg/l	ppd	mg/l	ppd
January	7.2	133	15.4	320
February	8.1	150	14.5	300
March	6.3	116	14.9	309
April	2.8	52	15.9	329
May	2.4	45	15.6	319
June	1.7	32	14.6	303
July	1.5	28	17.8	369
August	1.4	26	16.4	340
September	1.9	36	16.7	346
October	3.8	71	15.9	330
November	4.6	86	14.8	308
December	5.4	101	16.1	335
	Daily Minimum		Daily Maximum	
	Std Units		Std Units	
pH	6.0		9.0	
	Daily Minimum			
	mg/l			
Dissolved Oxygen	4.2			
	4.2			
	Ceriodaphnia		Pimephales	
Acute Toxicity	No Toxicity		No Toxicity	

5.2.2. ANTICIPATED LIMITATIONS

It is anticipated that future limitations for $CBOD_5$, TSS, and pH will not become more stringent. Based on recent changes to Iowa's water quality standards, more stringent ammonia limitations will be included when the facility's NPDES permit is reissued. *The anticipated ammonia limitations for the Middle River are indicated in the Waste Load Allocation presented in Appendix E.*

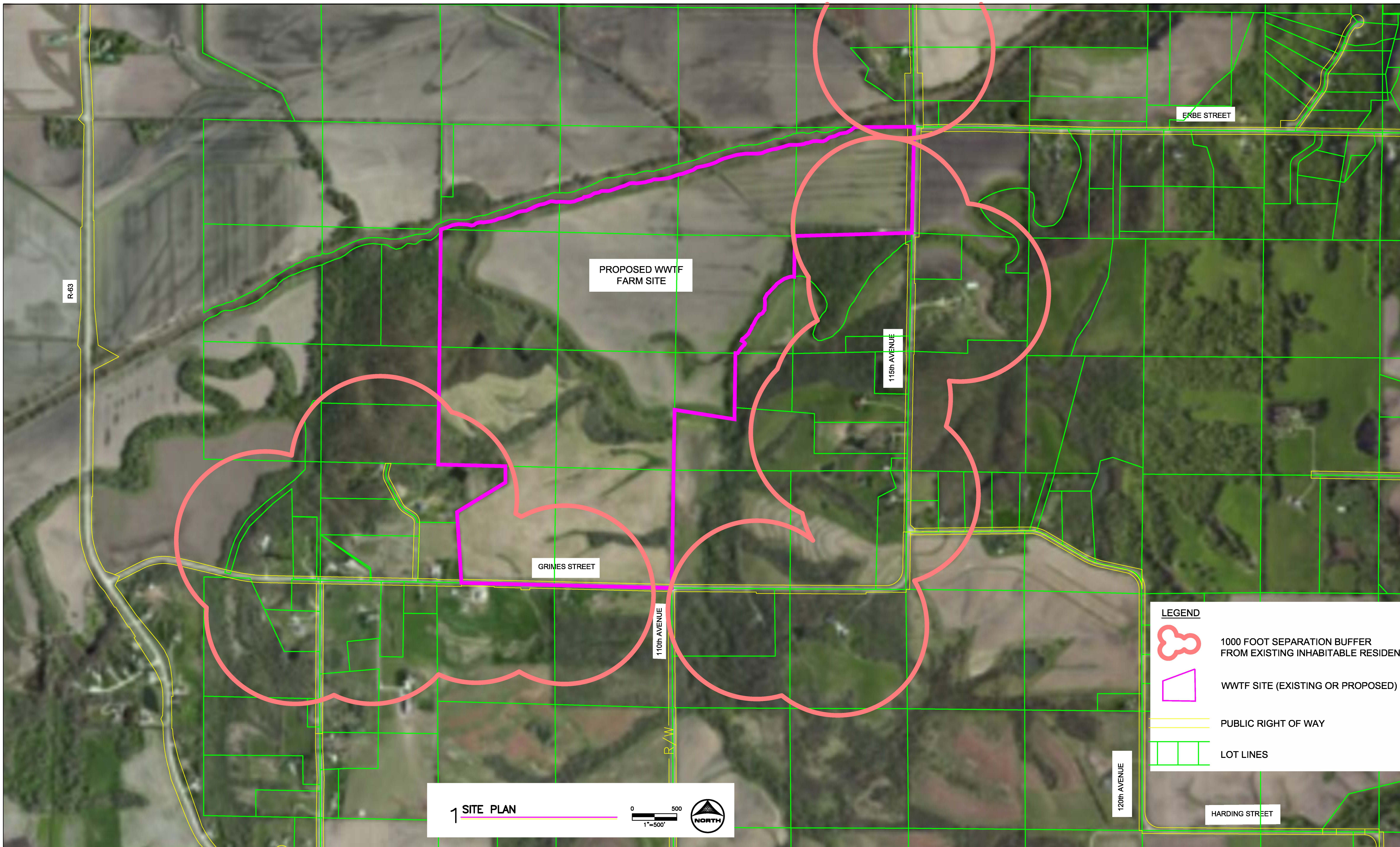
5.3. DESIGN WASTEWATER FLOWS AND CHARACTERISTICS

Forecasting the design flows and loads to the WWTF will be similar to the determinations for the design population. The permanent residential flows can be linearly interpreted by extrapolating the flow based on the per capita flows determined for the existing permanent residential population. ADW flows, Daily Average flows, AWW flows, MWW flows and PHWW flows are estimated by ratios from historical data. Average, Max Month, and Max Day loadings for cBOD, TSS, Ammonia-N, TKN, and total phosphorus were also linearly interpreted by extrapolating the loadings on the per capita loading rates determined for the existing permanent residential population.

According to the zoning map of the city, the industrial area is approximately 102 acres. The area also includes vacant, currently classified as agricultural, available for future industrial use. The current industrial contribution to the wastewater plant is not currently broken out from commercial/domestic contribution due to the small amount of existing industry in Indianola. The City plans to increase the amount of land zoned for industry in the future. In the City's future land use plan, part of the industry zone is "Light Industrial" and the other portion is "Heavy Industrial." Assuming portions of this future land use gets developed by the design year, industrial design flows and loads will be accounted for in the facility plan. 1000 gallons per day per acre (gpd/acre) and 2000 gpd/acre were used to calculate flows for light and heavy industry, respectively. cBOD, TSS, ammonia-N, and total phosphorus concentrations of industrial wastewater are assumed to be 300, 350 35 and 12 mg/L, respectively, according to the typical compositions of municipal wastewater. This is based on the fact that the industries will be required to pretreat their wastewater to the level of typical domestic flows as defined in the City's Sewer Ordinance. Permanent flows and loads shown in Table 5-2 include residential, industrial, and commercial sources.

Table 5-2 2040 Design Flows

Parameter	Residential Flow	Industrial Flow	Total
Flow (MGD)			
ADW	2.09	0.21	2.30
Daily Average	2.70	0.21	2.91
AWW	5.70	0.21	5.91
MWW	8.89	0.21	9.10
PHWW	14.20	0.21	14.41
cBOD (lbs/day)			
Average	2463	525	2988
Max Month	3262	525	3787
Max Day	5289	525	5815
BOD (lbs/day) – Calculated from cBOD data			
Average	3157	525	3683
Max Month	4181	525	4707
Max Day	6782	525	7307
TSS (lbs/day)			
Average	3283	613	3896
Max Month	5165	613	5778
Max Day	8738	613	9351
Ammonia-N (lbs/day)			
Ave Month	321	61	383
Max Month	493	61	554
Max Day	656	61	717
TKN (lbs/day) – Calculated from Ammonia-N data			
Average	494	94	588
Max Month	758	94	852
Max Day	1009	94	1103
TP (lbs/day)			
Average	92	14	106
Max Month	138	14	152
Max Day	203	14	217



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INDIANOLA – SITING STUDY
CITY OF INDIANOLA
INDIANOLA, IOWA 2013

FARM SITE SEPARATION PLAN

SHEET NO.
FIG 5–1

6. COLLECTION SYSTEM ALTERNATIVES

6.1. GENERAL

A more complete discussion of the existing collection system is included in Chapter 4. The City of Indianola has addressed in the past or is currently addressing many areas of the collection system where inflow and infiltration are concerns. Ongoing projects within the collection system are necessary to help limit the amount of excess clean water that needs to be treated in the wastewater treatment plant.

This chapter will focus on several aspects of the collection system that the City is recommended to evaluate moving forward. They include:

1. The Collection System Model that was recently developed
2. An evaluation of the lift stations within the collection system
3. Recommendations for the maintenance and improvements of the collection system

6.2. COLLECTION SYSTEM MODEL

The City recently completed a GIS survey for each manhole in the collection system and a Collection System Model. This model was developed starting in 2013 and submitted to the City in the summer of 2014, after the Administrative Consent Order work had been completed. The primary focus of this work was to examine the existing sanitary sewer system and establish a hydraulic model that can be utilized as a planning tool for future growth and design as more data is collected and input. The hydraulic model was developed to delineate problem areas by evaluating both the dry and wet weather conditions for the existing system. The model was then used to evaluate the adequacy of the collection and conveyance systems for existing and future flows. A summary of the collection system hydraulic model is included in Appendix F.

The first step in the development of the model was to collect physical attributes of the manholes and pipes. This included GPS data as well as a brief condition assessment. Incremental flow data was provided by the City. Daily flow data was also collected from the City's monthly operating reports as needed. The diurnal pattern associated with the baseline flow (portion of flow caused solely by sanitary use) was utilized as a template for sanitary loadings to individual utility structures throughout the system. The wet weather flow was modeled using a storm event (2.65 inches of precipitation) occurring on April 13, 2014.

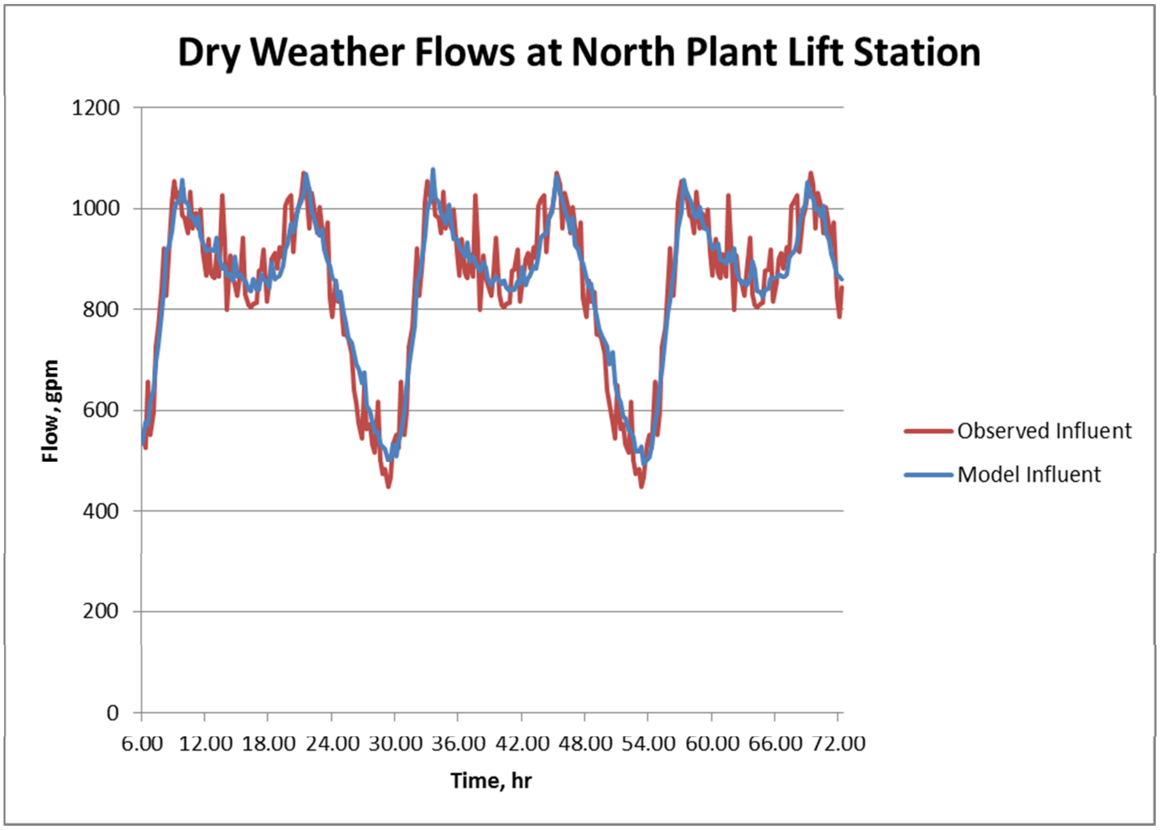


Figure 6-1 Collection System Model – City of Indianola Lift Station Influent Model Flows vs. North Lift Station Influent Observed Flows

Following calibration, four rainfall events were simulated within the model including the Base Flow Condition (aka dry weather flow). The model indicates that the existing piping is sized correctly to handle the dry weather base line flows. The system model indicates that during high rain events sewers in many of the catchment areas will start to surcharge and cause backups. These issues can generally be solved by either increasing the size of the collection system or decreasing the demand on the system by reducing I&I. Typically, eliminating inflow from the system is a more cost effective alternative then increasing the size of piping and utility structures and is the first choice of action. Based on the model results, a relatively small reduction in inflow would allow the system to accommodate a 100-year, 24-hour storm event without producing backups or overflowing any manholes in the collection system. In addition the sewer capacity evaluation, the lift stations were evaluated using modeled rain fall events. Most of the lift stations are sized adequately to handle wet weather flows. However, the Morlock Lift Station in particular should be further evaluated to address capacity issues. This lift station has a capacity that is significantly less than the required capacity during wet weather events. Improvements may include replacing pumps, adding storage volume near the Morlock Lift Station site, or adding a second discharge line to convey part of the flow to another basin.

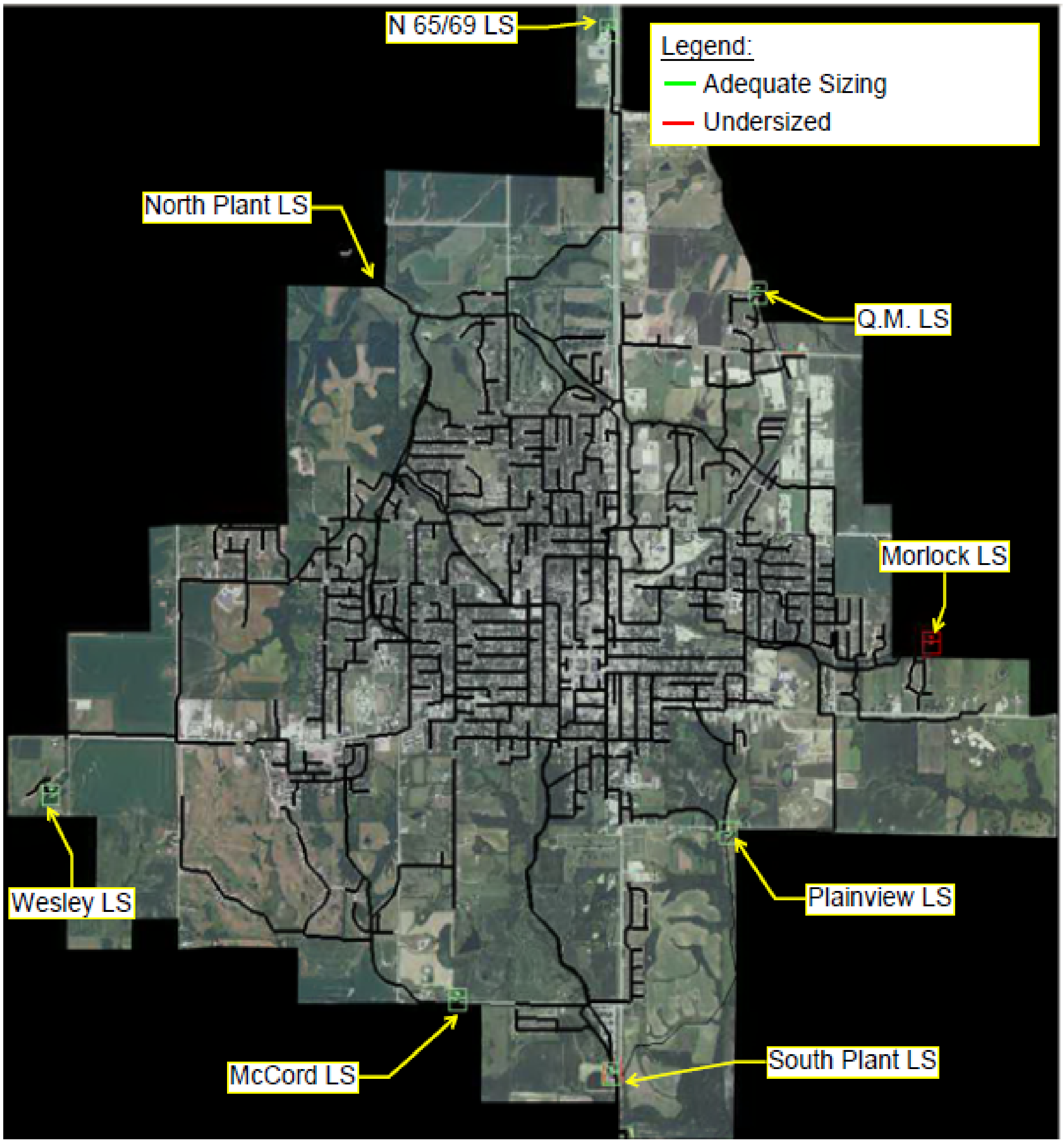


Figure 6-2 Model Output – Lift Station Analysis During 25-Year, 24-Hour Storm

Based on the information available, the model appears to be calibrated correctly to the existing system. Further calibration is recommended in the future to ensure accurate model results. In general, the large amount of inflow into the system is creating the most influential problems. The peaking factor of the wastewater is causing the collection system to be hydraulically overloaded. After the inflow has been addressed, the areas with the greatest amounts of infiltration should be identified. The system model should be utilized moving forward as a tool for assisting in the management of sanitary sewer collection system for resolving issues with the current system, and planning for future development and economic growth.

6.3. LIFT STATION EVALUATION

A lift station evaluation was conducted on April 30, 2015. Each of the 10 lift stations within the sanitary sewer system was evaluated to determine the existing capacity and condition. The evaluations focused on lift station facilities' condition (pump, piping, valves, flow meter, etc.), redundancy, structure integrity, emergency operation, mechanical features, alarming notification, and other miscellaneous characteristics of the lift stations. A summary of the observations and notes made during the lift station evaluation is shown in Table 6-1.

The lift stations in the system are generally sized correctly and in adequate condition to convey average dry weather flows. However, there are miscellaneous repairs and upgrades that should be periodically evaluated and made at the lift stations. The City is recommended to develop a maintenance program that includes all of the components of each lift station, the condition each component is in, and the priority for replacing or repairing the associated components. As noted from the hydraulic model, the Morlock Lift Station should be further evaluated for significant improvements. This lift station has significant capacity issues, especially during wet weather events. The force mains associated with each lift station should be included in the evaluation. The material, age, history of operation, air release valves, corrosion, and other elements should be considered when evaluating the force mains.

At the time of the revisions to this report a construction project is underway to upgrade and increase capacity of the Morlock Lift Station, its force main and a portion of downstream gravity sanitary sewer. The lift station observation notes corresponding to the Morlock Lift Station have been shaded to distinguish this proposed construction.

Table 6-1 Lift Station Observations and Notes

Lift Station	Pump Condition	Redundancy	Guiderails	Floats/Levels Control/Lead Lag	Structure (Concrete, coatings)	Site Grading/ Drainage
North Plant	Four - 35 HP Flygt submersible pumps; #3 and 4 were replaced within the last 5 years; #1 and 2 are original; 300 - 1000 gpm flow range each	All four pumps have operated at same time. With North Plant and North Plant Lagoon LS's both operating, can get about 14 MGD total flow. Can open valve and use lagoon pumps to pump to plant	Good condition - Recently replaced	Ultrasonic level sensor w/ backup floats; lead pump [is/previously was] operated off VFD	Wet well concrete structurally appears to be in good shape; tar coating. Valve vault concrete in good condition	No issues; 1.5 HP sump pump in valve vault
North Plant Lagoon	Two - 77 HP submersible pumps - about 3000 gpm each - original with plant construction; One - 9 HP submersible pump - about 500-600 gpm	All three pumps have operated at same time. With North Plant and North Plant Lagoon LS's both operating, can get about 14 MGD total flow.	Good condition	Ultrasonic level sensor w/ backup floats; constant speed	Wet well concrete structurally appears to be in good shape; tar coating. Valve vault concrete in good condition	No issues; 1.5 HP sump pump in valve vault
Morlock	Three total - 60 HP Crane Deming dry pit pumps; on VFD's. Each can pump around 650 gpm; max capacity is approx. 1250 gpm	Plant Staff did report that all three pumps have run at the same time. No redundancy; spot for a fourth pump	Monorail to lift dry pit pumps	Ultrasonic level sensor w/ backup floats; lead pump is operated off VFD	No coating in wet well; concrete has significant corrosion; dry pit concrete structure and building shell in good condition	No flooding. Needs better access to wet well
South Plant	Two total - 3171 Flygt dry pit pumps; constant speed; total combined flow approx. 650 gpm	Both pumps sometimes can't keep up; flow diverts then to EQ	Chain hoist for removal	Pressure transducer	Pump Station building and wet well appear to be in decent condition	Sump pump in pump station building
South Plant EQ	Four total - 40 HP submersible Vaughan Chopper pumps; total flow capacity approx. 4000 gpm; controlled by VFD's	Unsure if all four pumps have ever run at same time	Good condition	Ultrasonic level sensor with backup floats	Wet well and valve vault stucture in good condition - new	Sump pump in pump station building
McCord	Four total - 20 HP Pumps; Two - Flygt Model 3152 (original with plant ~1978); Two - Flygt Model 3153 (~3 years old); Constant speed, each pump can pump approx 350 gpm	Plant Staff did report that all four pumps have run at the same time	Good condition	Ultrasonic level sensor w/ backup floats; constant speed	Wet well concrete structurally appears to be in good shape; tar coating. Valve vault concrete in good condition	Site has been wet, but never flooded.Sump pump in valve vault
Plainview	Three total - 20 HP Pumps; Two - Flygt Model 3152 ; One - Flygt Model 3153; Constant speed, each pump can pump approx 250 gpm	Plant Staff did report that all three pumps have run at the same time	Moderate corrosion and build-up on guiderails	Ultrasonic level sensor w/ backup floats; constant speed	Wet well concrete structurally appears to be in good shape; tar coating. Valve vault concrete in good condition	No flooding issues. Sump pump in valve vault
N 65/69	Two total - 15-20 HP Flygt Model 3153 constant speed submersible pumps; each pump can pump approx. 250 gpm	Plant staff reported only one pump runs at a time	Good condition	Pressure transducer with backup floats	Concrete in good condition; no coating	Drain pipe from meter vault and valve vault into wet well
Quail Meadows	Two total - 2 HP Flygt Model 3068 constant speed submersible pumps; each pump approx. 65 gpm	Plant staff reported only one pump runs at a time	Good condition	Float control	Concrete in good condition; no coating	Drain pipe from valve vault into wet well; ditches/culverts for site drainage
Wesley	Two total Hydromatic 5 HP submersible constant speed pumps; each can pump approx. 20 gpm	Unsure if both pumps have ever run at same time	Good condition	Float control	Concrete in good condition; no coating; appears to be infiltration at joints	Water sitting in bottom of valve vault - drain pipe may be plugged

Table 6-1 (Continued)

Lift Station	Access/Hatch/Ladder	Emergency Operation	HVAC	Piping (Influent & Discharge)	Valves	Flow Meter/Air Release Valve	Protection from Clogging	Water Service	Odor Control	Alarm/Telemetry
North Plant	Cage Ladder down to old comminuters; valve vault stairs; aluminum hatches - all in good shape	Backed up on plant generator	Static vent for wet well; ventilator for valve vault runs for a while then kicks off	Significant corrosion on ductile iron pipe and fittings in wet well; light corrosion on valve vault piping; pump base/discharge elbow is corroded away on pipe-side	Check valves and plug valves appear to be in working condition; plug valve stem leaks	8" Magnetic Flow Meter	Upstream screenings facility	N/A	None	Plant SCADA; HWL, LWL alarms
North Plant Lagoon	Valve vault stairs; aluminum hatches - all in good shape	Backed up on plant generator	Static vent for wet well; ventilator for valve vault runs for a while then kicks off	Significant corrosion on ductile iron pipe and fittings in wet well; light corrosion on valve vault piping	Check valves and plug valves appear to be in working condition	10" Magnetic Flow Meter- off by factor of 2	Upstream screenings facility	N/A	None	Plant SCADA; HWL, LWL alarms
Morlock	MH casting to wet well has significant corrosion; stairs down to pump floor in decent condition	Standby generator; has underground diesel tank	Wet well blower doesn't work; ventilation inside building appears to work	Piping in building appears to be in good condition	Check valves in vertical orientation - have issues with not seating; two surge relief valves on discharge header	Magnetic flow meter needs to be verified	Communitors that are no longer being used. Solids buildup in wet well that needs to be removed	Used to have seal water but doesn't appear to be currently used	None	Alarms communicated via fiber
South Plant	Access stairwell in decent condition	Recently replaced generator and transfer switch	Ventilation not working in automated mode	DIP pipe has significant corrosion. Spool piece of PVC pipe used on north pump discharge piping	New gate valves on suction side; check valves in vertical orientation; surge relief valve and air release valve on discharge header	Krohn mag meter	Manually cleaned bar screen	Dry pit pumps don't appear to have seal water connections	None	Alarms communicated via fiber
South Plant EQ	Access hatches and steps in good condition	Recently replaced generator and transfer switch	Static vent for wet well and valve vault	All DIP is new and in good condition	Plug valves and check valves appear to be in good, working condition	None	Chopper pumps	N/A	None	Alarms communicated via fiber
McCord	Hatches don't have hinges. Valve vault ladder in good shape	Standby generator	Static vent for wet well and valve vault; Supply fan on valve vault disconnected/broken	DIP in wet well has light corrosision; piping in valve vault in good shape	Check/Plug valve in working condition; surge relief valve in valve vault also	6" magnetic flow meter	Guiderails for screen basket, but basket has been removed	N/A	None	Alarms communicated via fiber; Need to remove some existing abandoned conduit
Plainview	Hatches and ladder in good condition	Standby generator - will occaisionally kick off during test runs	Static vent for wet well and valve vault; Supply fan on valve vault disconnected/broken	DIP in wet well has mineral buildup; DIP in valve vault has light corrosion	Check valves and plug valves appear to be in working condition except for broken stem on pump 2 plug valve	6" magnetic flow meter	Guiderails for screen basket, but basket has been removed	N/A	None	Alarms communicated via fiber
N 65/69	MH castings on valve vault and meter vault and access hatch over wet well in good shape	Standby generator	Static vents on wet well and valve vault	DIP in good condition	Check valves and plug valve in good, working condition	8" Magnetic Flow Meter; air release valve in valve vault	Fiberglass screenings basket on guardrails	N/A	None	Alarms communicated via fiber
Quail Meadows	Hatches in good condition	Natural gas Standby generator	Static vent on wet well and valve vault	Stainless pipe that transitions into DIP; corrosion on DIP	Plug valves and check valves appear to be in working condition	Elapsed pump run-time counter	Screenings basket on guardrails	Have water yard hydrant on site	None	Autodialer
Wesley	Hatches on wet well and valve vault in good condition	Propane standby generator	None	Plastic discharge piping	Ball isolation valves and plastic check valves	Elapsed pump run-time counter	None - grinder pumps?	N/A	None	Autodialer

Figure 6-3 Morlock Lift Station Dry Pit Pumps (prior to Construction project)

Figure 6-4 South Plant Lift Station Dry Pit Pumps

Figure 6-5 McCord Lift Station Valve Vault

6.4. RECOMMENDATIONS

The City is recommended to move forward with identifying and removing deficiencies within the sanitary sewer collection system. The following is a list of recommendations and strategies that the City might consider:

- Data shows that inflow is occurring into the sanitary sewer collection system. The City is encouraged to further investigate potential locations of inflow in the system. The hydraulic model can be used to help identify the priority areas in the system to reduce inflow. The most cost effective way to reduce inflow is smoke testing and private residence inspections. This will allow the City to identify and reduce the number of clear water connections which directly connect to the sanitary system. Another location for high inflow potential is leaking manholes. There are a number of brick manholes in the system that could be contributing to the inflow. These manholes could be lined or replaced to assist in the reduction of inflow as well as infiltration. Typically, the next step after inflow has been addressed will be to determine the locations of greatest infiltration. This can either be completed using flow monitoring or televising. Flow monitoring is often better because televising is only a snapshot in time and planning televising to coincide with a rainfall event is problematic. Flow monitoring can be set up to measure flows at various points in the sewer system to help identify and isolate areas with high inflow and infiltration. Flows are measured continually over a period of time and can be correlated directly with rainfall events. Once problem lines are determined, the pipes could be lined or replaced. Typically longer or deeper runs are more cost effective to line than to replace. Again, the City is encouraged to use the hydraulic model as a tool for assisting in the management of sanitary sewer collection system, resolving issues with the current system, and planning for future development and economic growth.

- The City is also recommended to continue developing a maintenance program that includes all of the components of each lift station, its associated force main, the condition each component of the lift stations and force mains, and the priority for replacing or repairing the associated components. The Morlock Lift Station should be further evaluated for significant improvements, including capacity analysis and additional storage volume assessment.

- The City should continue efforts to televise and repair the sewers within the collection system. It is recommended that the collection system be broken out by the different catchment areas and evaluated on a systematic basis. Again, the hydraulic model will be an excellent tool to incorporate into the collection system analysis and will allow the City to better focus on key areas of the system that are critical in terms of capacity, condition, future development, and other considerations.

- Finally, the City is encouraged to conduct inspection and repairs of private services when a property is sold. An ordinance can be adopted that requires this inspection of private services at the time of sale of a home in lieu of completing the aggressive home inspection investigations that were conducted as part of the Administrative Consent Order work.

7. PRELIMINARY TREATMENT AND EQUALIZATION ALTERNATIVES

7.1. GENERAL

Preliminary treatment is used to remove large debris and grit from the incoming wastewater. In the case of influent screening the screens protect the downstream processes by removing debris and solids. Removing grit from the raw wastewater flow will keep grit from accumulating later in the treatment processes and significantly reduce maintenance. Influent flow measurement and influent sampling are important elements to develop into preliminary treatment also.

Primary treatment in the form of primary clarification can be an important physical process to reduce influent loadings ahead of secondary treatment. Primary treatment will not be considered for the Indianola wastewater treatment plant for several reasons: 1) influent loads are not high, 2) primary clarification is not needed for the secondary treatment alternatives considered, 3) primary clarification aligns best with anaerobic digestion for solids treatment and aerobic digestion for Indianola is much less expensive.

Equalization of influent wastewater flows has been an important strategy for handling the high PHWW flows through the wastewater treatment process at Indianola. Generally, flows above what can go thru the plant are shaved off into equalization and brought back through treatment after the peak flows subside (Store and Treat treatment concept). Because of the high ratio of peak to average flows, a WWTP design for handling peak influent flows will continue to be important for Indianola. Influent wastewater equalization can also be an important strategy to equalize the diurnal flows ahead of secondary treatment. This strategy will likely be more important as nutrient removal requirements continue to be lowered in the future.

Two options for preliminary treatment and equalization will be considered and evaluated for the new Indianola wastewater treatment facilities; P1) Reuse of screening, raw wastewater pumping and equalization (Store and Treat) at the existing treatment plant site with new fine screening and grit removal at the Farm Site; and P2) Convey all the influent flows to the Farm Site by gravity and construct larger new preliminary treatment and a much smaller equalization tank there, *then use Wet Weather Side Stream to treat the flows above the secondary treatment capacity and blend the two effluent streams*. The remaining portion of this section provides a detailed evaluation of these alternatives. *The end of this section (paragraph 7.4) includes an evaluation of Store and Treat treatment concept compared to Wet Weather Side Stream treatment and blending alternative.*

7.2. ALTERNATIVE P1

This alternative for preliminary treatment P1 consists of continuing to use the existing screening, raw wastewater pumping station, and equalization basin at the North WWTP; constructing a new sanitary sewer force main to the Farm Site; and, providing new fine screening and grit removal at the Farm Site. Flows up to 8.0 mgd would be conveyed to the Farm Site in the sanitary force main with peak flows above 8.0 mgd held in the existing 27 MG equalization basin for treatment later as the peak event subsides.

7.2.1. Existing Mechanical Screens

The existing mechanical bar screen in the existing Screening Building will continue to be used to keep debris from entering the pumps and equalization basin. The existing Screening Building was constructed in 2005 and includes one mechanical bar screen with automatic controls and a manual bar screen. The mechanical bar screen has a capacity of 12.0 mgd. Flows in excess of this screen are designed to be bypassed to the manual screen.

The existing Screening Building has experienced flooding in the past as a result of the downstream primary pump station not being able to keep up with the influent flows. At high flows the influent flow rises above the channel ahead of the mechanical bar screen and goes around the screen.

A second mechanical bar screen should be installed in the Screening Building in place of the manual screen to accommodate higher flows without bypass. Additionally, the existing mechanical bar screen will need to be replaced during the planning period to keep the Screening Building functional. No other major modifications are planned for the Screening Building.

7.2.2. Existing Influent Control Structure and Primary Pumping Station

The existing Influent Control Structure is part of the original plant construction and was designed to split flows to the plant pumps and the lagoon pumps. The structure is also where the flow from the equalization basin is returned and metered for treatment. The Primary Pump Station includes submersible pumps for the plant pumps and for the lagoon pumps. The Plant Pump Station was part of the original construction and later modified when the Screening Building was added around 2005. Much of the Primary Pumping Station pumps, piping, valves, flow meters, electrical and controls for the two pumping systems needs replacement to be used as part of this P1 preliminary treatment alternative. A new dry pit for discharge piping and flow measurement will be added to the Primary Pump Station structure for the discharge to the new force main to the Farm Site.

Significant electrical modifications to the existing power service entrance, switchgear, controls, etc. are planned for the remaining facilities.

7.2.3. Existing Equalization Basin

The North WWTF existing 27 million gallon earthen equalization basin will remain in service for this P1 Preliminary Treatment alternative. Generally, the equalization basin will continue to be operated as it is currently. The flows in excess of the new wastewater treatment plant's (at the Farm Site) capacity will be held until the influent flows following the peak flow event subside and then the equalized wastewater will be sent through the treatment plant.

The existing equalization basin currently holds a significant amount on grit and sludge and the real capacity is unknown. The City will need to complete a dredging project to restore the 27 MG of peak flow storage.

7.2.4. Sanitary Sewer Force Main

A new 18-inch sanitary sewer force main will be installed to convey flows from the existing North WWTF site to the Farm Site for wastewater treatment. The force main route has not been selected but is planned to generally follow the county road right-of-way. Combination air release and vacuum relief valve stations will be planned at each of the high points along the sanitary sewer force main alignment. The force main will be approximately 11,500 linear ft. Property acquisition costs for temporary and final easements for the sanitary force main are not included in project cost estimates at this time.

7.2.5. New Headworks Facilities at Farm Site

The new sanitary force main will convey the raw wastewater flow to a new Headworks Building at the Farm Site. The Headworks Building will include two new fine screens. A fine screen with openings of ¼-inches or less shall be used ahead of secondary activated sludge treatment systems. The actual fine screen selection will be based on a number of factors including; channel depth, amount of debris, desired capture rate, cleanliness of screenings, dryness of screenings, and maintenance. A bypass channel with manual screen will be provided also.

Fine screening increases the amount of organic material that is removed with the screenings. A screenings washer/compactor can be used to remove the organic material, dewater, and compact the screenings prior to disposal. This can be accomplished using an ancillary screenings washer/compactor, or by a screen with an integral screening washer/compactor.

Following fine screening, grit removal will be provided as part of the Headworks Building. Grit removal is used to remove fine particle inorganics from the waste stream. Removal of these materials from the wastewater reduces wear and maintenance on downstream processes such as pumps, tanks, etc. Grit not removed from the wastewater will end up in the downstream processes and reduce the capacity of these facilities. Also, land application of solids containing inorganic grit material is not desirable. Design criteria for the grit removal is 100% for particles 65 mesh or greater with a specific gravity of 2.65.

The Headworks Building will also house the influent sampling and flow measurement. Final selection of screening and grit removal equipment will occur in final design.

7.2.6. Benefits and Disadvantages of Preliminary Alternative P1

Benefits of Preliminary Treatment alternative P1

- Makes best use of existing wastewater preliminary treatment facilities at existing North WWTF
- Force main conveyance to Farm Site is minimal (8.0 mgd)

Disadvantages of Preliminary Treatment alternative P1

- Operation is difficult. Treatment facilities on two sites. May need larger operations and maintenance staff.
- Unable to re-purpose existing treatment plant site.
- May continue to have odor issues at existing North WWTF site.
- Will need small lift station at Farm Site to bring other gravity flows into the treatment process.
- Much of the facilities at the NWWTF are significantly into their useful life (may need attention during the planning period).

7.2.7. Alternative P1 – Opinion of Cost

A preliminary Opinion of Probable Construction Cost for alternative P1 is included in Table 7-1.

Table 7-1 Alternative P-1 Conceptual Opinion of Probable Construction Cost

Item	Description	Cost
North WWTP Site Improvements		
Lagoon Cleaning	dredging lagoon and LA of material	$180,000
Screening Building Improvements		
Added 2nd mechanical screen	modifications and new screen	$350,000
Replacement of original screen		$250,000
Primary Pumping station	8.0 mgd to the Farm Site	
Demolition w/ temp pumping		$60,000
Replacement of pumps	plant and lagoon pumps w/drives	$420,000
New Dry well		$100,000
Piping and valves		$200,000
Electrical and controls		$100,000
Site Electrical modifications	Service entrance, switchgear, enclosure	$270,000
	subtotal	**$1,930,000**
Force Main to Farm Site	approx 11,500 ft. of 18 inch	$1,700,000
Sitework	Sitework only related to alternative	
Yard Piping		$200,000
Return Pump station (1)	Submersible PS	$120,000
Headworks Building (1)	Influent screening and grit removal	
Building and substructure		$480,000
Mechanical Screens		$300,000
Slide Gates		$80,000
Vortex Grit System		$200,000
Grit pumps, piping and valves		$200,000
Mechanical/Plumbing		$80,000
Electrical/Controls		$140,000
	Total Alternative P1 Opinion of Construction Cost (2,3)	**$5,430,000**

(1) Includes concrete, excavation, backfill, superstructure, etc.
(2) Costs in Table do not include sitework, land acquisition, contractor overhead, demolition of old site, engineering or contingency
(3) Based on ENR Building Cost Index 5563 (Nov 2015)

7.3. ALTERNATIVE P2

This alternative for preliminary treatment P2 consists of abandoning all the wastewater preliminary treatment facilities at the existing North WWTF and conveying all the flows by gravity to the Farm Site for treatment. This alternative P2 includes a new gravity sanitary sewer to the Farm Site; new screening, pump station, grit removal, daily equalization and Wet Weather Side Stream treatment at the Farm Site. During peak flows the new wastewater treatment plant would treat the first 6.0 mgd of flow through the secondary treatment system with flows above 6.0 mgd being diverted around secondary treatment and treated by Wet Weather Side Stream treatment. The two effluent flow streams would then be blended and disinfected before discharge to the receiving stream.

7.3.1. New Gravity Sewer to Farm Site

A new gravity sanitary sewer to convey influent wastewater flows from the North WWTF to the Farm Site will be constructed to carry all the influent wastewater flows. The gravity sewer will be approximately 11,000 ft of 36-inch diameter. The sanitary sewer alignment will generally follow Cavitt Creek between the two wastewater treatment plant sites. Property acquisition costs for temporary and final easements for the sanitary sewer are not included in project cost estimates at this time.

7.3.2. Headworks Building

A new Headworks Building at the Farm Site will be constructed to provide influent screening and influent wastewater pumping to the downstream wastewater treatment processes. The influent screening and pumping capacity will be designed for the PHWW flow of 14.41mgd. The Headworks Building will sit just above the 100 year flood elevation (approximately elevation 806.00) at the Farm Site and pump up the hill to the remaining treatment facilities so that flows will flow by gravity through the plant.

The Headworks Building will include two fine screens. A fine screen with openings of ¼-inches or less shall be used ahead of secondary activated sludge treatment systems. The actual fine screen selection will be based on a number of factors including; channel depth, amount of debris, desired capture rate, cleanliness of screenings, dryness of screenings, and maintenance. A bypass channel with manual screen will be provided also.

Fine screening increases the amount of organic material that is removed with the screenings. A screenings washer/compactor can be used to remove the organic material, dewater, and compact the screenings prior to disposal. This can be accomplished using an ancillary screenings washer/compactor, or by a screen with an integral screening washer/compactor. Selection of fine screening equipment manufacturers will occur later in final design.

Several options for influent pumping are available for the flow and head range for the project. Submersible pumps are probably the least expensive option but would also generally require the most maintenance, particularly with the grit in the influent wastewater flow. A self-cleaning type wetwell with companion pumping equipment arrangement would be a good solution for pumping the influent wastewater flow with grit up the hill to the grit removal process.

The Headworks Building will also house the influent sampling and flow measurement. Final selection of screening and influent wastewater pumping equipment will occur in final design.

7.3.3. Grit Removal

The influent wastewater from the influent pumping station will enter the grit removal facility. The grit removal facility will remove grit from the influent wastewater over the entire range of flows including the PHWW flow. Several equipment configuration alternatives for grit removal are available for the flow range needed. Systems with low headloss will be a good starting point for equipment selection.

Grit removal is used to remove fine particle inorganics from the waste stream. Removal of these materials from the wastewater reduces wear and maintenance on downstream processes such as pumps, tanks, etc. Grit not removed from the wastewater will end up in the downstream processes and reduce the capacity of these facilities. Also, land application of solids containing inorganic grit material is not desirable. Design criteria for the grit removal is 100% for particles 65 mesh or greater with a specific gravity of 2.65.

Following grit removal, influent wastewater peak flows higher than 6.0 mgd will be diverted through an automatic downward opening gate to daily equalization. The base flow will flow by gravity to the secondary treatment system and the peak flows (higher than 6.0 mgd) will be; 1) equalized and treated thru secondary treatment, or 2) bypassed around secondary treatment and sent thru wet weather side stream treatment.

7.3.4. Daily Equalization Tank

A 2.0 million gallon cast-in-place concrete tank will be used for daily and peak flow equalization. The mode of operation method of the dual purpose tank will be selected by the operator.

In the “Daily Equalization” mode of operation, the downstream treatment plant is designed to treat a constant flow all day long. The operator selects the average daily flow anticipated for the 24 hour period. During that day the diurnal peak flows (flows above the preset average) are shaved into the daily equalization tank and then automatically returned back to the treatment process at night during low diurnal flows. This mode of operation is the best for consistent performance because the biology in the secondary treatment process sees the same load and flow all day. In the “Peak Flow” mode of operation, the equalization tank holds the pretreated wastewater for; 1) return to the treatment process when maximum flows through the treatment system subside, or 2) until the Wet Weather Side Stream Treatment system is on-line.

If the operator has selected the “Daily Equalization” mode of operation and suddenly a rain event is eminent or flows increase rapidly, the equalization system can be manually (or automatically) switched to the “Peak Flow“ mode of operation.

As part of the daily equalization tank, an excess flow pumping station will be provided to return the flows back to the treatment process or divert them to the Wet Weather Side Stream Treatment process. This excess flow pump station will have automatic controls with preset pumping ranges for each selected mode of operation.

7.3.5. Wet Weather Side Stream Treatment

Wet Weather Side Stream treatment (sometimes referred to as Peak Flow Treatment) is a new approach available to EPA Region 7 wastewater facilities to treat peak flows under extreme weather conditions. A guidance document entitled “Key Principles and Consideration Factors for Incorporation on Non-Biological Peak Flow Processing Approaches in Iowa Wastewater Facilities” has been developed for IDNR review. A copy of this guidance document is included in Appendix A of this document.

Indianola’s range of peak flows to average flows is excessive. The City is committed to continue to make improvements to the collection system and within the City to reduce I/I and minimize sanitary sewer overflow (SSOs) events.

Peak flow treatment technologies are developing at a fast rate as the pressure to eliminate SSOs from peak flow events occurs. Particularly in EPA Region 7 states where peak flow treatment may be considered as an acceptable alternative for peak flows. Generally the technologies are physical treatment focusing on removing suspended solids to produce a low cBOD and TSS effluent. A coagulant is frequently added where removing phosphorus is required. For this report two wet weather side stream treatment technologies were considered 1) Ballasted flocculation system – Actiflo, and 2) effluent filtration system – Aqua Prime. A final selection of Wet Weather Side Stream treatment technology will be completed during final design. Details of these two wet weather side stream treatment technologies are included in Appendix G.

This Alternative P2 for preliminary treatment includes a 10 mgd ballasted flocculation peak flow treatment system (such as Actiflo). The peak flow treatment system will be started up during extreme weather events to provide physical treatment to the remaining flows above the treatment plant’s secondary treatment capacity.

The Actiflo process (manufactured by Kruger) is a high rate, compact process for peak flow treatment. The process operates with microsand which enhances floc formation and acts as a ballast to aid in rapid settlement of coagulated material. The microsand ballasted flocs display unique settling characteristics, which allow for clarifier designs with very high overflow rates and short retention times. The Actiflo system design for peak flow treatment results in footprints that are a fraction of the size of conventional clarifier systems. Actiflo is an approved technology by the US EPA for peak flow treatment. An Actiflo peak flow treatment process can be started-up and ready for processing in less than 15 minutes.

7.3.6. Benefits and Disadvantages of Preliminary Alternative P2

Benefits of Preliminary Treatment alternative P2

- All wastewater treatment facilities are on the Farm Site.
  - ✓ Easier to operate/maintain and control access.
  - ✓ Re-purpose of existing site is possible.
  - ✓ Reduced pumping energy needed.
- No large equalization basin is necessary.
- Better opportunity to separate wastewater treatment facilities from the public at larger Farm Site.
- Concept of Peak Flow Treatment has benefits;
  - ✓ Get thru peak flow event quickly and get back to normal operation.
  - ✓ Protect secondary treatment system from peak flow upsets.

Disadvantages of Preliminary Treatment alternative P2

- Peak Flow Treatment design is new to IDNR and may take significant effort to gain approval.

7.3.7. Alternative P2 – Opinion of Cost

A preliminary Opinion of Probable Construction Cost for alternative P2 is included in Table 7-2.

7.4. STORE AND TREAT VS. WET WEATHER SIDE STREAM TREATMENT

HR Green completed an analysis comparing two different strategies for handling wet weather peak flows for the City of Indianola as discussed in this report. "Store and Treat" is the practice of shaving off the peak flows above the WWTP capacity and diverting the excess flow to equalization then bring that flow back for treatment through the WWTP as the peak flows subside. This practice for treatment of peak flows has been used for ages in Iowa. An alternative practice now gaining some attention is Wet Weather Side Stream treatment of flows above the WWTP's secondary treatment capacity and then blending the flow from the side stream with the secondary treatment effluent. Depending on the nature of the peak flows to the WWTP, this alternative may be best suited for the community. A Technical Memorandum comparing these alternatives for Indianola is included in Appendix H.

In this report the recommended alternative for treatment of peak flows is Wet Weather Side Stream treatment. A summary of the deciding factors that led the decision to the selected alternative are:

- *The addition of denitrification for Total Nitrogen removal requires a high level of stability in the biological process. The dilute peak flows that are common in the Indianola influent waste stream would make this process less stable. So removing the peak dilute flows from secondary treatment help the stability of the denitrification process.*
- *The effluent quality of the side stream treatment selected is very good and based on the Biowin modeling for a variety of operating scenarios (see Appendix X) the blended effluent meets all the effluent requirements for the Middle River receiving stream.*
- *The City has decided to abandon the existing NWWTF site including the existing 27 million gallon equalization basin. The estimated construction cost to construct a new 18 million gallon equalization basin at the Farm Site is significantly more expensive than providing 10 mgd of wet weather side stream treatment.*
- *The Store and Treat process can lose temperature while stored or grow algae. Either of these conditions makes secondary treatment more difficult.*
- *In the Store and Treat mode, during the time immediately following a design peak flow event, the system is more susceptible to have an SSO occurring.*

Table 7-2 Alternative P2 – Conceptual Opinion of Probable Const. Cost

Item	Description	Cost
Sitework	Sitework only related to alternative	
Sanitary Sewer w/manholes	approx 11,000 lin ft	$3,600,000
Yard Piping		$250,000
Headworks Building (1)	Influent screening and pumping station	
Screening Building	30x30 building	$260,000
Raw Wastewater PS Building	Self cleaning wetwell type	$280,000
Mechanical Screens		$300,000
Slide Gates		$80,000
Raw Wastewater Pumps	Vertical turbine solids handling	$320,000
Piping and valves		$200,000
Mechanical/Plumbing		$60,000
Electrical/Controls		$80,000
Excess Flow Pump Station		
Structure (submersible)	Submersible PS	$80,000
Pumps, piping and valves		$75,000
Electrical/Controls		$20,000
Grit Removal System		
Grit Building and structure (1)		$300,000
Vortex Grit System		$200,000
Grit pumps, piping and valves		$100,000
Slide gates		$20,000
Mechanical/Plumbing		$60,000
Electrical/Controls		$100,000
Wet Weather Side Stream Treatment		
Package Equipment	Actiflo system	$800,000
Enclosure/Structure (1)		$400,000
Mechanical/Plumbing		$80,000
Electrical/Controls		$120,000
Daily Equalization Tank		
Prestressed Tank (1)		$1,200,000
Mixers		$80,000
Piping and valves		$20,000
Electrical/Controls		$20,000
	Total Alternative P2 Opinion of Construction Cost (2,3)	**$9,105,000**

(1) Includes concrete, excavation, backfill, superstructure, etc.
(2) Costs in Table do not include sitework, land acquisition, contractor overhead, demolition of old site, engineering or contingency
(3) Based on ENR Building Cost Index 5563 (Nov 2015)

8. SECONDARY TREATMENT ALTERNATIVES

8.1. GENERAL

The secondary treatment process is the heart and soul of the wastewater treatment facility. Secondary treatment includes the biological systems required to reduce organic and nutrient concentrations to levels that can be safely discharged to the receiving stream without adverse impacts on water quality or elevated risks to human health. Therefore, design and operation of the secondary treatment process must focus on providing the environment and conditions necessary to maintain a healthy population of target microorganisms under a wide range of influent flows, loadings and operating temperatures.

In addition, the secondary treatment process must be flexible and provide professional operating staff with the ability to make process adjustments as needed to accommodate changes in wastewater characteristics or as necessary to meet more restrictive effluent treatment targets developed during the life of the wastewater treatment facility. Proper selection and operation of the secondary treatment system is essential for meeting performance requirements as described in the City's National Pollutant Discharge Elimination System (NPDES) permits as issued by the Iowa Department of Natural Resources (IDNR), which regulates wastewater discharges to lakes, streams, wetlands and other surface waters under the jurisdiction of the U.S. Environmental Protection Agency.

8.1.1. Iowa Nutrient Reduction Strategy

The Iowa Nutrient Reduction Strategy will apply to this project. The strategy is a technology-based approach to reducing nutrients delivered to Iowa's waterways. As with most other communities in Iowa, the City of Indianola currently does not have restrictions on the amount of total nitrogen and phosphorus that can be discharged to the receiving stream. Under the Iowa Nutrient Reduction Strategy, technology-based limits will be implemented as part of renewing a facility's NPDES permit. Nutrient limits will be no more stringent than 10 mg/l for total nitrogen and 1 mg/l for total phosphorus.

Requirements for evaluating nutrient reduction potential at Indianola's Water Pollution Control Facility are expected to be specified in the next NPDES permit cycle. Implementation of a nutrient reduction program, which is consistent with the Iowa Nutrient Reduction Strategy, most likely will be required under the subsequent NPDES permit issued by the IDNR. Therefore, this Facility Plan evaluation assumes that future treatment facilities will be required to reduce total nitrogen and phosphorus discharges to technology-based levels.

Of particular note, after nutrient reduction systems are installed in Indianola's wastewater treatment plant, the City will be protected from stricter limits for at least 10 years.

8.1.2. Biological Nutrient Reduction

In issuing the Iowa Nutrient Reduction Strategy, IDNR stated the following:

> *"Although continuously evolving, many nutrient removal technologies in wastewater treatment are already proven and well-established. Thus, nutrient removal for Iowa's wastewater treatment facilities is technologically feasible."*

In addition, biological nutrient reduction is described as…

> *"...commonly associated with sequenced combinations of aerobic, anoxic and anaerobic processes which facilitate biological denitrification via conversion of nitrate to nitrogen gas and "luxury" uptake of phosphorus by biomass with subsequent removal through wasting of sludge (biomass)."*

An explanation of terms and processes may be helpful. Figure 8-1 provides schematic representations of the various BNR processes, which are summarized as follows:

- Aerobic or oxic activated sludge processes (Schematic (a)) are those in which biological growth is managed by controlling the oxygen concentration and recycling flows, such as return activated sludge (RAS) and mixed-liquor recycle (MLR), to a reactor. The wastewater's oxygen concentration is kept near or above 2.0 mg/L, because nitrification declines when dissolved oxygen concentrations drop below 0.5 mg/L.

- Anoxic zones or conditions (Schematic (b)) are those in which the aerators in that area are shut off. Little dissolved oxygen is present (less than 0.5 mg/L) in this zone, but chemically bound oxygen (in the form of nitrite and nitrate) may be present in RAS or MLR flow.

- Anaerobic zones or conditions (Schematic (c)) contain neither dissolved oxygen nor chemically bound oxygen. They are typically created by sending MLR to denitrification selector cells rather than to the head of the anaerobic zone, which would increase chemically bound oxygen levels too much. Sometimes a supplemental source of carbon is necessary to ensure that dissolved and chemically bound oxygen are rapidly removed.

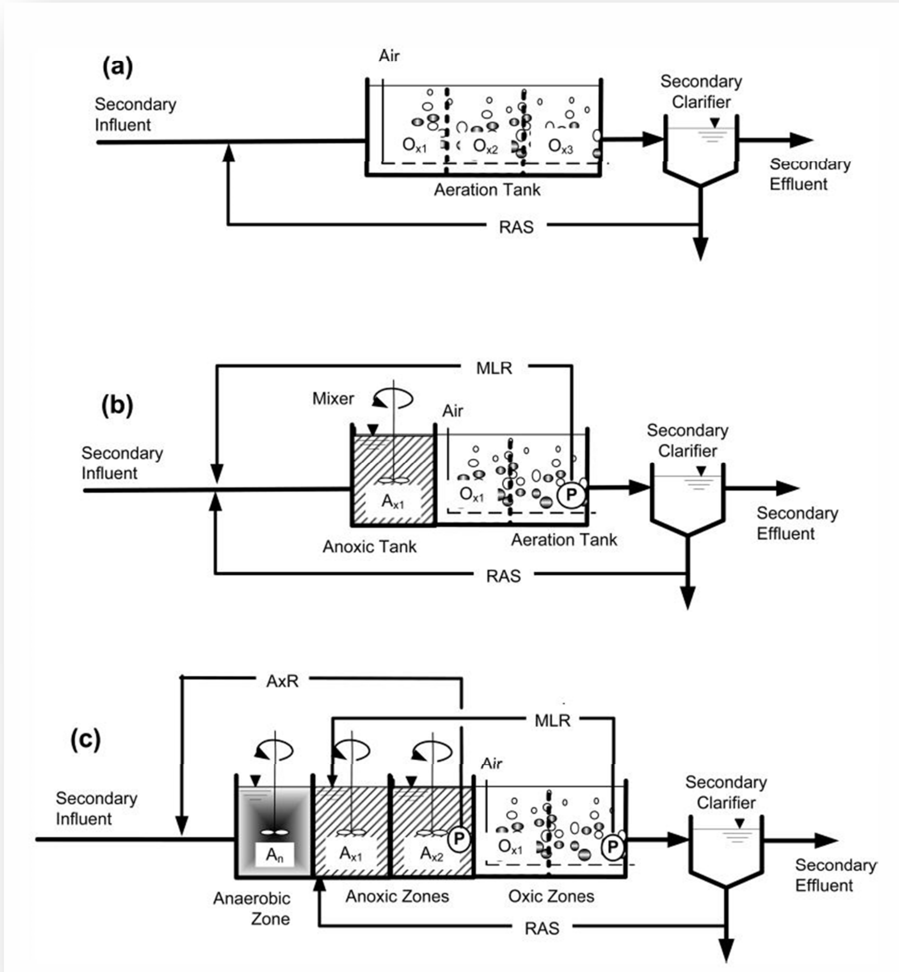


Figure 8-1 Schematic of BNR Processes

Of particular note in the evaluation of secondary treatment alternatives for Indianola are the following key parameters:

- Accurate control of dissolved oxygen concentrations in the various tanks or operating zones necessary to create conditions necessary for aerobic, anoxic and anaerobic activity.
- Accurate monitoring and control of recycle streams from secondary clarifiers, aerobic "activated sludge" basins and anoxic selector tanks.
- In the case of biological phosphorus reduction as represented by Schematic (c) above, when influent wastewater offers a relatively-low carbon source (e.g., low BOD concentrations when diluted by peak flow events), supplemental carbon feed in the form of ethanol, methanol, high sugar wastewater, or other commercial or waste product is required to facilitate the "luxury uptake" process.

IDNR has described the biological nutrient reduction process as technologically feasible, but it's important to note that effective implementation largely depends on the characteristics of influent wastewater at the facility.

8.1.3. Indianola Wastewater Flows and Loadings

Design wastewater flows and characteristics were previously addressed in Section 5.3, but it's important to note that the Indianola WPCF receives a wide range of flows and loadings at the treatment facilities. In general, secondary treatment facilities are most efficient when the ratio of maximum day to average day flow is 3:1 or less. In the case of Indianola, that ratio is greater than 3:1, which represents periods of high flow rates that dilute the wastewater strength. When designing for high flow rates, tanks, piping and pumping equipment must be upsized to minimize the risk of surcharging or overflow. But when operating a facility with diluted wastewater strength, it becomes difficult to consistently maintain the conditions necessary to achieve biological nutrient reduction.

It's also important to note that this Facility Plan was developed with an assumed 20-year planning period, and therefore, includes allowances for additional flows and loadings associated with expected economic growth and minor industrial development. Predicting the speed at which this economic development occurs is outside the expertise of engineers. Considering that industrial flows in the City of Indianola will be gradually developed, the secondary treatment facilities will be designed with flexibility to accommodate the loadings either with or without industrial contribution. Total design flows and loads under both conditions are listed in Table 5-2.

However, in evaluating secondary treatment alternatives, we have considered potential flow and loading conditions that may be expected at the time of start-up.

8.1.4. Iowa DNR Design and Permitting Requirements

Current design and permitting requirements as published by the Iowa DNR for secondary treatment systems are partially based on the *Recommended Standards for Wastewater Facilities* as published by the Great Lakes -- Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers, which is commonly referred to as the "Ten States Standards." In preparing this facility plan, other IDNR documents were also referenced, including *A Regulatory Guide to Sequencing Batch Reactors*, which has established unique criteria for design and permitting of facilities that utilize the sequencing batch reactor process for secondary treatment and nutrient reduction.

Of particular interest in preparing this Facility Plan are the various interpretations and applications of IDNR's requirements for secondary treatment. Chapter 18B of the Iowa Wastewater Facilities Design Standards was adopted in 1984 and is primary regulatory standard for Activated Sludge Biological Treatment. More specifically, Table 1 is entitled, "Typical Aeration Tank Loadings and Design Parameters" and summarizes the design requirement for several categories of activated sludge treatment processes.

As mentioned the IDNR design standards cover in detail the carbonaceous BOD and ammonia removal by nitrification in secondary treatment. The existing design standards for secondary treatment are not as detailed for denitrification for total Nitrogen removal and for biological or chemical phosphorus removal as planned by this WWTP design. The design of a WWTP to meet the Iowa Nutrient Strategy is generally not covered by the current design standards.

To address the portion of the secondary treatment design for nutrient removal that might not meet the current design standards we plan to request variances from the IDNR as needed. A general variance request letter has been sent to the IDNR regarding some of the requested variances that might be needed for the secondary treatment design.

Sequencing Batch Reactor Process:

As stated in the document entitled *A Regulatory Guide to Sequencing Batch Reactors*, “SBRs should be similar to other conventional and extended aeration processes.” In particular, the design F:M ratio for domestic wastewater is specified as 0.05 to 0.10, which corresponds to the process criteria for “Extended Aeration” systems as listed in Table 1 of Chapter 18B. For extended aeration systems, Table 1 also specifies a solids retention time (SRT) of 20 – 30 days and a Mixed Liquor Suspended Solids concentration of 3,000 – 5,000 mg/l.

Although biology within a sequencing batch reactor is similar when operated for carbon reduction and ammonia nitrification, the design/permitting requirements place the process at a competitive disadvantage when compared with other activated sludge processes. The process can be adjusted to operate to remove total Nitrogen as well.

Oxidation Ditch Process:

Table 1 of Chapter 18B identifies an activated sludge process categorized as “Combined Carbon Oxidation – Nitrification.” In summary, this process describes secondary treatment systems that have primary effluent targets for BOD/cBOD and Ammonia. “Carbon Oxidation” is the biological process for reducing organic waste load, which for performance and compliance purposes is measured as Biochemical Oxygen Demand (BOD) or Carbonaceous Biochemical Oxygen Demand (cBOD). “Nitrification” is the biological process of converting potentially toxic ammonia into nitrate.

Under the current permitting requirements, an oxidation ditch process designed for BOD/cBOD and Ammonia reduction is given less-conservative design criteria. As with an SBR process, the Maximum Aeration Tank Organic Load is 15 lbs. BOD5 per day /1,000 cft. of reactor volume. However, allowable F:M ratio is increased to 0.08 – 0.16, the MLSS design concentration is reduced to 2,000 – 5,000 mg/l and the SRT is also reduced to 15 – 25 days.

When sizing tank volumes and process equipment, this difference in design criteria is advantageous for the oxidation ditch process. The oxidation ditch can be designed easily with anoxic and anaerobic zones to make it practical also for total Nitrogen and phosphorus removal.

<u>MLE Activated Sludge Process:</u>

As described in a later section of this Facility Plan, the Modified Ludzak-Ettinger (MLE) Activated Sludge process is simply a two-stage secondary treatment system that can be employed to biologically achieve Total Nitrogen reduction. A separate Anoxic basin is used to create conditions where there is no available dissolved oxygen, which encourages microorganisms to break down the nitrate molecules into oxygen and nitrogen gas. The nitrogen gas is released back into the atmosphere, thereby resulting in a Total-Nitrogen reduction through the wastewater treatment system.

However for sizing the Aerobic (oxygen-rich) Basins, we understand that the design and permitting criteria for “Combined Carbon Oxidation – Nitrification” as listed in Table 1 of Chapter 18B applies similarly to an Oxidation Ditch Process.

8.1.5. Process Evaluation Workshop

During early stages of the planning project, a Process Workshop was held that identified several secondary treatment processes for preliminary selection by City staff. These alternatives were discussed in great detail during this workshop and narrowed down based on ability to meet nutrient removal goals, operation and maintenance, capital cost, flexibility with future permit, regulatory acceptance, and ability to handle extreme flow range. A matrix was completed by the attendees of the workshop to document the planning direction.

From this workshop the preferred secondary treatment approach was for removal of Total Nitrogen through biological nitrification and denitrification processes followed by chemical phosphorus removal.

The secondary treatment processes specifically selected for further evaluation were oxidation ditches, MLE activated sludge, and sequencing batch reactors.

8.1.6. Strategies for Secondary Treatment Evaluations

One of the strategies used for the secondary treatment process with biological nutrient removal is to limit flow variations through the process to maintain consistent and reliable treatment without excessive operational attention. The denitrification process is much more susceptible to process upsets based on changing influent conditions. For the Indianola wastewater treatment plant several concepts were proposed that support this strategy:

- Size the secondary treatment process for flows just higher than average wet weather (AWW) flows. Flows above secondary treatment capacity during peak events will be diverted to Wet Weather Side Stream treatment and then blended with secondary treated flows prior to disinfection and discharge.

- Break the secondary treatment into treatment trains, where one treatment train can be shut down if the flow range doesn't support it.
- Include the capability to equalize the daily diurnal peak flows to treat an operator selected daily average flow.

8.1.7. Secondary Treatment Alternatives

Three options for secondary treatment will be considered and evaluated for the new Indianola wastewater treatment facilities; 1) Oxidation ditch with final clarifier; 2) MLE activated sludge including reactor tank and final clarifier; and 3) Sequencing batch reactors (SBRs). Ultraviolet (UV) disinfection will be used for disinfection for each of the secondary treatment options. The remaining portion of this section provides a detailed evaluation of these alternatives.

8.2. ALTERNATIVE ST1 – OXIDATION DITCHES WITH FINAL CLARIFIERS FOLLOWED BY UV DISINFECTION

This alternative for secondary treatment ST1 consists of cast-in-place concrete oxidation ditches (reactors) followed by three cast-in-place concrete circular final clarifiers. Effluent from the two stage(anoxic and aerobic) oxidation ditch secondary treatment process will be disinfected by UV disinfection. A concrete flow splitter ahead of the oxidation ditches and a second concrete flow splitter ahead of the final clarifiers are also included. The final number of oxidation ditches will either be two or three depending on the final design layout and required flexibility.

8.2.1. Oxidation Ditch Reactors

The cast-in-place concrete oxidation ditches will serve as reactor tanks for total nitrogen removal. Sizing for the oxidations ditches is driven by biological treatment requirements.

<u>Aerobic/Nitrification</u>. The aerobic volume is specified by IDNR and "10 States Standards" for extended aeration activated sludge system based on a maximum organic loading of 15 ppd BOD / 1,000 cft of aerobic reactor volume. Using the Maximum Month BOD loading of 4,707 ppd, the minimum aeration volume is 2,250,000 gallons. At an Annual Average flow rate of 2.91 mgd, the equivalent Hydraulic Retention Time is approximately 19.4 hours.

Figure 8-2 Oxidation Ditch Aerator

Anoxic/Dentritication. The anoxic zone for denitrification is determined based on estimated denitrification rates for the microorganisms. In practice the denitrification rate is influenced by a wide range of variables. However for conceptual sizing, the expected volume is estimate to be 650,000 gallons based on an HRT of 2.75 hours.

Total volume for the oxidation ditches is estimated to be around 3,000,000 gallons. Side water depth will be verified during design but is expected to be approximately 12 feet, depending on the type of mixer selected and the size of the impeller. Detail for a proposed two-stage Oxidation Ditch by WesTech is included in Appendix I.

8.2.2. Final Clarifiers

Mixed liquor leaving the oxidation ditches are routed through final clarifiers where microorganisms settle to the bottom of the structures and clear supernatant at the top water surface flows over finger weirs before being piped to the UV disinfection system. Settled microorganisms are either returned to the oxidation ditches as “return activated sludge (RAS)” or wasted to the solids processing facilities as “waste activated sludge (WAS)”.

Sizing for the final clarifiers is generally based on four criteria:

- Surface Overflow Rate: ≤ 1,000 gpd/sft at PHWW flow
- Solids Loading Rate: ≤ 30 ppd MLSS at AWW flow
- Solids Loading Rate: ≤ 50 ppd MLSS @ PHWW flow

- IDNR Reliability Criteria: provide ≥ 75% design load capacity with largest unit out of service.

For this application, three cast-in-place concrete 60-ft diameter clarifiers with a 14 ft. side water depth will be provided. The final clarifiers for this alternative would be the same as for the MLE activated sludge option. See paragraph 8.3.2.

Figure 8-3 Oxidation Ditch with Clarifiers

Ferric chloride or aluminum sulfate (alum) can be fed at the flow split structure for the final clarifiers in the secondary process to chemically precipitate a portion of the soluble phosphorus. Additional evaluations will be completed during the design portion of the project to determine the most appropriate feed points and dosages. The ferric addition to the final clarifiers will enhance settling of microorganisms.

8.2.3. Ultraviolet (UV) Disinfection

Treated secondary treatment effluent from the oxidation ditch process will pass through a UV disinfection channel prior to final discharge to the receiving stream. The UV disinfection system is described in more detail in Section 8.5.

8.2.4. Benefits and Disadvantages of Secondary Treatment Alternative ST1

<u>Benefits of Secondary Treatment alternative ST1</u>

- Oxidation ditch process is a proven and reliable secondary treatment process for biological reduction of organic matter and ammonia-nitrogen.

- The large aerobic volumes required under IDNR standards make the system less susceptible to shock loads or toxic conditions that may come to the wastewater treatment plant.
- If mixing and aeration can be controlled, simultaneous nitrification and denitrification can occur in the oxidation ditch without a selector basin.
- Mixing/aeration equipment is relatively easy to maintain and service, although a crane would be required for major repairs.

Disadvantages of Secondary Treatment alternative ST1

- Control of aeration rates and dissolved oxygen concentrations are difficult to control accurately throughout the basin.
- For systems that reduce the speed of the aerators as a method of reducing aeration rates, flow velocities within the ditches can decrease to the point were mixed liquor begins to settle out and accumulate in the basins.
- Basin depths are typically shallower than other secondary treatment option, which translates into a larger footprint and higher heat loss during winter months.

8.2.5. Alternative ST1 – Opinion of Cost

A preliminary Opinion of Probable Construction Cost for alternative ST1 is included in Table 8-1.

Table 8-1 Alterative ST1 – Conceptual Opinion of Probable Construction Cost

Item	Description	Cost
Sitework	Sitework only related to alternative	
Yard Piping		$150,000
Influent Flow Splitter (1)	Low head FS	$50,000
Oxidation Ditch - MLE		
Oxidation Ditch Tanks (1)	3 tanks at 3.0 MG	$3,900,000
Oxidation Ditch Equipment	Aerator, submersible mixers, gates	$1,200,000
Secondary Flow Splitter (1)	Low head FS	$60,000
Secondary Clarifiers		
Secondary Clarifier tanks (1)	60 ft diameter x 12 ft SWD	$835,000
Clarifier Equipment	Center feed, Spiral collectors	$384,000
Secondary Treatment Building		
Building/Structure (1)	4,000 sq ft with basement	$800,000
RAS Pumps	4 at 3 mgd each	$88,000
WAS Pumps	3 at 100 gpm each	$29,000
RAS/WAS Piping and Valves		$190,000
Mechanical/Plumbing	for entire building	$160,000
Electrical/Controls	Aerator drives, and for building	$280,000
Laboratory	Equipment and furniture	
Locker Rooms	Furniture	
Effluent Water System	(included elsewhere)	
Carbon Feed System	Storage tank, pumps, piping	$70,000
Iron Salt Feed System	Storage tank, pumps, piping	$100,000
UV Disinfection - 8 mgd		
Channel/structure (1)		$112,000
UV Equipment	Vertical or horizontal w/ finger weirs	$250,000
Slide gates		$8,000
Mechanical/Electrical		$25,000
	Total Alternative ST1 Opinion of Construction Cost (2,3)	**$8,691,000**

(1) Includes concrete, excavation, backfill, superstructure, etc.
(2) Costs in Table do not include deep foundations, contractor overhead, engineering or contingency
(3) Based on ENR Building Cost Index 5563 (Nov 2015)

8.3. ALTERNATIVE ST2 – MLE ACTIVATED SLUDGE PROCESS INCLUDING FINAL CLARIFIERS FOLLOWED BY UV DISINFECTION

The Modified Ludzack-Ettinger Process (MLE) is a modification of a conventional activated sludge process where an anoxic zone is created or added upstream of the aerobic zone. The process uses an internal recycle that carries nitrates created in the nitrification process in the aerobic zone along with the mixed liquor to the front of the anoxic zone. Under proper conditions, microorganisms strip oxygen from the nitrate molecules. The result is formation of nitrogen gas bubbles to the top of the water surface and dissipates back into the atmosphere. The amount of nitrates potentially removed in the anoxic zone depends on the recycle flow and availability of influent BOD. If BOD concentrations are not sufficient, a supplemental carbon source may be required to support the denitrification process.

This alternative for secondary treatment ST2 consists of three cast-in-place concrete reactor tanks followed by three cast-in-place concrete circular final clarifiers. Effluent from the MLE activated sludge treatment process will be disinfected by UV disinfection. A concrete flow splitter ahead of the reactor tanks and a second concrete flow splitter ahead of the final clarifiers are also included.

8.3.1. Reactor Tanks

In conventional activated sludge an aeration tank is provided to maintain a population of biological organisms. The activated sludge process uses a suspension of flocculant microorganisms composed of bacteria, fungi, protozoa, and rotifers to remove biologically degradable organic compounds (e.g. BOD) from the wastewater. The organisms are then settled in secondary clarifiers and returned to the aeration tank to provide the concentration of organisms targeted. Many different activated sludge configurations can be used to accomplish treatment. Each configuration has its special application. The activated sludge configuration chosen for Indianola shall provide removal capabilities for BOD, ammonia and nitrogen. The process will complete staged nitrification/denitrification in one tank with separated specific zones to create the environment desired. The process is called the Modified Ludzack-Ettinger (MLE) process. A simplified flow schematic is shown below.

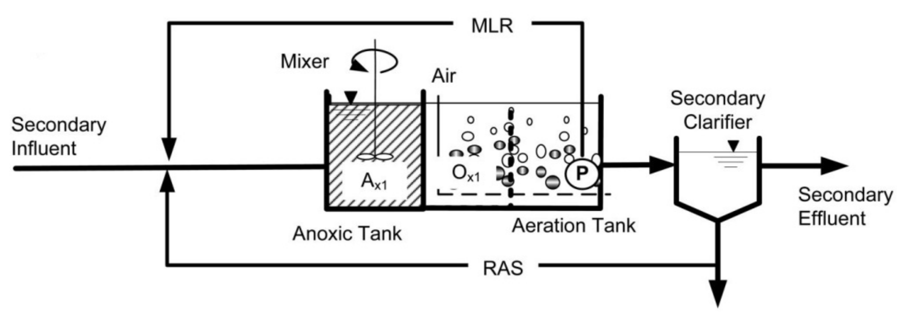


Figure 8-4 Modified Ludzack-Ettinger (MLE) Process

Aerobic Zone. The aerobic zone would complete the majority of the BOD and ammonia removal (nitrification). These processes require air to provide the BOD uptake and the conversion of ammonia to nitrate. Longer solids retention times (SRTs) are needed to establish microorganisms in the aeration tanks to remove ammonia. SRT is the amount of time that a microorganism remains in the system to grow and thrive. The relative age corresponds to the level of treatment that the organism can accomplish. Microorganism growth is dependent on many factors (temperature, pH, dissolved oxygen, etc.). At warmer temperatures organisms will grow faster than at lower temperatures. So an organism grown at 20 degrees Celsius (C) for 5 days may be able to accomplish the same level of treatment as an organism aged for 12 days at 10 degrees C. A preliminary SRT of 12-days will be used to achieve nitrification at future design flows and loads for a design temperature of 10 degree C.

Fine bubble membrane diffusers are recommended due to high oxygen transfer efficiency and advances in technology allowing for longer service life. Oxygen would be supplied based on the following ratios 1.1 lb oxygen/lb BOD removed and 4.6 lb oxygen/lb TKN removed based on the projected future flows and loadings. This aeration would be provided by new positive displacement (PD) blowers. To provide for redundancy three blowers shall be sized to be able to supply the 3,523 scfm with one additional blower for standby. The blowers will be housed in an enclosure or other structure. Variable frequency drives (VFDs) will be used to control the blowers based on oxygen needs to the system.

Figure 8-5 Aerobic Zone Photo

Anoxic Zone. The anoxic zone will provide conversion of the nitrates in the RAS flows or recycle flows to nitrogen gas. This is the removal pathway for nitrogen. A carbon source is needed for this conversion. The anoxic tank is located at the front of the reactor tanks to allow the influent wastewater flow to provide the carbon source. If the BOD/TKN ratio (recommended TKN/BOD >4) is low then a supplemental carbon source may be needed routinely. Recycle ratios of 2-3 x Q are typical.

Figure 8-6 Photo of Recycle Pump Installation

Anoxic tank size can be reduced by including multiple stages in series. Also, multiple stages would be used at the influent end of each reactor tank to provide for filamentous control in the aeration tanks and will also help to increase the settling properties of the activated sludge. Mixing will be included to keep solids in suspension and to create good food to microorganism contact.

The three cast-in–place reactor tanks will be tanks 60 ft. x 155 ft. by 15 ft. deep each. Tanks will be constructed with common walls. Each tank will include an anoxic zone with volume of approximately 10% of the entire tank volume at the front end, a swing zone in the middle of approximately 20% and 70% volume of aerobic zone. Each of the zones will be separated by baffle walls. The anoxic and swing zones will be mixed with mechanical mixers and diffused aeration equipment will distribute fine bubble air supply to the swing and aerobic zones.

Advantages of MLE.

- Saves energy; BOD is removed in the anoxic zone without the use of air.
- Alkalinity is produced
- Better settling characteristics
- Targeted for 5-8 mg/L effluent total nitrogen.

Limitations-

- DO needs to be controlled to limit recycle DO
- Recycle rates can be high(pumping energy).

Aeration piping to the basin from the blowers will be either light wall steel or ductile iron pipe (DIP) outside the tank and light wall stainless steel within the tank.

A flow splitter will be used to equally split flow to the reactor tanks. Stop plates or slide gates will be used to isolate tanks from service. The flow splitter will also receive the return sludge pumped back from the secondary clarifiers and the recycle flow.

8.3.2. Final Clarifiers

Final clarifiers are required with activated sludge to settle the microorganisms from the mixed liquor exiting the aeration tanks. The settled mixed liquor is then returned back to the aeration tanks to maintain a targeted ratio. The sludge flow returned is termed return activated sludge (RAS). Final clarifiers sizing is based on solids loading rate (SLR) and overflow rate. Using 6.0 MGD and 4,000 mg/l MLSS concentration as design conditions, three clarifiers will be needed, and each of them is designed to be 60 feet in diameter and 14 feet deep.

The final clarifiers will serve as a feed point for iron salts added for the chemical precipitation of phosphorus. A secondary iron salt feed point will be in the aeration basins. *The final clarifiers will have better settling capabilities due to the dual purpose of chemical precipitation. The current IDNR design standards don't cover this design consideration. Variances will be requested as needed where the final clarifiers don't meet the IDNR design standards.*

The new clarifiers would utilize a clarifier optimization package that incorporates center-feed technology and peripheral draw. The clarifier optimization package includes a center column, energy dissipating inlet (EDI), flocculating feed well (FFW), spiral scrapers, scum removal system, current baffling, and a sludge drum. The center column, EDI, and FFW are designed to minimize floc breakup and optimize settling performance. The current baffling is designed to minimize solids scouring during high flow periods. The spiral scrapers effectively and efficiently transport sludge to the sludge hopper for withdrawal.

The new clarifier's hydraulic and loading parameters are listed in Table 8-2. As can be seen, the clarifiers will be under loaded based on solids and hydraulics. There may be times during the year that aeration tanks and clarifiers may be taken offline.

Table 8-2 Indianola Wastewater Treatment Plant Improvements Secondary Clarifier Hydraulics and Loadings

	Future Avg	Future MD
Flow, MGD	2.91	6.0
RAS, MGD	1.2	4.8
RSS, mg/l	9,000	9,000
MLSS, mg/l	2,500	4,000
Clarifiers		
Quantity	3	3
Diameter, ft	60	60
Area each, SF	2,827	2,827
SWD, ft.	14	14
OFR, gpd/SF.	343	707
Floor Slope, ft/ft	1/12	1/12
SLR, lb/SF./d	11.4	47.8
Volume, cu ft.	118,734	118,734
, gal	888,192	888,192
Detention time, hrs.	5.2	2.0

A flow splitter will be used to divert mixed liquor suspended solids (MLSS) equally to the clarifiers. Stop plates or slide gates will be used to isolate clarifiers from service for maintenance or low flow situations.

A RAS pump station will be required to pump the sludge off the bottom of the clarifier back to the secondary treatment flow splitter. The RAS pumping facilities will be sized to pump 100% of the average flow or the required RAS flow for 6.0 MGD. The design pumping rate will be 6.0 mgd, firm capacity. The structure will be configured with slide gates on the pipes from each clarifier sludge hopper. The slide gates will modulate the proportioning of the sludge from each clarifier into the wetwell. The RAS pumps will pump from the wetwell back to the secondary treatment flow splitter. Locations shall be provided for RAS pumps to be added in the future. A waste activated sludge (WAS) pump will pump WAS to the solids treatment process.

Solids loading calculations are as follows:

Max Day Solids Loading Rate

At PHWW flow plus max day loadings the required RAS flow rate to sustain 4,000 mg/l MLSS at 6.0 mgd influent is 4.8 mgd, therefore:

=[(0.75)(4,000mg/l)(8.34)(6.0mgd + 4.8mgd)]/[(2)(3.14/4)(60 ft)(60 ft)] = 48 ppd/sf < 50 ppd/sf OK

Max Month Solids Loading Rate

At AWW flow plus max month loadings the required RAS flow rate for the max month condition is 40% of the influent Q (2.4 mgd) to sustain a MLSS concentration of 3,000 mg/l

=[(0.75)(3,000mg/l)(8.34)(6.0mgd + 2.4mgd)]/[(2)(3.14/4)(60 ft)(60 ft)] = 28 ppd/sf < 30 ppd/sf OK

8.3.3. Ultraviolet (UV) Disinfection

Treated secondary treatment effluent from the MLE activated sludge process will pass through a UV disinfection channel prior to final discharge to the receiving stream. The UV disinfection system is described in more detail in Section 8.5.

8.3.4. Benefits and Disadvantages of Secondary Treatment Alternative ST2

<u>Benefits of Secondary Treatment alternative ST2</u>

- Conventional activated sludge process is a flexible, reliable treatment process familiar to the City operations staff.
- MLE modifications for adding an anoxic selector tank to a conventional activated sludge process should be a relatively easy transition from current operations.
- The MLE process is not patented and, therefore, does not depend on propriety process equipment furnished through a particular manufacturer.
- All process variables including aeration rates, recycle flows, sludge wasting, dissolve oxygen monitoring and ORP control can be automated and customized to the preferences of operating staff.
- Process is flexible and will accommodate future expansion. Addition of an anaerobic selector basin for biological phosphorus reduction can be added at a later date if found to be beneficial or cost effective.

<u>Disadvantages of Secondary Treatment alternative ST2</u>

- Most equipment-intensive of the alternatives. Long term operation and maintenance costs would be expected to be higher.
- Process controls are custom-developed for the application, which will require operating staff to make manual programing tweaks and changes as operating experience develops.

8.3.5. Alternative ST2 – Opinion of Cost

A preliminary Opinion of Probable Construction Cost for alternative ST2 is included in Table 8-3.

Table 8-3 Alterative ST2 – Conceptual Opinion of Probable Construction Cost

Item	Description	Cost
Sitework	Sitework only related to alternative	
Yard Piping		$150,000
Influent Flow Splitter (1)	Low head FS	$50,000
MLE Reactor Tanks		
Activated Sludge Tanks (1)	3 tanks at 155 x 60 x 15 ft deep	$3,800,000
Aeration Blowers	4 at 1,450 scfm, outside in enclosures	$260,000
Fine bubble diffused aeration system		$270,000
Blower piping and supports		$182,000
Anoxic mixer	1 per anoxic zone, 3 total	$80,000
Secondary Flow Splitter (1)	Low head FS	$60,000
Secondary Clarifiers		
Secondary Clarifier tanks (1)	60 ft diameter x 14 ft SWD	$870,000
Clarifier Equipment	Center feed, Spiral collectors	$384,000
Secondary Treatment Building		
Building/Structure (1)	4,000 sq ft with basement	$800,000
Recycle Pumps	3 pumps in basin	$60,000
Recycle piping and valves		$120,000
RAS Pumps	4 at 3 mgd each	$88,000
WAS Pumps	2 at 100 gpm each	$29,000
RAS/WAS Piping and Valves		$190,000
Mechanical/Plumbing	for entire building	$160,000
Electrical/Controls	Drives, and for building	$360,000
Effluent Water System	(included elsewhere)	
Carbon Feed System	Storage tank, pumps, piping	$70,000
Iron Salt Feed System	Storage tank, pumps, piping	$100,000
UV Disinfection - 8 mgd		
Channel/structure (1)		$112,000
UV Equipment	Vertical or horizontal w/ finger wiers	$250,000
Slide gates		$8,000
Mechanical/Electrical		$25,000
	Total Alternative ST2 Opinion of Construction Cost (2,3)	**$8,478,000**

(1) Includes concrete, excavation, backfill, superstructure, etc.
(2) Costs in Table do not include deep foundations, contractor overhead, engineering or contingency
(3) Based on ENR Building Cost Index 5563 (Nov 2015)

8.4. ALTERNATIVE ST3 – SEQUENCING BATCH REACTORS (SBRs) FOLLOWED BY UV DISINFECTION

Alternative ST3 for secondary treatment consists of a four basin sequencing batch reactor (SBR) system followed by UV disinfection. Each tank will be cast-in-place concrete and custom-designed to complement performance characteristics of the selected process equipment. Similar to other options considered in this Facility Plan, effluent from the SBR process will be disinfected through a UV disinfection system prior to discharge to the receiving stream.

A sequencing batch reactor (SBR) is a specialized secondary treatment process utilizing suspended growth micro-organisms for biological reduction of soluble and suspended organic material, along with a reduction in targeted nutrients such as nitrogen and phosphorus. The microbial functions are much the same as previously described for the MLE activated sludge process and the multi-stage oxidation ditch system, except that the various biological conditions are created within each SBR basin instead of in a series of distinct tanks. No recycle pumps or piping is required with an SBR system.

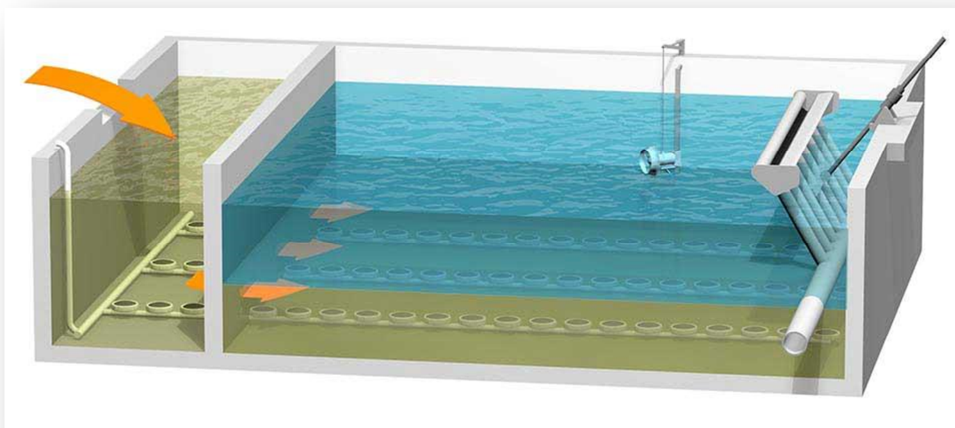


Figure 8-7 SBR Process

In a typical SBR process, wastewater flows into one of the SBR basins where it is blended with settled biomass from the previous cycle. Depending on the biological conditions that are targeted, this fill cycle can be quiescent or mixed. For biological nutrient reduction the initial fill period is typically quiescent to introduce fresh organic material into the concentrated biomass to encourage anoxic or anaerobic conditions. After a set period of time or when the basin reaches its full capacity, the mixing and aeration equipment is activated to create aerobic conditions for consumption of carbon-based organic matter. Instrumentation monitors dissolved oxygen levels and other characteristics to adjust the aeration process for optimal performance. After completing the react cycle, the basin contents are again returned to quiescent conditions where the microorganisms settle to bottom of the basin to prepare for decanting of the treated and clarified effluent. The final step is to decant clarifier effluent from the top of the basin and return the basin to an “idle” mode where it will remain ready for receiving the next batch of influent wastewater for treatment.

Each of the four SBR basins receives influent wastewater in either a sequential rotation or continuously in parallel.

- In a sequential batch system, the first basin will be in fill mode, while the second basin is in react mode and the third basin is in a settle phase and the last basin is decanting. This sequence continues to rotate through the four basins such that one tank is available to accept influent wastewater at all times. In normal operations, the fill and decant modes do not take place concurrently, thereby limiting the potential for discharging untreated wastewater to the receiving stream.
- In a continuous fill SBR system, influent wastewater is evenly divided between all four basins and is fed on a continuous basis regardless of the treatment stage. To reduce the risk of discharging incompletely-treated wastewater to the receiving stream, the basin configuration is typically longer and narrower from inlet to outlet, with a baffle wall constructed to create an inlet zone. Benefits with the continuous influent systems are that flow rates into the basins are reduced and any loading "slugs" are evenly divided between the four basins rather than concentrated in a single basin. A flow split structure ahead of the continuous fill SBR system is required to ensure balanced flow and loading distribution.

For SBR systems, the operating volume is variable depending on the influent flow rates. Each basin will have a Top Water Level (TWL) which is the maximum water depth that a basin can receive without initiating overflow protection controls. In addition, each basin will have a Bottom Water Level (BWL) which provides adequate holding volume for the settled biomass with a design buffer zone over the sludge blanket. Water depth varies between these two elevations based on influent flow rates, preprogrammed operational controls and operator input. In addition, the stage or cycle times are automatically adjusted by the process control system based on influent flow variations for optimal performance. For example, cycle times are automatically shortened for peak flow events to increase the number of "batches" processed through each basin, which maintains a high-level of effluent quality over the full range of design flow rates.

Reactor layout and design is dependent on the type of SBR system selected. For example continuous feed SBR's tend to be longer and narrower to maximize the distance between the influent feed and effluent decant. In contrast, systems that employ jet aeration/mixing headers tend to be shorter and wider to take advantage of the mixing technology and create conditions similar to a complete mix activated sludge process. With enhanced aeration and mixing, most SBR systems have Top Water Levels between 18 and 20-feet for the enhanced oxygen transfer efficiencies.

Figure 8-8 SBR Piping

Preliminary sizing based on IDNR criteria suggest a total volume of approximately 3,000,000 gallons divided between 4 basins. Assuming the Top Water Level to be 20 feet, the footprint of each basin is approximately 5,000 sq. ft. Therefore depending on the type of aeration/mixing system chosen, the basin footprint could be 50'x100' for a jet header type system or 25'x200' for a continuous feed system.

The SBR process requires blowers and aeration equipment to provide air to the basins. Typically, for the size required positive displacement type air blowers are recommended. Four blowers can be designed for dedicated use in their respective basins or two blowers can be selected with shared service between two basins. IDNR reliability criteria suggest dedicated blowers are preferred.

The air supply can be transferred to the wastewater many different ways. SBR system manufactures utilize jet-aeration, fine bubble diffusers, and surface mixers for aeration equipment. Typically, jet-aeration and diffused air are the most popular due to the high transfer efficiency. Where fixed diffusers are installed within a basin, IDNR guidelines state that a minimum of four basins are required.

The design of the decanter provides removal of clarified effluent without entraining settled sludge or removing floating material and scum. Similar to the aeration system, many different configurations are available for decanters. The type chosen for design will be further evaluated in final design phase.

Decanters are sized and designed for the maximum hydraulic conditions they could be expected to process. Under average conditions this leads to short periods of high rate decant flows that need to be addressed when sized downstream piping and equipment.

Each basin will be provided with one waste sludge pump. The waste sludge will be removed from the SBR either during the mix or decant cycle. These pumps are generally the submersible non-clog sewage type. The waste sludge will be pumped to the solids treatment process.

8.4.1. Ultraviolet (UV) Disinfection

Treated secondary treatment effluent from the SBR process will pass through a UV disinfection channel prior to final discharge to the receiving stream. The UV disinfection system is described in more detail in Section 8.5.

8.4.2. Benefits and Disadvantages of Secondary Treatment Alternative ST3

Benefits of Secondary Treatment alternative ST3

- SBR process is a flexible, reliable treatment process and has the capacity to handle a large fluctuation in flows and loads with minimal decrease in treatment efficiency.
- Only process where reactor volumes can be adjusted by changing the programmed top and bottom water elevations.
- Final clarifiers and return sludge pumping facilities are not required.
- Minimal footprint due to design water elevations up to 20 feet, which also minimizes heat loss in winter months.
- Inherent microorganism selection through sequenced aerobic, anoxic and anaerobic environments minimizes sludge bulking and controls filaments.
- Biological nitrogen and phosphorus reduction and low Total-P potential with chemical addition.
- Fully automated process control and monitoring including blowers, pumps, mixers and effluent decanters.

Disadvantages of Secondary Treatment alternative ST3

- The higher decant rates for SBR’s requires oversizing of the UV disinfection system or effluent equalization.
- Equipment is proprietary and basin configuration is largely determined by the selected manufacturer’s operating strategy.
- May require higher degree of operator familiarity with computer-based control systems than required in the current a conventional activated sludge system.
- Rely on sole-source supplier for replacement equipment for future life of the plant.

8.4.3. Alternative ST3 – Opinion of Cost

A preliminary Opinion of Probable Construction Cost for alternative ST3 is included in Table 8-4.

Table 8-4 Alterative ST3 Conceptual Opinion of Probable Construction Cost

Item	Description	Cost
Sitework	Sitework only related to alternative	
Yard Piping		$150,000
Influent Flow Splitter (1)	Low head FS	$50,000
SBRs		
SBR Tanks (1)	4 tanks - 3.3 MG	$4,000,000
SBR Equipment	Blowers, aeration, decanters, controls	$1,600,000
Blower piping and supports		$200,000
Secondary Treatment Building		
Building/Structure (1)	4,000 sq ft with basement	$800,000
WAS Pumps		$80,000
WAS Piping and Valves		$250,000
Mechanical/Plumbing	for entire building	$160,000
Electrical/Controls	Drives, and for building	$360,000
Laboratory	Equipment and furniture	
Locker Rooms	Furniture	
Effluent Water System	(included elsewhere)	
Carbon Feed System	Storage tank, pumps, piping	$70,000
Iron Salt Feed System	Storage tank, pumps, piping	$100,000
UV Disinfection - 10 mgd	Larger due to decant process	
Channel/structure (1)		$140,000
UV Equipment	Vertical or horizontal w/ finger wiers	$300,000
Slide gates		$8,000
Mechanical/Electrical		$30,000
	Total Alternative ST3 Opinion of Construction Cost (2,3)	**$8,298,000**

(1) Includes concrete, excavation, backfill, superstructure, etc.
(2) Costs in Table do not include deep foundations, contractor overhead, engineering or contingency
(3) Based on ENR Building Cost Index 5563 (Nov 2015)

8.5. ULTRAVIOLET (UV) DISINFECTION

Common to each of the secondary treatment options is UV disinfection. Treated secondary treatment effluent will pass through a UV disinfection channel prior to final discharge to the receiving stream. For the Oxidation Ditch and MLE Activated Sludge alternative, the UV disinfection systems would be the same and sized for a hydraulic capacity of 6.0 mgd. For the SBR alternative, where instantaneous decant rates could be expected to be higher than the secondary hydraulic rate, we assumed a peak capacity of 10.0 mgd.

UV radiation does not inactivate microorganisms by chemical interaction. UV inactivates organisms by absorption of light, which causes a photochemical reaction that alters the nucleic acids (DNA and RNA) that are essential for cell function. UV radiation quickly dissipates into water to be absorbed or reflected off material within the water. The UV disinfection process produces negligible disinfection by-products.

Figure 8-9 UV Disinfection

UV dose is defined using IT (intensity and time) values similar to CT (concentration and time) values using chlorine. UV dose, IT, is a product of UV light intensity and exposure time in seconds, stated in units of milliWatt second per square centimeter ($mW \cdot s/cm^2$) or milliJoule per square centimeter (mJ/cm^2). *Giardia* and *Cryptosporidium* are more sensitive to UV than bacteria, and viruses are more resistant than bacteria.

Recent advances in UV technology have led to more effective lamp designs and space saving configurations including low-pressure, medium-pressure, and pulsed UV irradiation in channel mounting and pipe mounting configurations. IDNR requires doses at 20 mJ/cm^2 to achieve 4-log inactivation of *Cryptosporidium*, *Giardia,* and viruses respectively.

The UV system would be installed in a concrete channel. Space will be provided to add modules the UV system in the future. Chemical phosphorus removal using ferric addition generally reduces UV transmittance and will need to be considered carefully during the design process. Alternate chemicals for phosphorus precipitation or feeding ferric earlier in the treatment process can reduce impacts on the disinfection system.

9. SOLIDS PROCESSING AND DISPOSAL ALTERNATIVES

9.1. GENERAL

Stabilization of wastewater treatment plant sludge is required to meet the EPA 503 regulations if land application is used for disposal. To meet these requirements with aerobic or anaerobic digestion, specific requirements must be met for pathogen and vector attraction reduction. Wastewater sludge that has been stabilized through digestion is referred to as “biosolids”. Given the proximity and availability of farm/crop land near the Farm Site, it is assumed that the City will land apply their biosolids produced. Land applied biosolids will be required to meet Class B criteria.

Either aerobic or anaerobic digestion is an option for treatment of secondary treatment waste solids. Aerobic digestion is a power-intensive process. It is more often used when primary treatment is absent and typically found in smaller treatment plants with average flow less than approximately 5.0 MGD. Capital cost for aerobic digestion is typically 25-40% of the capital cost of anaerobic digestion. Normally anaerobic digestion is the best option if primary treatment is provided. It is also considered more cost effective (from operational standpoint) than aerobic digestion if the energy recovered from digester gas is sufficient to meet or exceed the sludge heating needs. Anaerobic digestion is a “Green” initiative.

During the Indianola Process Workshop three secondary treatment technologies were selected to be considered. Neither of the secondary treatment alternatives recommended from the workshop included primary treatment. In addition, due to the project capital cost constraints, aerobic digestion was selected for further consideration.

Two solids processing alternatives will be evaluated at the end of this section; 1) aerobic digestion followed by thickening (to 5% solids) then thickened biosolids storage with mixing and load-out, and 2) aerobic digestion followed by biosolids storage (2.5% solids) with mixing and load-out.

9.2. SLUDGE PRODUCTION FROM SECONDARY TREATMENT

The waste sludge produced from each secondary treatment process alternative evaluated in Chapter 8 will be very similar. The waste sludge off either of the secondary treatment processes is expected to be approximately 9,000 mg/l (clarifier underflow concentration) as feed sludge into the aerobic digestion process.

Additional waste sludge volume will be produced with total phosphorus nutrient removal using chemical removal. The additional waste sludge is expected to be around 20% more volume than without P removal. Jar testing can be completed to provide a more detailed estimate of additional waste sludge prior to final design of the solids treatment process.

9.3. AEROBIC DIGESTION

Because each of the secondary treatment processes reviewed did not include primary treatment, aerobic digestion was selected as a low cost option for meeting digestion requirements.

The EPA 503 Regulations require that 60 days or 40 days of detention time be provided at 15 or 20 degrees Celsius, respectively. Design temperature for Indianola's aerobic digestion will be 15 degrees C. Aerobic sludge digestion can use multiple tanks in series or parallel. If the aerobic digesters are set up to operate in series, the EPA allows a credit of 30% of the required detention time tank volume. The required detention time for series flow aerobic digestion prior to biosolids storage would then be 42 days. Several configurations of aerobic digesters, thickening and biosolids storage tank configurations are possible to meet current and future waste sludge volumes.

Thickening of solids in the digester to 2.5% solids can generally be achieved by gravity thickening and decanting thinner liquid from the top of the digester. Table 9-1 shows the aerobic digester systems and biosolids storage tank preliminary design parameters.

Table 9-1 Aerobic Digester and Biosolids Storage Tank Summary

Item	Units	Current Flows w/ P Removal	Future Flows w/ P Removal
Digester			
Feed solids	**%**	0.90%	0.90%
Number of digester tanks		4	4
SWD	ft	23	23
tank diameter	ft	75	75
Influent solids concentration	mg/L	9000	9000
SRT	days	65	42
Operation		Dual Train, Series feed	Dual Train, Series feed
Aearation Needs			
Oxygen Transfer Efficiency	**%**	10%	10%
SCFM Delivered	CFM	2,316	3,594
Digested sludge Storage			
Number of storage tanks		1	1
SWD	ft	23	23
tank diameter	ft	99	99
Solids concentration	%	2.5%	5.0%
Detention time (includes SRT in digester)	days	184	190

Four aerobic digester tanks at 75 ft diameter will be required to stabilize current and future flows. WAS will be fed to two trains of digesters with two digesters in each series. Each of the second aerobic digesters in series will be designed to take decant off the top of the digester and return the decant back to the head of the plant. The sludge will be transferred from the second digester in series into the biosolids storage tank. Table 9-1 shows that for the future design flows, one biosolids storage tank at approximately 100 ft. diameter is adequate to store biosolids, if the biosolids are thickened to 5% solids concentration. A second biosolids storage option would be to store biosolids at 2.5% solids and add a second biosolids storage tank (without doing digested sludge thickening).

Aeration to the aerobic digesters will be provided by four blowers (3 duty, 1 standby at design conditions). Each blower shall have a capacity of 1200 scfm, operating at 9.5 psig. Diffusers will be used for aerating the sludge and for mixing. Multiple types of diffuser systems will be evaluated further in final design. Blowers will be installed either in a building or outside in weather-proof enclosures and will be approximately 100 HP each.

9.4. BIOSOLIDS THICKENING AND STORAGE

Thickening of aerobic digested biosolids can be a beneficial process to reduce the biosolids storage volume required and land application costs. A minimum biosolids storage volume equal to 180 days of digested biosolids is recommended. To show the impact of solids concentration, three times more biosolids storage volume is required for 2.5% solids biosolids than for a 7.5% solids biosolids.

Several thickening technologies can thicken biosolids to a 5.0%-7.5% solids target. See Table 9-2 for the technologies and typical thickened solids percentages from each technology.

Table 9-2 Liquid Biosolids Thickening Technologies

Technology	Expected Thickened Solids Concentration
Rotary Drum Thickener	5-8%
Gravity Belt Thickener	5-7%
Centrifuge	>8%

Additional evaluation of thickening equipment will be completed during preliminary design, but for this evaluation a Rotary Drum Thickener (RDT) has been selected due to the following advantages:

- Technology can easily meet the solids goal
- Expected polymer use is small (12 lbs/dry ton)
- Cost for RDT is competitive with other technologies and between manufacturers
- Low energy use
- Easy to operate and provide normal maintenance with City staff
- Can also be used for thickening of WAS ahead of digestion

Thickener filtrate will be returned to the liquid flow stream ahead of secondary treatment. This return flow can be a significant side stream high in nutrients and can sometimes disrupt overall nutrient removal processes. The need for side stream equalization or treatment of this flow will be reviewed during final design.

A biosolids storage volume equal to 180 days of production will be stored at the Farm Site. This volume of biosolids storage will help the plant staff manage the land application process. The biosolids storage facilities will include a storage tank with mixing and a biosolids load out station for filling tanker trucks.

Land application of biosolids at Indianola is currently contracted out to a specialty contractor. We expect this practice to continue.

9.5. ALTERNATIVE SP1

This alternative for solids processing SP1 consists of stabilizing waste sludge through aerobic digestion and then thickening the digested biosolids to 5.0% solids, then storing 180 days of thickened biosolids volume in a biosolids storage tank on site. The aerobic digestion process, thickening and biosolids storage will include all sub-systems and equipment needed for the solids treatment process.

Four aerobic digester tanks will be provided for two trains of series treatment. The second tank in the series will have capabilities to decant lighter liquid off the top of the tank to provide some gravity thickening of the tank contents.

A single-story Thickening Building will house the process equipment to thicken the digested sludge as biosolids before biosolids storage. The equipment will include rotary drum thickeners, feed pumps, polymer storage and feed systems, thickened sludge pumps, load-out pumps, biosolids mixing pumps, piping, valves, electrical and mechanical systems.

A single open-top biosolids storage tank will be provided to store at least 180 days of processed biosolids ready for land application. The biosolids storage tank will include a pumped recirculation jet nozzle mixing system.

9.5.1. Benefits and Disadvantages of Solids Processing Alternative SP1

<u>Benefits of Solids Processing alternative SP1</u>

- Very flexible process to handle a variety of waste sludge concentrations
- Can increase biosolids concentration to boost days of storage
- Can use storage in digester for volume ahead of thickening
- Land application of biosolids will be with higher solids concentration product – less hauling and less time

<u>Disadvantages of Solids Processing alternative SP1</u>

- Lots of tankage required
- Decant of top of digester and thickener underflow will be high in nutrients and the return streams will have an impact on secondary treatment design
- Aerobic digestion and thickening processes have significant operational impacts (energy and polymer)

9.5.2. Alternative SP1 – Opinion of Cost

A preliminary Opinion of Probable Construction Cost for alternative SP1 is included in Table 9-3.

Table 9-3 Alternative SP1 Opinion of Probable Construction Cost

Item	Description	Cost
Sitework	Sitework only related to alternative	
Yard Piping		**$100,000**
Aerobic Digesters		
Structure (1)	Four 75 ft dia 25 ft swd	$1,700,000
Aeration and blowers	Medium bubble, blowers outside	$390,000
Piping and valves		$50,000
Electrical/Controls		$40,000
	subtotal	**$2,180,000**
Solids Treatment Building		
Building - Substructure (1)	30x40	$240,000
Thickening equipment	Rotary drum thickeners - 2	$300,000
Polymer system	Drum feed system	$40,000
Thickener feed pumps		$50,000
Thickened sludge pumps		$50,000
Piping and valves		$150,000
Mechanical/Plumbing		$80,000
Electrical/Controls		$150,000
	subtotal	**$1,060,000**
Biosolids Storage Tank		
Prestressed Tank (1)	1.5 millon gallon	$1,400,000
Mixing system		$100,000
Sludge load out	pumps and piping	$100,000
Piping and valves		$60,000
Electrical/Controls		$40,000
	subtotal	**$1,700,000**
	Total Alternative SP1 Opinion of Construction Cost (2,3)	**$5,040,000**

(1) Includes concrete, excavation, backfill, superstructure, etc.
(2) Costs in Table do not include sitework, contractor overhead, engineering or contingency
(3) Based on ENR Building Cost Index 5563 (Nov 2015)

9.6. ALTERNATIVE SP2

This alternative for solids processing SP2 consists of stabilizing waste sludge through aerobic digestion and then storing 180 days of 2.5% solids biosolids volume in biosolids storage tanks on site. The aerobic digestion process and biosolids storage will include all sub-systems and equipment needed for the solids treatment process.

Alternative SP2 is similar to Alternative SP1 except:

- No biosolids thickening is provided. Biosolids will be stored at 2.5% solids concentration.
- Two biosolids storage tanks will be required.
- Biosolids mixing pumps, load out pumps, piping, valves, electrical and mechanical equipment will be provided in a small single-story building.

9.6.1. Benefits and Disadvantages of Solids Processing Alternative SP2

<u>Benefits of Solids Processing alternative SP2</u>

- Very flexible process to handle a variety of waste sludge concentrations
- Not relying on thickening processes (operator and polymer)
- Land application process may work best with high volume umbilical system – more efficient process

<u>Disadvantages of Solids Processing alternative SP2</u>

- More tankage required than SP1
- Decant from top of digester will be high in nutrients and return stream will have an impact on secondary treatment design
- Aerobic digestion has significant operational impacts (energy)

9.6.2. Alternative SP2 – Opinion of Cost

A preliminary Opinion of Probable Construction Cost for alternative SP2 is included in Table 9-4.

Table 9-4 Alternative SP2 Opinion of Probable Construction Cost

Item	Description	Cost
Sitework	Sitework only related to alternative	
Yard Piping		**$50,000**
Aerobic Digesters		
Structure (1)	Four 75 ft dia 25 ft swd	$1,700,000
Aeration and blowers	Medium bubble, blowers outside	$390,000
Piping and valves		$50,000
Electrical/Controls		$40,000
	subtotal	**$2,180,000**
Biosolids Pump station		
Structure (1)	Submersible pump station	$75,000
Sludge pumps		$50,000
Piping and valves		$40,000
Mechanical/Plumbing		$15,000
Electrical/Controls		$20,000
	subtotal	**$200,000**
Biosolids Storage Tank		
Prestressed Tank (1)	Two 1.5 million gallon	$2,800,000
Mixing system		$200,000
Sludge load out	pumps and piping	$100,000
Piping and valves		$80,000
Electrical/Controls		$50,000
	subtotal	**$3,230,000**
	Total Alternative SP2 Opinion of Construction Cost (2,3)	**$5,660,000**

(1) Includes concrete, excavation, backfill, superstructure, etc.
(2) Costs in Table do not include sitework, contractor overhead, engineering or contingency
(3) Based on ENR Building Cost Index 5563 (Nov 2015)

10. ANCILLARY TREATMENT FACILITIES IMPROVEMENTS

10.1. ADMINISTRATION BUILDING

A new Administration Building will be provided at the Farm Site to support operations of the Indianola Wastewater Treatment Plant. The Administration Building will include space for; laboratory, control room, training room, reception area, operator's offices, records storage, restrooms, locker rooms, electronics repair area, electrical, mechanical and garage. Some additional building spaces will be provided in the Administration Building to house the effluent sampler and UV disinfection equipment. The Administration Building will be a single story metal framed building with approximately 4,000 sq.ft of floor space. A breakdown of each space by approximate floor area is as follows:

Space	Approx. Sq. Ft.
Laboratory	600
Offices (3)	450
Training room	300
Locker rooms	250
Rest rooms	200
Reception area	200
Storage	120
Electrical	250
Mechanical	130
Electronics repair	400
Garage	900
UV Disinfection	200

10.2. SITE FACILITIES

The new Indianola Wastewater Treatment Plant site will include gravel-surfaced access roads and concrete parking areas around each of the buildings. Concrete sidewalks will be supplied around the site as needed for plant operations.

The area around the Administration Building will be seeded with lawn type grasses and the rest of the grass areas will be seeded in native prairie grasses.
The perimeter of the plant site will be enclosed by chain link or decorative fencing. Two security gates will be provided for access to the treatment facility.

10.3. PLANT EFFLUENT WATER SYSTEM

A plant effluent water system will be provided to supply plant effluent water throughout the wastewater treatment plant for wash down water and for processes uses. Plant effluent water will be pulled from downstream of the final clarifiers prior to disinfection. An automatic operated package pump station will be provided to supply the plant effluent to the non-potable water distribution system at the plant.

The City will also pump plant effluent water from the wastewater treatment plant back to Indianola Country Club golf course to supply irrigation water to a pond. Additional disinfection would be required for this water supply to the golf course as required by IDNR.

Figure 10-1 - Effluent Water System

10.4. VACTOR RECEIVING STATION

A vactor receiving station will be provided near the Headworks Building to allow for dumping of the City's vactor truck. The vactor receiving station will be provided with flushing water to help clean the area and push the dumped debris into the mechanical screens for removal. The vactor receiving station is <u>not</u> planned to receive other hauled wastes from other sources.

Figure 10-2 - Vactor Receiving Station

10.5. EMERGENCY ENGINE GENERATOR

An emergency engine generator will be provided for stand-by power service for the Indianola wastewater treatment plant. The stand-by generator will be a self-enclosed generator with base fuel tank. An automatic transfer switch will transfer the plant load to the stand-by generator on loss of power. The emergency engine generator will not be used for peak load shaving.

10.6. VEHICLE STORAGE BUILDING

A 6,000 sq.ft. Vehicle Storage Building will be provided for storage and service of WWTP vehicles and equipment. The building will be a metal-framed building with six overhead bays.

Figure 10-3 Vehicle Storage Building

Table 10-1 Ancillary Systems – Conceptual Opinion of Probable Construction Cost

Item	Description	Cost
Sitework		
Grading	Site grading	$80,000
Seeding and finishes		$18,000
Concrete Drives	Around buildings only	$50,000
Gravel drives		$100,000
Concrete sidewalks	Between processes	$30,000
Site fencing	Perimeter chain-link	$60,000
Gates	Two access gates	$12,000
Yard Piping	Misc. Yard Piping	$300,000
Site drainage	Storm drainage	$150,000
Site Electrical	Engine generator separately	$200,000
	subtotal	**$1,000,000**
Vactor Receiving Station (1)		**$50,000**
Administration Building (1)	4,000 sq ft metal building	$600,000
Laboratory furnishings	Counters, cupboards	$50,000
Lab equipment	Allowance	$30,000
Control system	Computers hardware and software	$300,000
Mechanical/plumbing	HVAC and plumbing	$180,000
Electrical		$100,000
	subtotal	**$1,260,000**
Effluent Water System	Package system	**$80,000**
Emergency Engine Generator	850 KW/hr with integral fuel tank	**$350,000**
Vehicle Storage Building (1)	6,000 sq ft modular building	$360,000
Concrete foundation		$120,000
Mechanical/Plumbing		$40,000
Electrical		$40,000
	subtotal	**$560,000**
	Total Ancillary Opinion of Construction Cost (2,3)	**$3,300,000**

(1) Includes concrete, excavation, backfill, superstructure, etc.
(2) Costs in Table do not include sitework, contractor overhead, engineering or contingency
(3) Based on ENR Building Cost Index 5563 (Nov 2015)

11. RECOMMENDED TREATMENT FACILITY ALTERNATIVE IMPROVEMENTS

11.1. GENERAL

This Section shows four comparative overall wastewater treatment plant options by selecting individual preliminary, secondary and solids processing options (from Sections 7-9) and combining them to logical overall treatment plant selections. A recommended treatment plant option for treatment process selection will emerge from this analysis of configurations.

11.2. P2 + ST1 + SP1

(Gravity sewer to Farm Site, Headworks Building, Grit Removal, Daily Equalization, Wet Weather Side Stream Treatment; Flow Splitter, Oxidation Ditch, Flow Splitter, Final Clarifier, UV Disinfection; Aerobic digestion, WAS thickening and Biosolids Storage of 5% solids)

This alternative grouping includes gravity flow of all wastewater flows to the Farm Site. All preliminary treatment, secondary treatment and solids processing and storage would be completed at this site. A two or three train oxidation ditch system followed by secondary clarifiers would be the selected secondary treatment alternative. Final effluent would be disinfected by UV disinfection then discharged to the receiving stream. Waste activated sludge from the secondary treatment process would be processed by series flow aerobic digestion then mechanically thickened and stored as biosolids in a storage tank. Note that additional UV disinfection would be required for this alternative when the wet weather side stream treatment system is operational during disinfection season. Table 11-1 shows the combined opinion of construction cost for this grouping of alternatives.

Table 11-1 Combined Alternative Opinion of Probable Construction Cost

Item	Description	Cost
Preliminary Treatment Alternative P2	from Table 7-2	**$9,105,000**
Secondary Treatment Alternative ST1	from Table 8-1	**$8,691,000**
Solids Processing Alternative SP1	from Table 9-3	**$5,040,000**
Additional Peak Flow Trmt UV Disinfection	Lump sum	**$300,000**
	subtotal combined alternative (1,2)	**$23,136,000**

(1) Costs in Table do not include contractor overhead, engineering or contingency
(2) Based on ENR Building Cost Index 5563 (Nov 2015)

11.3. PT2 + ST2 + SP1

(Gravity sewer to Farm Site, Headworks Building, Grit Removal, Daily Equalization, Wet Weather Side Stream Treatment; Flow Splitter, Conventional activated sludge, Flow Splitter, Final Clarifier, UV Disinfection; Aerobic digestion, WAS thickening and Biosolids Storage of 5% solids)

This alternative grouping includes gravity flow of all wastewater flows to the Farm Site. All preliminary treatment, secondary treatment and solids processing and storage would be completed at this site. A three train conventional activated sludge system followed by secondary clarifiers would be the selected secondary treatment alternative. Final effluent would be disinfected by UV disinfection then discharged to the receiving stream. Waste activated sludge from the secondary treatment process would be processed by series flow aerobic digestion then mechanically thickened and stored as biosolids in a storage tank. Note that additional UV disinfection would be required for this alternative when the wet weather treatment system is operational during disinfection season. Table 11-2 shows the combined opinion of construction cost for this grouping of alternatives.

Table 11-2 Combined Alternative Opinion of Probable Construction Cost

Item	Description	Cost
Preliminary Treatment Alternative P2	from Table 7-2	**$9,105,000**
Secondary Treatment Alternative ST2	from Table 8-3	**$8,478,000**
Solids Processing Alternative SP1	from Table 9-3	**$5,040,000**
Additional Peak Flow Trmt UV Disinfection	Lump sum	**$300,000**
	subtotal combined alternative (1,2)	**$22,923,000**

(1) Costs in Table do not include contractor overhead, engineering or contingency
(2) Based on ENR Building Cost Index 5563 (Nov 2015)

11.4. PT2 + ST3 + SP1

(Gravity sewer to Farm Site, Headworks Building, Grit Removal, Daily Equalization, Wet Weather Side Stream Treatment; Flow Splitter, SBRs, UV Disinfection; Aerobic digestion, WAS thickening and Biosolids Storage of 5% solids)

This alternative grouping includes gravity flow of all wastewater flows to the Farm Site. All preliminary treatment, secondary treatment and solids processing and storage would be completed at this site. A four tank sequenching batch reactor (SBR) system would be the selected secondary treatment alternative. Final effluent would be disinfected by UV disinfection then discharged to the receiving stream. Waste activated sludge from the secondary treatment process would be processed by series flow aerobic digestion then mechanically thickened and stored as biosolids in a storage tank. Note that additional UV disinfection would be required for this alternative when the wet weather treatment system is operational during disinfection season. Table 11-3 shows the combined opinion of construction cost for this grouping of alternatives.

Table 11-3 Combined Alternative Opinion of Probable Construction Cost

Item	Description	Cost
Preliminary Treatment Alternative P2	from Table 7-2	**$9,105,000**
Secondary Treatment Alternative ST3	from Table 8-4	**$8,298,000**
Solids Processing Alternative SP1	from Table 9-3	**$5,040,000**
Additional Peak Flow Trmt UV Disinfection	Lump sum	**$300,000**
	subtotal combined alternative (1,2)	**$22,743,000**

(1) Costs in Table do not include contractor overhead, engineering or contingency
(2) Based on ENR Building Cost Index 5563 (Nov 2015)

11.5. PT1 + ST3 + SP1

(Upgrade and reuse facilities at NWWTF, force main to Farm Site, Headworks Building, Grit Removal, Mechanical fine screens; Flow Splitter, SBRs, UV Disinfection; Aerobic digestion, WAS thickening and Biosolids Storage of 5% solids)

This alternative grouping includes reuse of some of the NWWTF preliminary treatment process units followed by pumping the wastewater to the Farm Site. The remaining preliminary treatment, secondary treatment and solids processing and storage would be completed at this site. A four tank sequenching batch reactor (SBR) system would be the selected secondary treatment alternative. Final effluent would be disinfected by UV disinfection then discharged to the receiving stream. Waste activated sludge from the secondary treatment process would be processed by series flow aerobic digestion then mechanically thickened and stored as biosolids in a storage tank. Table 11-4 shows the combined opinion of construction cost for this grouping of alternatives.

Table 11-4 Combined Alternative Opinion of Probable Construction Cost

Item	Description	Cost
Preliminary Treatment Alternative P1	from Table 7-1	**$5,430,000**
Secondary Treatment Alternative ST3	from Table 8-4	**$8,298,000**
Solids Processing Alternative SP1	from Table 9-3	**$5,040,000**
Additional Peak Flow Trmt UV Disinfection	Lump sum	**$250,000**
	subtotal combined alternative (1,2)	**$19,018,000**

(1) Costs in Table do not include contractor overhead, engineering or contingency
(2) Based on ENR Building Cost Index 5563 (Nov 2015)

12. SUMMARY OF RECOMMENDED IMPROVEMENTS

12.1. GENERAL

The recommended Indianola Wastewater Treatment Plant is a new treatment facility at the Farm Site. The new wastewater treatment plant will eliminate the existing NWWTF at the Hoover Street site and allow the City to sell or re-purpose the existing 32 acre wastewater treatment plant site. The proposed site plan for the Indianola Wastewater Treatment Plant at the Farm Site is shown in Figure 12-1. The combined overall treatment process recommended for the City of Indianola as outlined in Chapter 11 is P2 + ST1 + SP1.

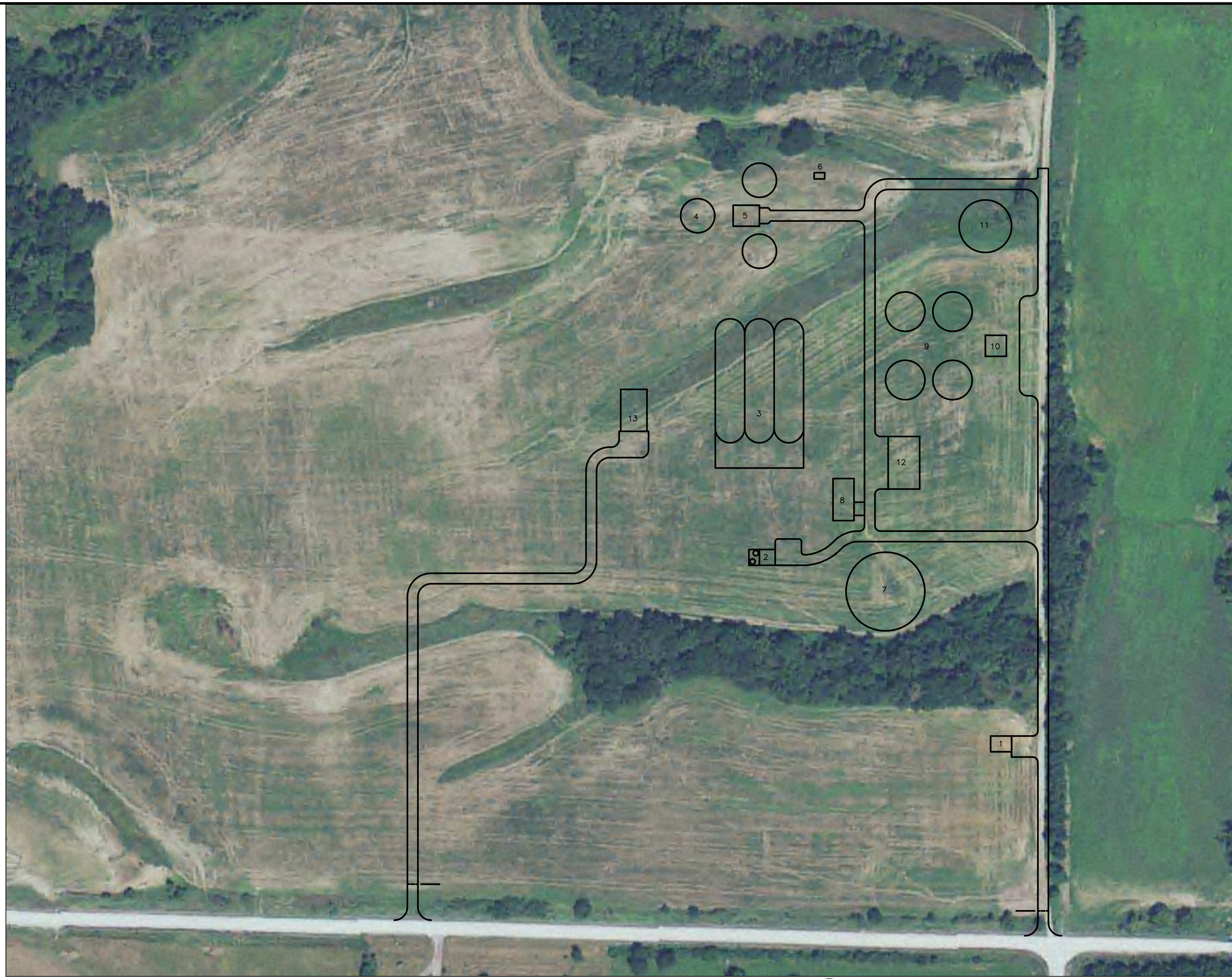
12.2. CONVEYANCE

Wastewater flows to the new treatment plant will convey by gravity through a new interceptor sewer. The new 36-inch gravity sewer will connect to the existing interceptor sewer ahead of the existing NWWTF. The new 36-inch interceptor will generally follow Cavitt Creek to the north to the new Farm Site (approximately 11,000 feet). A final alignment will be selected during the preliminary design phase. Permanent and temporary easements will be acquired for the sewer construction over the next couple of years. The new gravity interceptor sewer will convey all the City's sanitary sewer flows to the new wastewater treatment facility.

12.3. WASTEWATER TREATMENT PROCESS

The wastewater treatment process schematic for the recommended treatment process is included in Figure 12-2. Raw wastewater flows into the Headworks Building where the flow goes through fine screens and then into a self-cleaning style trench wetwell for pumping up the hill to the grit removal process. Influent wastewater will be sampled and metered in the Headworks Building. The screening and pumping preliminary treatment processes will be sized to handle the full range of wastewater flows that reach the treatment plant through the interceptor sewer.

The raw wastewater is pumped up the hill to the grit removal system. From this process unit the liquid treatment process is completely done by gravity flow through all the process units. Two trains of grit removal will be provided to remove grit from all the flow. Grit will be removed from the channels at the Grit Building and stored into dumpsters for ultimate disposal at the landfill. Flows up to 6.0 mgd will be metered and sent on to secondary treatment. Flows over 6.0 mgd will be diverted automatically to the equalization tank. The equalization tank will either hold the flows for treatment when the plant flow subsides below 6.0 mgd or divert peak flows to the Wet Weather Side Stream Treatment system. The equalization tank can also be operated as a diurnal flow equalization tank to provide a constant feed to the secondary treatment system over a 24 hour daily average rate. An excess flow pump station will be provided to; 1) return all wastewater flows passing thru the equalization tank to the secondary treatment system (when influent flows are less than 6.0 mgd), or 2) pump all excess flows above 6.0 mgd to the Wet Weather Side Stream Treatment process. The excess flow pump station will be a submersible pump station with a connected valve vault.



SITE LEGEND

1 HEADWORKS BUILDING
2 GRIT REMOVAL
3 OXIDATION DITCH (3 TRAINS)
4 SECONDARY CLARIFIERS (3 TRAINS)
5 SECONDARY TREATMENT BUILDING
6 UV DISINFECTION/BUILDING
7 2.0 MG EQUALIZATION TANK
8 PEAK FLOW TREATMENT
9 AEROBIC DIGESTERS (4 TANKS)
10 SOLIDS PROCESSING BUILDING
11 BIOSOLIDS STORAGE TANK
12 VEHICLE STORAGE BUILDING
13 ADMINISTRATION BUILDING

1 SITE PLAN

0 100
1"=100'

NORTH

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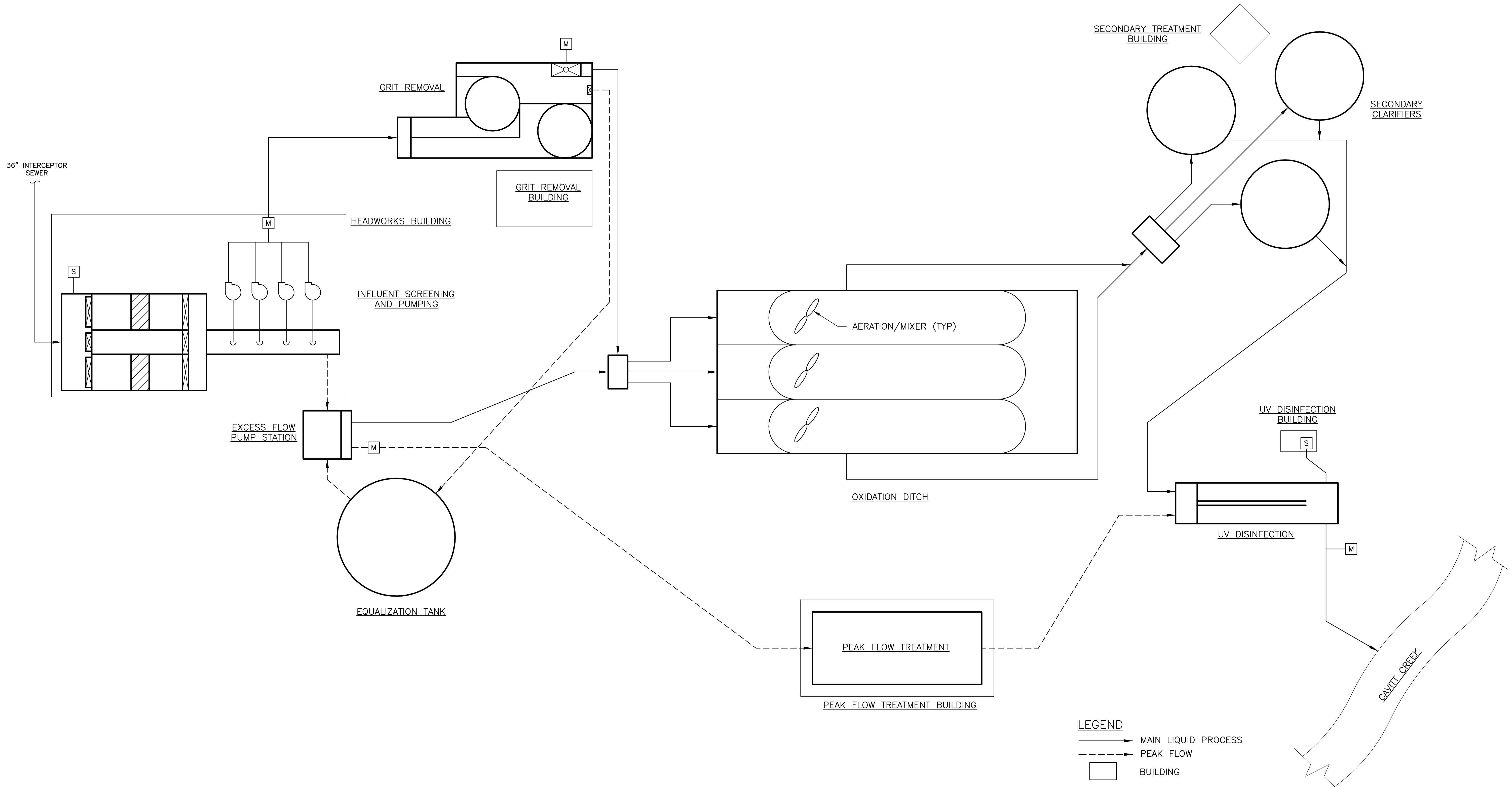
INDIANOLA – SITING STUDY
CITY OF INDIANOLA
INDIANOLA, IOWA 2013

WASTEWATER TREATMENT
SITE PLAN

SHEET NO.
FIG 12–1



LEGEND

- MAIN LIQUID PROCESS
- PEAK FLOW
- BUILDING
- M METER LOCATION
- S SAMPLE LOCATION

LIQUID TREATMENT PROCESS SCHEMATIC

PRELIMINARY
NOT FOR CONSTRUCTION

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NO.	DATE	BY	REVISION DESCRIPTION

HRGreen HRGreen.com

INDIANOLA WASTEWATER SYSTEM IMPROVEMENTS
CITY OF INDIANOLA
INDIANOLA, IOWA

GENERAL
LIQUID TREATMENT PROCESS SCHEMATIC

SHEET NO.
FIG 12–2

Xrefs: xgt-1-dh01; XP-O-PID

The Wet Weather Side Stream Treatment system will be either a 10 mgd ballasted flocculation peak flow treatment system (such as Actiflo), or a 10 mgd cloth media filter system. The wet weather treatment system will be started up during extreme weather events to provide physical treatment to the remaining flows above the treatment plant's secondary treatment capacity.

The Actiflo process (manufactured by Kruger) is a high rate, compact process for wet weather treatment. The process operates with microsand which enhances floc formation and acts as a ballast to aid in rapid settlement of coagulated material. The microsand ballasted flocs display unique settling characteristics, which allow for clarifier designs with very high overflow rates and short retention times. The Actiflo system design for peak flow treatment results in footprints that are a fraction of the size of conventional clarifier systems. Actiflo is an approved technology by the US EPA for peak flow treatment.

The cloth disc media filter system as manufactured by AquaAerobics – Aqua Prime is a fine particle filtration system using cloth media. The filter discs when coupled with a coagulant feed system filter the effluent at high rates with high capture rates for BOD, TSS, TKN and Phosphorus.

The recommended secondary treatment process for the Indianola Wastewater Treatment Plant is an oxidation ditch. The oxidation ditch process will provide nitrification and denitrification for total nitrogen removal as well as BOD removal. Two or three trains of oxidation ditches will be provided. During low flow periods the plant staff may choose to take one of the treatment trains out of service. A flow splitter will be provided ahead of the secondary treatment process to equally split flow to the treatment trains. A single aerator/mixer is the main piece of equipment needed in the oxidation ditch.

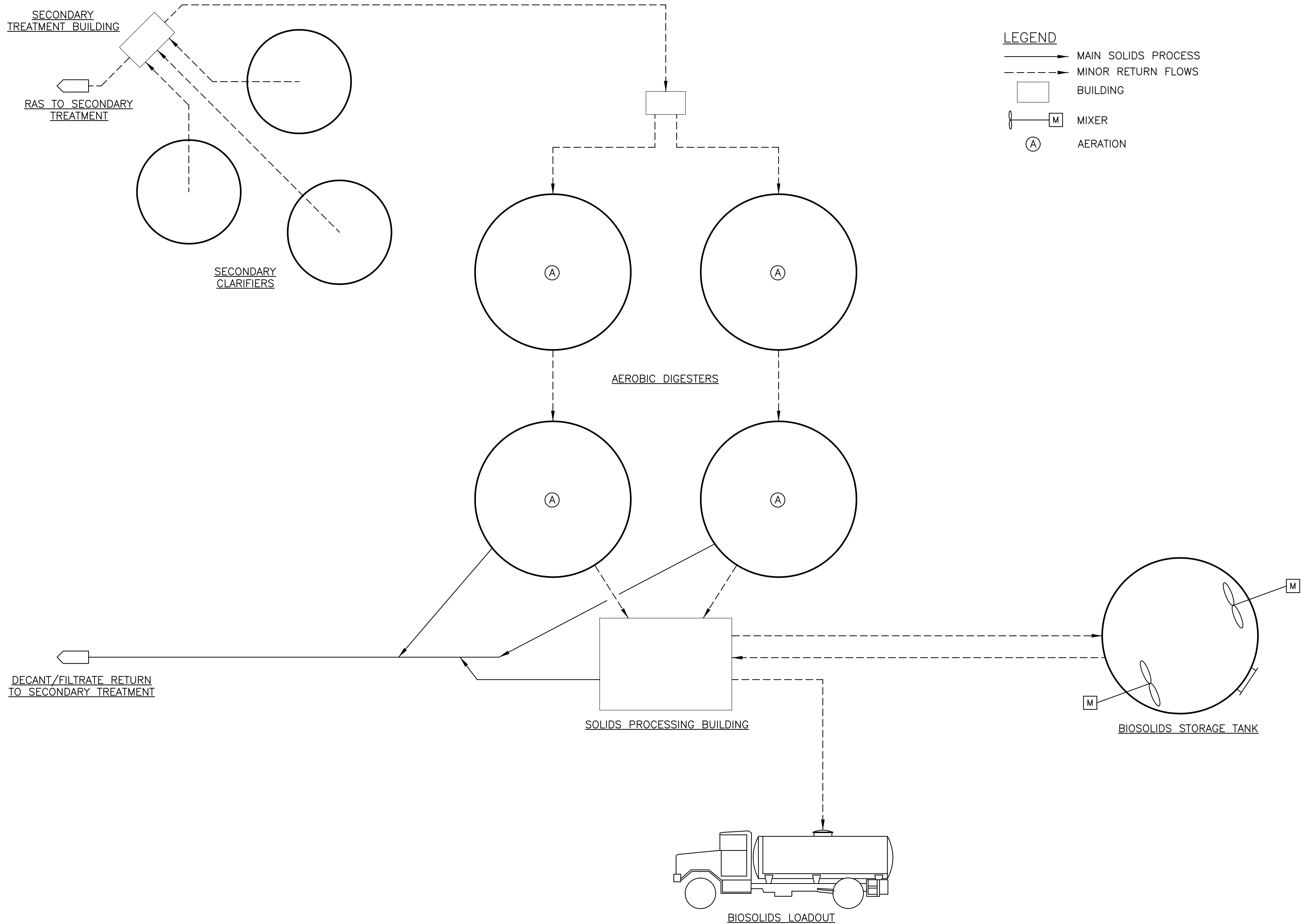
Three secondary clarifiers will be provided to settle the activated sludge following the oxidation ditches and to chemically precipitate phosphorus. The clarified effluent will flow over weirs to the disinfection process. The activated sludge settling in the clarifiers will be pumped back to the treatment process as return activated sludge from the Secondary Treatment Building. Waste sludge pumps also located in the lower level of the Secondary Treatment Building will pump waste sludge to the solids treatment process. A flocculant such as ferric chloride will be added just ahead of the secondary clarifiers to precipitate out the remaining phosphorus. A secondary flow splitter will be installed ahead of the secondary clarifiers to equally split flow to each of the three clarifiers.

An ultraviolet (UV) disinfection system will be installed downstream of the secondary clarifiers to disinfect the effluent prior to discharge to the Middle River. The UV disinfection will also disinfect flows from the Wet Weather Side Stream Treatment system prior to blending the physically treated peak flow with the effluent from the secondary treatment system. A small building will be included next to the effluent channel to house the electrical equipment and effluent sampler.

12.4. SOLIDS TREATMENT PROCESS

Waste sludge from the secondary treatment process will be stabilized by aerobic digestion. A solids treatment schematic is included as Figure 12-3. Two trains of two aerobic digesters will be included to provide a flexible solids processing arrangement and to meet the requirements of the EPA 503 regulations. Aeration blowers and a diffused aeration system will be provided to supply the needed oxygen for the process.

A Solids Processing Building near the digester complex will house the blowers, pumps, sludge thickening equipment, polymer feed system, sludge load out equipment, mechanical and electrical. Digested sludge (biosolids) will be stored in a biosolids storage tank for disposal by land application in the fall. The above-grade, open-top biosolids storage tank will store more than 180 days of biosolids at the future flow and solids production condition. Decant from the second stage aerobic digesters and filtrate from the sludge thickening process will be returned back to the wastewater treatment process ahead of secondary treatment.



SOILDS TREATMENT PROCESS SCHEMATIC

PRELIMINARY
NOT FOR CONSTRUCTION

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INDIANOLA WASTEWATER SYSTEM IMPROVEMENTS
CITY OF INDIANOLA
INDIANOLA, IOWA

GENERAL
SOLIDS TREATMENT PROCESS SCHEMATIC

SHEET NO.
FIG 12–3

Xrefs: xgt-1-dh01; XP-0-PID

12.5. SUMMARY OF DESIGN PARAMETERS

Item		Size/Capacity
WWTP Flows		
ADW		2.30 mgd
AWW		5.91 mgd
MWW		9.10 mgd
PHWW		14.41 mgd
WWTP Loads	Avg. Day	Max Day
cBOD, lbs/day	2,988	5,815
TSS, lbs,day	3,896	9,351
Ammonia-N, lbs/day	383	717
TKN, lbs/day	588	1,103
Total Phosphorus, lbs/day	106	217
Mechanical Screens		
No. of units		2
Clear opening size, in		¼
Max flow per screen, mgd		16.0
Influent Pumping		
Type		vertical turbine solids handling
No. of units		4
Rated capacity each, gpm		TBD
Rated head, ft		TBD
Grit Removal		
Type		vortex or aerated
No. of units		2
Concentrator		cyclone
Dewatering		inclined screw
Equalization Tank		
Type		above grade, open top concrete
No of units		1
Capacity, mg		2.0
Dimensions		130 ft dia x 22 ft swd
Excess Flow Pumping Station		
Type		Submersible
No of units		4
Rated Capacity each, gpm		TBD
Rated head, ft		TBD
Oxidation Ditches		
No of units		2
Tank volume, each, gallons		1,320,000

Equipment	Mixer/Aerator
Additional mixing	Submersible mixers

Secondary Clarifiers

Type	Circular center-feed, peripheral draw
No of units	3
Diameter, ft	60
Sidewater depth, ft	14
Volume, each, cu ft	39,584

RAS Pumps

Type	Centrifugal
No of units	5
Rated Capacity each, gpm	TBD
Rated head, ft	TBD
Max RAS rate, mgd	6.0

Digester Feed Pumps (WAS Pumps)

Type	Centrifugal
No of units	2
Rated Capacity each, gpm	TBD
Rated head, ft	TBD

UV Disinfection

Type	TBD
No of channels	2
UV Transmittance	60

Aerobic Digesters

Type	series flow
No of units	4
Tank dia, ft	75
Tank swd, ft	23
SRT, days	42
Aeration, SCFM	3,594
No of blowers	4
Type	Positive displacement

Digested Sludge Thickening

Type	Rotary Drum
No of units	2
Rated capacity, each, gpm	100

Biosolids Storage Tank

Type	above grade, open top concrete
No of units	1
Capacity, mg	1.4
No of mixers	2
Type	Submersible

12.6. BIOWIN PROCESS MODELING

HR Green completed Biowin process modeling on the proposed wastewater treatment plant alternative to simulate the performance of the WWTP as a whole and the individual process units. The Biowin modeling was completed to verify the proposed WWTP's ability to meet the effluent limits under a practical range of design influent flows and loads. A summary of the Biowin process modeling results is included in Appendix J.

Five Biowin process model runs were completed at the following various WWTP flow/load conditions:

1. *AWW flow at max day loading*
2. *2 times AWW flow at max day loading (includes wet weather side stream treatment)*
3. *AWW flow at average month loading*
4. *¼ AWW flow at average month loading*
5. *Daily average flow at average month loading*

Two additional Biowin process model runs were completed comparing "Store and Treat" process vs. Wet Weather Side Stream Treatment. These runs were defined as:

6. *Store & Treat – 30 day wet weather*
7. *Side Stream Treatment blended with secondary treatment – 30 day wet weather*

The results from the Biowin process modeling showed the following:

- *All modeling runs met the desired effluent quality (less than 10 mg/l for BOD, TSS and Total Nitrogen, less than 1.0 mg/l for Total phosphorus, and less than required by WLA for Middle River for monthly ammonia)*
- *Wet weather side stream treatment blended with secondary treatment effluent produced slightly better effluent quality than "Store and Treat" through secondary treatment effluent quality.*
- *Biowin model identified a few conditions where supplemental carbon needs to be added.*
- *RAS return rates in the activated sludge process for all conditions are less than 50% of influent flow.*

12.7. RECOMMENDED ALTERNATIVE COST OPINION

Table 12-1 shows the Opinion of Probable Construction Cost for the recommended wastewater treatment alternative. The cost opinion is based on a Engineering News Record (ENR) Building Cost Index for cost metrics representative of the time of this Facility Plan was developed.

Table 12-1 Recommended Alternative Opinion of Probable Construction Cost

Item	Description	Cost
Preliminary Treatment Alternative P2	from Table 7-2	**$9,105,000**
Secondary Treatment Alternative ST1	from Table 8-1	**$8,691,000**
Solids Processing Alternative SP1	from Table 9-3	**$5,040,000**
Additional Peak Flow Treatment UV Disinfection	Lump sum	**$300,000**
Ancillary Systems	from Table 10-1	**$3,300,000**
	subtotal	**$26,436,000**
Contingency	20%	**$5,287,000**
	Total OPC (1,2)	**$31,723,000**

(1) Costs in Table do not include contractor overhead or engineering
(2) Based on ENR Building Cost Index 5563 (Nov 2015)

13. FUNDING

The City is planning to use a Planning and Design Loan administered by the Iowa Finance Authority ("IFA") to fund the engineering effort. The City is planning to use IFA's Clean Water State Revolving Fund (CWSRF) process and financing for the construction of improvements. The CWSRF program has been the City's primary option for recent wastewater improvements due to the low cost of financing and flexibility to draw funds as needed. No grant money has currently been identified.

The City of Indianola has recently passed a Local Option Sales Tax (LOST) to help fund the wastewater treatment plant project. This will allow the City to repay a significant portion of the CWSRF financing from LOST revenues.

Currently, the City budget and expenditures balance. The last rate sewer rate increase was in 2013. The operations and maintenance and loan payback will be funded by increasing sewer rates as needed in combination from revenues from the LOST. Other funding options will continue to be investigated by the City in an effort to provide the lowest cost of financing and minimize rate impact on wastewater users.

14. IMPLEMENTATION SCHEDULE

Below is a proposed implementation schedule for the improvements identified in this Facility Plan. This implementation schedule is based on estimated durations for IDNR review, final design, SRF funding and construction.

Complete Facility Plan	April 2016
Submit Facility Plan to IDNR	May 2016*/revised April 2018*
Complete Antidegradation Analysis - Submit to IDNR	May 2016*/revised Mar 2018*
Meet with IDNR to present Facility Plan	June 2016
IDNR to Approve Facility Plan	TBD
Submit Application for SRF Funding	March 2018
Begin WWTP Final Design	~~January 2019~~*/ Sept 2018*
30% Complete	~~March 2019~~*/ Nov 2018*
60% Complete	~~June 2019~~*/ Jan 2019*
90% Complete	~~August 2019~~*/ Mar 2019*
Submit Final Design for IDNR Construction Permit	~~September 2019~~*/ April 2019*
Construction Permit Issued	~~December 2019~~*/ July 2019*
Bidding/Award	~~January 2020~~*/ Sept 2019*
Construction Begins	~~March 2020~~*/ Oct 2019*
Construction Substantially Complete	~~November 2021~~*Dec 2021*
Construction Complete	June 2022

Appendix A - IDNR Planning Documents

IOWA DEPARTMENT OF NATURAL RESOURCES

NUTRIENT REDUCTION STRATEGY

FOR WASTEWATER TREATMENT PLANTS

The Iowa Nutrient Reduction Strategy is a science- and technology-based approach to assess and reduce nutrients delivered to Iowa waterways and the Gulf of Mexico. The strategy outlines efforts to reduce nutrients in surface water from point sources, such as municipal and industrial wastewater treatment plants, and nonpoint sources, including farm fields and urban areas, in a scientific, reasonable and cost-effective manner.

The Iowa strategy was developed in response to the 2008 Gulf Hypoxia Action Plan, which calls for the 12 states along the Mississippi River to craft strategies to reduce nutrients reaching the Gulf of Mexico. The Iowa strategy follows the recommended framework provided by the U.S. Environmental Protection Agency (EPA) in 2011. The DNR will work with wastewater facilities throughout the state to reduce nutrient discharges from point sources with a goal of reducing total phosphorus by 16 percent and total nitrogen by 4 percent. In addition to impacting the Gulf, nutrients also negatively affect local Iowa receiving streams. Nutrient reduction will help better protect those streams, especially during low flows.

WHAT FACILITIES ARE AFFECTED?

- 102 major municipal and 46 industrial wastewater facilities where biological nutrient removal is economically and technically feasible.
- Minor municipal wastewater facilities (less than 1 million gallons per day) will evaluate nutrient reduction alternatives when increasing design loads.
- Major industrial treatment plants that do not have biological treatment will assess nutrient removal possibilities during regularly scheduled permit renewals.

HOW WILL NUTRIENTS BE REMOVED?

- Biological nutrient removal, or BNR, was considered in this strategy. Other options for nutrient removal are available and can be evaluated.

HOW WILL THIS BE IMPLEMENTED?

- When a National Pollutant Discharge Elimination System (NPDES) permit is renewed, the permit will require that the facility conduct a two-year study to evaluate the costs and feasibility of installing biological nutrient removal and submit a proposed schedule for installation. After the study is completed, the schedule will be incorporated in the facility's NPDES.
- Timeframes for construction will be based on the negotiated schedules for major municipal and certain industrial facilities, case by case.

HOW ARE LIMITS SET?

- Technology-based limits will be implemented in a facility's NPDES permit. Many nutrient removal technologies are feasible, as they are already proven and well-established.
- Limits will be no more stringent than 10 mg/L for total nitrogen and 1 mg/L for total phosphorus.
- In general, these levels of nutrient reduction are technically and economically achievable for Iowa facilities.

HOW WILL COMPLIANCE BE DETERMINED?

- After BNR is installed and operational, the facility will have one year to conduct a process optimization evaluation prior to limits being established.
- Total nitrogen and phosphorus limits will be based on demonstrated plant performance, but no more than 10 mg/L (nitrogen) and 1 mg/L (phosphorus).
- Plants will be protected from stricter limits for 10 years if nutrient removal is installed.
- The facility will have monthly limits for nitrogen and phosphorus discharged. Compliance will be determined by the annual average, rather than by the monthly limits.

WWW.NUTRIENTSTRATEGY.IASTATE.EDU

GENERAL QUESTIONS
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MUNICIPAL QUESTIONS
Eric Wiklund, DNR: 515-725-0313 or eric.wiklund@dnr.iowa.gov

INDUSTRIAL QUESTIONS
Wendy Hieb, DNR: 515-725-8405 or wendy.hieb@dnr.iowa.gov

Key Principles and Consideration Factors for Incorporation of Non-Biological Peak Flow Processing Approaches in Iowa Wastewater Facilities

Various Iowa Communities are in the process of addressing peak flow management issues under federal and state consent agreements intended to assess sewer overload conditions, combined sewer overflow long term control planning (LTCP) and as part of facility planning to ensure optimum wastewater management under extreme weather conditions. It is anticipated that non-biological peak flow processing in a split treatment mode will be incorporated into Iowa facilities for four primary reasons:

1. To allow maximum flow processing and minimize sanitary system overflows/basement backups while sewer system corrective actions are being implemented;
2. As part of the LTCP for CSO communities, where sewer separation is not complete and as necessary to minimize the remaining overflow conditions in accordance with state/federal CSO program requirements;
3. As a measure to protect plant operations and process the maximum flows possible through the existing wastewater facilities under conditions that meet the reasonable threshold for a split treatment approach at the wastewater facility; and,
4. When necessary to limit flow variations to sensitive processes, such as biological nutrient removal (BNR) facilities.

As discussed in the *Iowa League of Cities v. EPA* decision, federal law does not allow EPA to dictate how facilities are designed to achieve applicable effluent limits. While the facility design is generally within the purview of the facility owner (and their design engineer), DNR does maintain responsibility to ensure that the design is reliable, will operate as intended and will meet the applicable permit limits. The basic principles/consideration factors for DNR's approval of the non-biological peak flow processing approach as part of the wastewater system design and the intended plant design-operation include the following items:

A. Is the utility currently addressing infiltration/inflow problems to reduce the system's susceptibility to backups and overflows?
B. Is peak flow processing needed to address CSO LTCP objectives?
C. Is peak flow processing needed to protect wastewater treatment operations, including advanced treatment processes such as BNR?
D. Will the peak flow processing approach allow the facility to maximize treatment, protect facility operations and minimize overflows while other corrective measures are being implemented?
E. Has the permittee demonstrated that incorporation of non-biological peak flow processing results in a design that meets applicable effluent quality requirements?

F. Is there a plan for addressing peak flows, and are the conditions that require the use of split treatment adequately defined?
G. How do receiving water conditions compare to anticipated effluent quality when peak flow processing is being employed?
H. If necessary, have load limitations based on dry/drought flow conditions been adjusted to reflect conditions occurring under wet weather/high flow conditions?
I. Has the permittee provided appropriate notice to the Department regarding the intended design-operation of the facilities that would be used for peak flow management and prepared a Peak Flow Operating Procedures manual?
J. Is the intended design consistent with "good engineer practices" for sizing the biological systems (e.g., appropriate capability to process peak flows that would be expected to exist absent the higher peak flows presently encountered by the system and/or as necessary to protect biological system performance)?
K. Does the treatment scenario that would be used for peak flow management provide the equivalent of primary clarification (e.g., overflow from an EQ basin, additional stand-by primary treatment unit(s), ballasted flocculation) for the portion of flow routed around biological or other advanced treatment units?
L. Has the facility been designed to ensure that reasonably anticipated peak flows (excepting those associated with extreme wet weather events caused by localized or area wide flooding that are inimical to contact recreation uses) will be disinfected?

DNR Approval/Permit Language

Assuming that the peak flow processing design and intended facility operations reasonably address the issues discussed above and the methods being applied will ensure that permit limitations are achieved when peak flow processing is employed, the construction of such facilities will be approved. In addition, the NPDES permit will contain the following information and permit language:

Fact Sheet

- Include a copy of the facility design schematic clearly indicating the process operation intended to be implemented to address peak flow conditions
- Identify the flow condition that is anticipated to exceed the capabilities of the biological system
- A reference to the Peak Flow Operating Procedures manual that has been prepared by the discharger to describe the sequence of events and operating procedures that will be used to trigger the initiation and termination of peak flow processing.

NPDES Permit Language

In accordance with the facilities Peak Flow Operating Procedures manual, this facility is authorized to operate non-biological treatment technologies to process peak wet weather wastewater flows when such flows exceed --- MGD or when, in the opinion of the permittee, the continued operation of the biological system could be jeopardized due to excessive flows (e.g., system washout). Use of the peak flow processing mode of operation is not authorized under any other condition without the express authorization of the Department. The permittee shall, as part of its 5 year permit application, include a report detailing the frequency of peak flow processing use, its effect on permit compliance, the progress made in reducing peak flows to the facility and a projection on the continued operation of such facilities over the next permit term.

Monitoring provisions will also be included to ensure "primary equivalent" performance when a EQ basin is used to provide such treatment.

Appendix B - Revised Loads and Flows

HRGreen

December 15, 2017

Terry Kirschenman, P.E.
Wastewater Section
Wallace State Office Building
502 E. 9th St.
Des Moines, IA 50319

Re: City of Indianola WWTF – Facility Plan – Revised Flows and Loads Report

Dear Terry:

On behalf of the City of Indianola, HR Green is submitting this Revised Flows and Loads Report for the Facility Plan project. The development of the report includes existing flow and load information and future growth allowance supporting data and calculations. Design Schedule G has also been enclosed. The Design Year in Schedule G is 2040.

The Flows and Loads Report is being revised as discussed during a meeting held between the Iowa DNR, the City of Indianola, and HR Green on October 18, 2017. Specifically, the ammonia-nitrogen results need to be revised due to erroneous results. The report and appendices provide further discussion of revised ammonia-nitrogen loadings.

Please review this report. If you have any questions or comments regarding the report please contact me at (515) 278-2913.

Sincerely,
HR Green, Inc.

James R Rasmussen

James R. Rasmussen, P.E.
Vice President

cc: Rick Graves, WWTF Superintendent
file

3.0 EXISTING CONDITIONS AND PROJECTIONS

3.1 EXISTING SERVICE AREA

The Indianola North WWTF treats wastewater from the incorporated areas of town. Residential, commercial and industrial sources make up the wastewater flow. The plant is located on Cavitt Creek on the northwest side of town. There are approximately 83 miles of sanitary sewer in the city. The collection system has historically received significant Inflow and Infiltration (I/I) to the sanitary system. In 2014, the City completed construction of a four phased program to reduce I/I to eliminate overflows and bypassing that is associated with the heavy I/I. Since this program has been completed, the City has noticed a reduction in sanitary sewer flows. The new WWTF needs to be designed to accommodate and/or handle reasonable peak flows during wet weather.

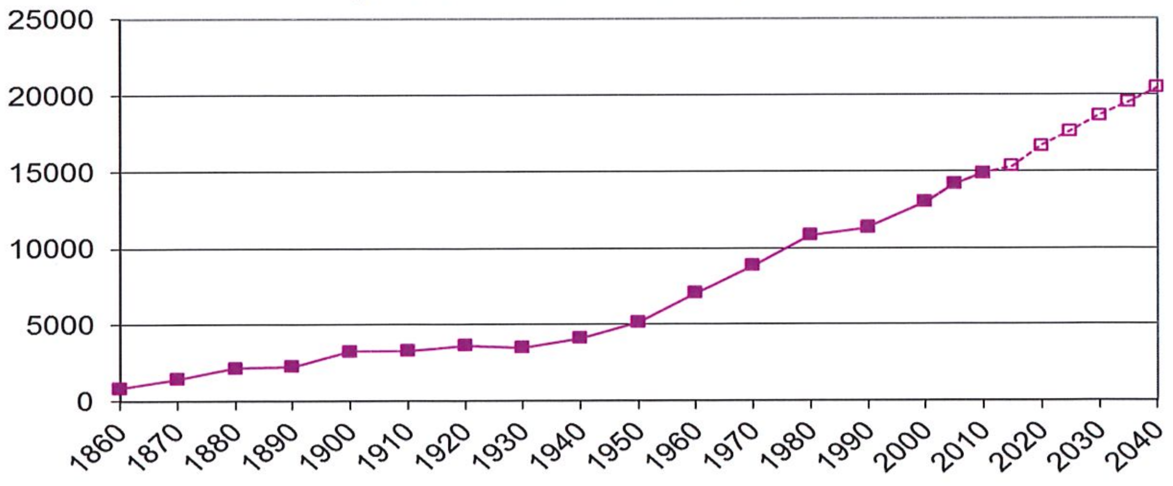
3.2 POPULATION

The population serviced by the Indianola North WWTF is assumed based on census information. The current population of Indianola is estimated at 15,310.

Census population data for the years 1860-present is shown in Figure 3-1 below. A comprehensive plan had been completed for the City in October 2011. The comprehensive plan forecasted population trends through 2030 using up-to-date growth trends and extrapolated population projections. The same increasing rate used in the comprehensive plan has been used to estimate future population through the end of the facility planning period (2040). The projected values are also plotted in Figure 3-1.

In 2007, Central Iowa Regional Transportation Planning Alliance (CIRTPA) released its Long Range Transportation Plan. A more aggressive growth rate was used in the 2011 comprehensive plan and in this facility plan to estimate the 2040 design population.

Figure 3-1. Indianola Population



The population for the future is assumed to follow the same general progression as in the past. See Table 3-1 for population projections.

TABLE 3-1
Population Projection Estimates

Year	Population
2020	16,657
2030	18,655
2040	20,491

3.3 EXISTING WASTEWATER FLOWS AND CHARACTERISTICS

Flow

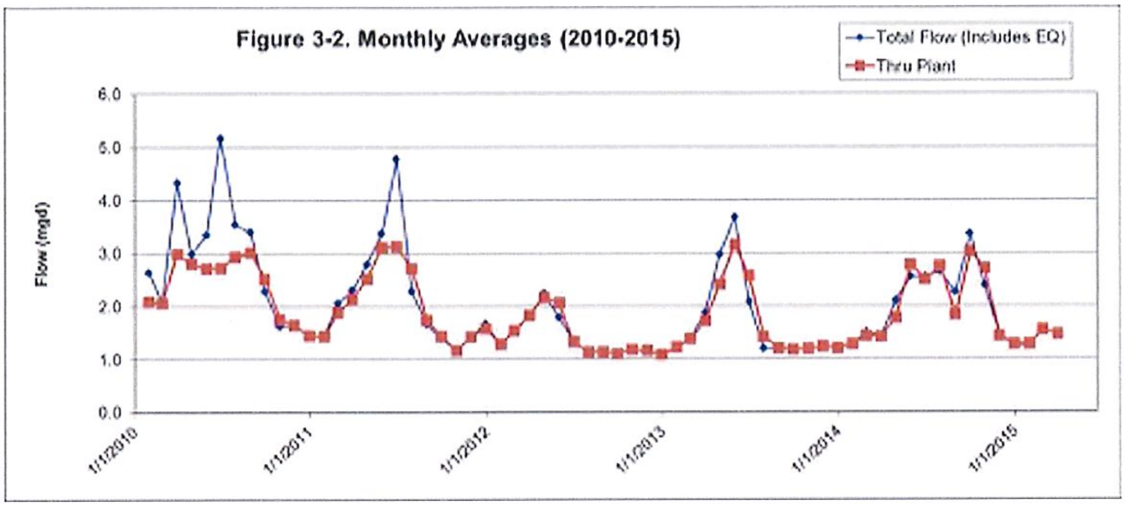
Table 3-2 is a summary of the total influent wastewater flows discharged to the North WWTF for the period from 2010 through 2015. Total annual, daily average, and maximum day wastewater flows are shown. Also shown in Table 3-2 is the calculated ratio of maximum day flows to daily average flows.

TABLE 3-2
Influent Wastewater Flow Data for 2002 thru 2007

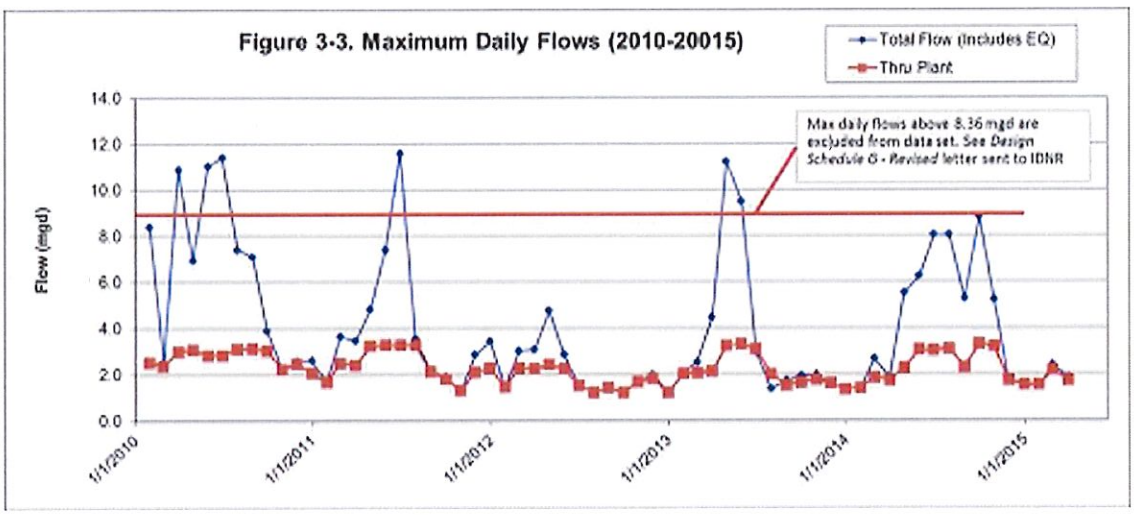
Year	Total Annual flow, MG	Daily Average Flow, MGD	Maximum Day Flow, MGD	Ratio of Max/Ave day
2010	1000	2.87	11.40	3.97
2011	799	2.19	11.58	5.28
2012	511	1.40	4.76	3.40
2013	623	1.70	11.21	6.58
2014	753	2.06	8.82	4.28
Average	737	2.04	9.55	4.70
Maximum	1000	2.87	11.58	6.58

(2015 data not shown in this table.)

The monthly average data from January 2010 thru March 2015 is charted in Figure 3-2. There are two sets of data plotted on this chart and several of the subsequent North WWTF flow charts. The data range titled "Total Flow (Includes EQ)" represents the entire wastewater flow that is conveyed to the North WWTF and is measured before excess flows are diverted to the equalization basin. The other data range titled "Thru Plant" only measures the flow that gets pumped through the plant after the diversion takes place.



The monthly data from January 2010 thru March 2015 was reviewed for max daily flows and is charted in Figure 3-3.



Average dry weather (ADW) is the daily average flow when the groundwater is at or near normal and runoff is not occurring. Average wet weather (AWW) is the daily average flow for the wettest thirty (30) consecutive days for mechanical plants. The maximum wet weather (MWW) is the total maximum flow received during any 24 hour period when groundwater is high and runoff is occurring. Peak hourly wet weather (PHWW) is the total maximum flow received during one hour when the groundwater is high, runoff is occurring, and the domestic, commercial and industrial flows are at their peak. Table 3-3 summarizes the ADW, AWW, MWW, and PHWW flows (through March 2015).

TABLE 3-3
Current Flows

Parameter	Value
ADW	1.56 MGD
Daily Average	2.02 MGD
AWW	5.17 MGD
MWW	8.36 MGD
PHWW (est.*)	13.67 MGD

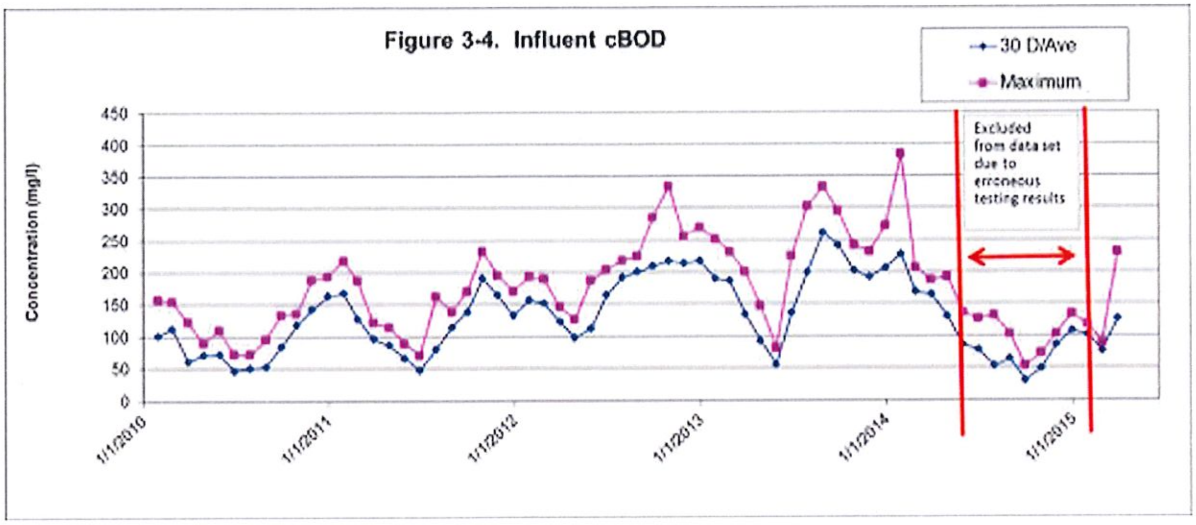
* PHWW flow estimated from sanitary sewer model. This flow was based on the maximum flow received during one hour when the groundwater is high, runoff is occurring, and domestic, commercial, and industrial flows are at their peak.

Since the initial submittal of flows and loads report to the IDNR, the MWW and PHWW flows have been revised. See the *Design Schedule G – Revised* letter and the corresponding IDNR concurrence letter dated July 13, 2017 in the appendices of this report for additional discussion and justification for these revisions. Also located in the appendix

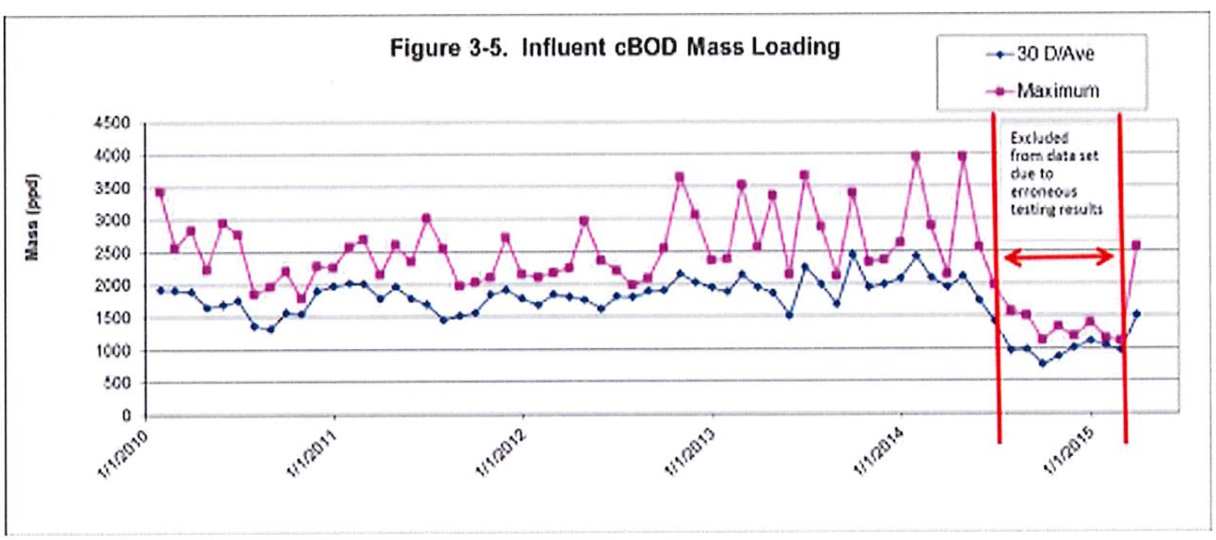
Biochemical Oxygen Demand

Biochemical oxygen demand (BOD) is a measure of the strength of pollutants or oxygen reduction potential of the waste stream. Since effluent regulations have required nitrification, regulators have allowed carbonaceous biochemical oxygen demand (cBOD) tests to be used. These tests inhibit the effects of nitrifying biomass in the sample. The nitrifying biomass can give false readings in the BOD test. Therefore, cBOD tests have been completed. This test is also allowed on the influent samples for simplicity. The cBOD test has been shown to underestimate BOD strength of the influent wastewater by 15% or even more. The relationship between cBOD and BOD is plant specific, and possibly seasonal. This should be confirmed on a case-by-case basis. Through a range of plant testing in which BOD5 tests have been run alongside cBOD tests at the existing Indianola North WWTF, a ratio of 0.78 to 1.0 has been established for the relationship between CBOD and BOD, respectively. These results are also in line with a Study of Raw Wastewater BOD and cBOD Relationship article that was published by the Water Environment Foundation in 2006. The City has run these cBOD/BOD tests as 24 hour composite samples at multiple times during this year, in an attempt to establish the most representative and accurate ratio between the two tests.

The cBOD data was reviewed for period from 2010-2015 and is shown in Figure 3-4. The cBOD concentration is typical of low to medium strength wastewater. It should be noted that data from June 2014 through February 2015 was thrown out since it is believed the deionized water used in the cBOD test was contaminated with copper from the copper still used. The contamination of residual copper can inhibit bacterial activity and skew results from the cBOD test. The Figure 3-4 compares the 30-day cBOD concentration averages and maximums.



cBOD mass loading is shown in Figure 3-5. The seasonal fluctuation has no clear pattern. This chart again compares the 30-day averages with the maximum daily loading. The cBOD has been relatively steady throughout the data set that was evaluated, although there has been some slight increase in cBOD concentrations. This could be due to some of the improvements that the City has done to eliminate overflows and bypasses in the collection system. These improvements are intended to help reduce the infiltration and inflow to the sanitary system during peak flow events. Another effect is the waste concentrations in sanitary flows will be higher than those with higher contributions of I/I, and the organic loading to the sanitary system will be increased.



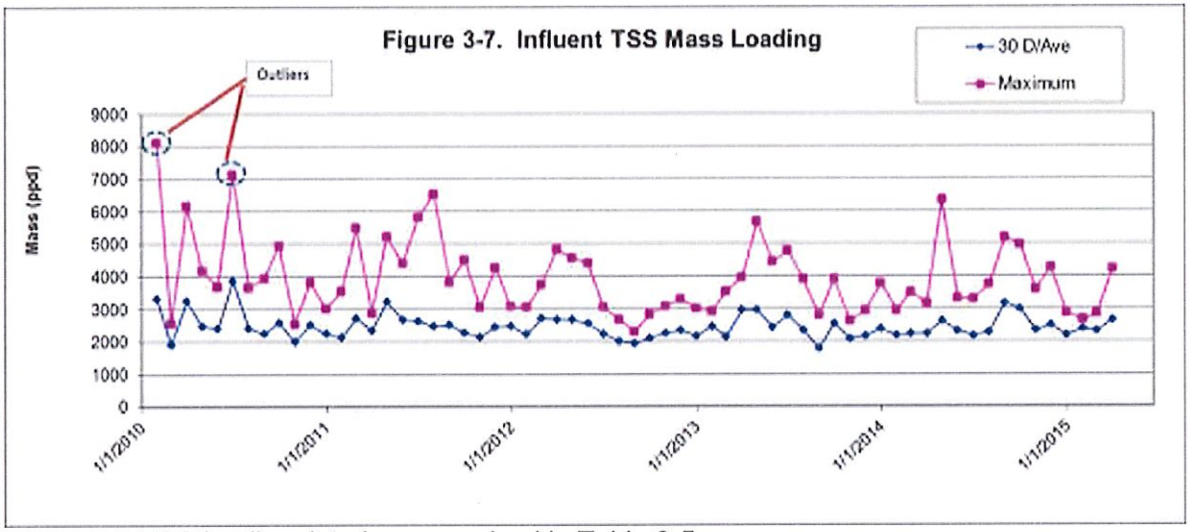
Organic loading data is summarized in Table 3-4.

TABLE 3-4
Current cBOD Loading (through 3/15)

Parameter	Value (ppd)
Average Month	1,840
Max Month	2,437
Max Day	3,952

Total Suspended Solids

Total suspended solids (TSS) data was reviewed from 2010 -2015. Figure 3-7 shows TSS loading of wastewater from January 2010 to March 2015. This chart compares the 30-day averages with the maximum daily loading. The January and June 2010 values are outliers.



TSS loading data is summarized in Table 3-5.

TABLE 3-5
Indianola North WWTF Historical TSS Loading
2010-2015

Parameter	Value (ppd)
Average Month	2,453
Max Month	3,859
Max Day	6,529*

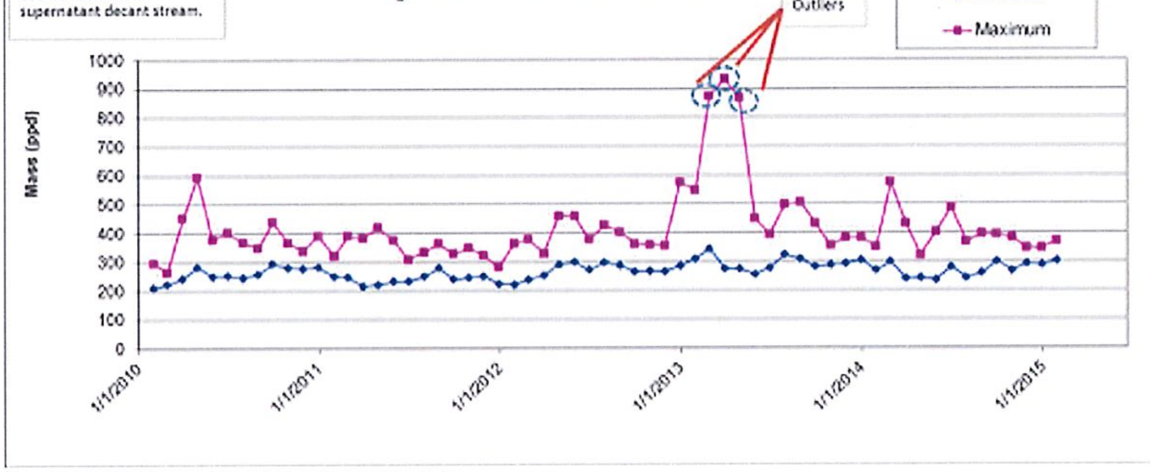
* Outliers: 8118 and 7130

Ammonia-Nitrogen and Total Kjeldahl Nitrogen

The influent ammonia-N data was reviewed from 2010 -2015. Figure 3-9 shows influent ammonia-N loading of wastewater from January 2010 to March 2015. This chart compares the 30-day averages with the maximum daily loading. The high ammonia-N maximum loadings from April – June of 2013 are uncharacteristic and nominally 50% higher than other data reported for the evaluation period. After further evaluation of these spikes, it was discovered that the likely cause of these spikes was false readings from an ammonia-selective electrode used to determine ammonia content of samples. These three spike readings should be discarded from the evaluation results. See the *Revised Ammonia Loadings* memo in the appendices for more discussion on this topic.

It was also discovered that influent ammonia readings have generally been artificially high due to the influence of a supernatant flow stream from the plant's biosolids storage tank. Testing has shown that the ammonia results are higher when decanting than without decanting. In addition to the supernatant from the biosolids storage tank, there is also a supernatant line from the anaerobic digesters that decants less frequently but also contributes to superficial ammonia readings. To establish the max month and max day ammonia loadings, typical peaking factors can be assumed. Metcalf and Eddy, 2003, *Wastewater Engineering, Treatment and Reuse, 4th Edition. Metcalf and Eddy* gives typical information on the ratio of averaged peak and low-constituent mass loadings to average mass loadings. Typical ammonia peaking ratios for max day to average and for max 30 day to average are 2.0 to 1 and 1.5 to 1, respectively. Therefore, the current max day ammonia load can be taken as 490 ppd and 368 ppd. As stated above, further information and discussion is included in the *Revised Ammonia Loadings* memo in the appendices.

Figure 3-9. Influent Ammonia



Total kjeldahl nitrogen (TKN) data was not regularly monitored in history. For facility planning purposes, TKN was estimated based off the typical relationship between ammonia-N and TKN. This relationship was estimated using Metcalf and Eddy, 2003, *Wastewater Engineering, Treatment and Reuse, 4th Edition.* Ammonia loading data is summarized in Table 3-6.

TABLE 3-6
Indianola North WWTF Historical Ammonia Loading 2010-2015

Parameter	Value (ppd)
Average Month	240
Max Month	368
Max Day	490

Population Equivalent Analysis

The flows and pollutant loadings were reviewed for data spanning January 2010 through March 2015. The monthly flows were reviewed for each year, and the months (typically November through February) where the groundwater table was historically near normal with little or no runoff occurring were selected for each year and averaged to find the ADW. The ADW from 2010 to 2015 is 1.56 MGD. This flow per capita (15,310 persons) is 102 gal/capita/day which is close to typical (typical value is 100 gal/capita/day for domestic wastewater flow). The cBOD loading during the same time period is 1,840 lbs/day and 2,437 lbs/day for average and max month conditions, respectively. The BOD loading during the same time period is 2,359 lbs/day and 3,124 lbs/day for average and max month conditions, respectively.The ratio is 1.32 max month/average. The average loading per capita is 0.15 lb/capita/day, which is on the low side of the typical value (0.17 lb/capita/day of BOD). The TSS loading during this time period is 2,453 lbs/day and 3,859 lbs/day for average and max month conditions respectively. This ratio is 1.57 max month/average. The average loading per capita is 0.16 lb/capita/day, which is slightly low but within the typical range (0.13-0.33 lb/capita/day). The ammonia-N loading during this time period is 240 lbs/day and 368 lbs/day for average and max month conditions respectively. This ratio is 1.53 max month/average. The average loading per capita is 0.016 lb/capita/day, which is within the typical range (0.011-0.026 lb/capita/day).

See Table 3-7 for a summary of the historic Flow, cBOD, BOD, TSS, and Ammonia loadings during the indicated time period.

TABLE 3-7
Indianola North WWTF Historical Flows and loads 2010-2015

Parameter	Value	Per Capita (Est)
Flow		
ADW	1.56 MGD	102 gal/cap/day
AWW	5.17 MGD	
MWW	8.36 MGD	
PHWW	13.67 MGD	
cBOD		
Average	1840 lbs/day	0.12 lbs/cap/day
Max Month	2437 lbs/day	
Max Day	3952 lbs/day	
BOD (calculated from cBOD influent data)		
Average	2359 lbs/day	0.15 lbs/cap/day
Max Month	3124 lbs/day	
Max Day	5067 lbs/day	
TSS		
Average	2453 lbs/day	0.16 lbs/cap/day
Max Month	3859 lbs/day	
Max Day	6529 lbs/day	
Ammonia-N		
Average	240 lbs/day	0.016 lbs/cap/day
Max Month	368 lbs/day	
Max Day	490 lbs/day	

Total Phosphorous

The Iowa Nutrient Strategy applies to Indianola. The State has adopted the Iowa Nutrient Strategy which will require Grade IV WWTPs to meet more stringent effluent requirements for Total Nitrogen and Phosphorus removal. In anticipation for these effluent limits, the City of Indianola has performed testing of their raw influent total phosphorous (TP). The testing to date has been performed in the spring of 2015 and the fall of 2017. Generally, the testing has shown that influent TP is within the range of 4.4 – 6.3 mg/L with an average value of 5.3 mg/L. These results are typical of domestic wastewater.

The average TP loading during the testing is 69 ppd. To establish max month and max day TP loadings, typical peaking factors can be assumed. Metcalf and Eddy, 2003, *Wastewater Engineering, Treatment and Reuse, 4th Edition. Metcalf and Eddy* gives typical information on the ratio of averaged peak and low-constituent mass loadings to average mass loadings. Typical TP peaking ratios for max day to average and for max 30 day to average are 2.2 to 1 and 1.5 to 1, respectively. Therefore, the current max day ammonia load can be taken as 152 ppd and 103 ppd.

TP loading data is summarized in Table 3-8.

TABLE 3-8
Indianola North WWTF TP Historical Loading 2015 & 2017

Parameter	Value (ppd)
Average Month	69
Max Month	103
Max Day	152

4.0 DESIGN CONDITIONS

4.1 DESIGN WASTEWATER FLOWS AND CHARACTERISTICS

Forecasting the design flows and loads to the WWTF will be similar to the determinations for the design population. The permanent residential flows can be linearly interpreted by extrapolating the flow based on the per capita flows determined for the existing permanent residential population. ADW flows, Daily Average flows, AWW flows, MWW flows and PHWW flows are estimated by ratios from historical data. Average, Max Month, and Max Day loadings for cBOD, BOD, TSS, Ammonia-N, TKN, and TP were also linearly interpreted by extrapolating the loadings on the per capita loading rates determined for the existing permanent residential population.

According to the zoning map of the city, the industrial area is approximately 102 acres. The area also includes vacant, currently classified as agricultural, available for future industrial use. The current industrial contribution to the wastewater plant is not currently broken out from commercial/domestic contribution due to the small amount of existing industry in Indianola. The City plans to increase the amount of land zoned for industry in the future. In the City's future land use plan, part of the industry zone is "Light Industrial" and the other portion is "Heavy Industrial." Assuming portions of this future land use gets developed by the design year, industrial design flows and loads will be accounted for in the facility plan. 1000 gallons per day per acre (gpd/acre) and 2000 gpd/acre were used to calculate flows for light and heavy industry, respectively. BOD, TSS, ammonia-N, and TP concentrations of industrial wastewater are assumed to be 300, 350 35, and 8.0 mg/L, respectively, according to the typical compositions of municipal wastewater. This is based on the fact that the industries will be required to pretreat their wastewater to the level of typical domestic flows as defined in the City's Sewer Ordinance. Permanent flows and loads shown in Table 4-1 include residential, industrial, and commercial sources.

TABLE 4-1
2040 Design Flows and Loads

Parameter	Residential Flow	Industrial Flow	Total
Flow (MGD)			
ADW	2.09	0.21	2.30
Daily Average	2.70	0.21	2.91
AWW	5.70	0.21	5.91
MWW	8.89	0.21	9.10
PHWW	14.20	0.21	14.41
cBOD (lbs/day)			
Average	2463	525	2988
Max Month	3262	525	3787
Max Day	5289	525	5815
BOD (lbs/day) – Calculated from cBOD data			
Average	3157	525	3683
Max Month	4181	525	4707
Max Day	6782	525	7307
TSS (lbs/day)			
Average	3283	613	3896
Max Month	5165	613	5778
Max Day	8738	613	9351
Ammonia-N (lbs/day)			
Ave Month	321	61	383
Max Month	493	61	554
Max Day	656	61	717
TKN (lbs/day) – Calculated from Ammonia-N data			
Average	494	94	588
Max Month	758	94	852
Max Day	1009	94	1103
TP (lbs/day)			
Average	92	14	106
Max Month	138	14	152
Max Day	203	14	217

APPENDIX A

REVISED SCHEDULE G – 12/15/17

Exhibit 11C

Iowa Department of Natural Resources Wastewater Construction Section

Construction Permit Application

SCHEDULE G, Treatment Project Design Data

DATE PREPARED 8/19/15	PROJECT IDENTITY City of Indianola WWTF Facility Plan	DNR USE PROJECT NO.
DATE REVISED 12/15/17		PERMIT NO.

1. Project Description	Wastewater Treatment Facility Plan

2. Design Flows	Present Year (2017)		Design Year (2040)	
Design Condition →	AWW (MGD)	MWW (MGD)	AWW (MGD)	MWW (MGD)
Domestic/Commercial Flow	1.56	1.56	2.09	2.09
Industrial				
Flow			0.21	0.21
Rated Flow			0.21	
Other Flow (specify)				
Infiltration/Inflow	3.61	6.80	3.61	6.80
Total				
Flow (Includes flow thru plant and to Equalization Basin.)	5.17	8.36	5.91	9.10
Rated Flow	5.17	8.36	5.91	9.10

Average Dry Weather Flow (ADW):	Peak Hourly Wet Weather Flow (PHWW):	Demographic Data:
1.56_ MGD (present year)	13.67_MGD (present year)	Population 15310_ (present year)
2.30_MGD (design year)	14.41_MGD (design year)	Population 20491_ (design year)

3. Organic Design Loadings		Present Year (2017)		Design Year (2040)	
Design Condition→		Max. 30 day (#/day)	Max. Day (#/day)	Max. 30 day (#/day)	Max. Day (#/day)
Domestic/Commercial	BOD_5	3124	5067	4181	6782
	TSS	3859	6529	5165	8738
	TKN	566	754	758	1009
Industrial	BOD_5			525	525
	TSS			613	613
	TKN			94	94
Other (Specify)	BOD_5				
	TSS				
	TKN				
Total	BOD_5	3124	5067	4707	7307
	TSS	3859	6529	5778	9351
	TKN	566	754	852	1103

4. Effluent Limitations		BOD_5		TSS		NH3-N (most stringent month)		Other		Other	
		Avg	Max	Avg	Max	Avg	Max	Avg	Max	Avg	Max
Operation Permit Effluent Limits*	mg/l	25.0	40.0	30.0	45.0	1.8	8.1				
	#/day	1232		1478		64.6					

*Date of Waste Load Allocation (WLA) determination: _ August 11, 2017 – Middle River

5. Major Industrial/Commercial contributors or Significant Industrial User:

Waste Contributors	Pre-Treat (Y/N)	Operation Hrs/ Day	Operation Days/ Week	Design Loadings Flow Ave. MGD	Flow Max. MGD	BOD5 #/day	Susp. Solids #/day	TKN #/day	Oil & Grease #/day	#/day

APPENDIX B

DESIGN SCHEDULE G – REVISED LETTER

HRGreen

February 24, 2017

Terry Kirschenman, P.E.
Wastewater Engineering Section
Wallace State Office Building
502 E. 9th St.
Des Moines, IA 50319

Re: City of Indianola WWTF – Design Schedule G - Revised

Dear Terry:

This letter is in response to your July 11, 2016 letter to the City of Indianola regarding Design Schedule G for the Indianola WWTF project. On behalf of the City of Indianola, HR Green is responding to each of the Department's questions and resubmitting a revised Design Schedule G. The July 11 letter is attached for reference.

HR Green, the City of Indianola and IDNR met in August 2016 to review the Facility Plan submittal and discussed a number of items related to the Schedule G information. Based on this review meeting a number of the responses to the IDNR questions were discussed. The following responses are numbered to directly match the questions asked by the IDNR in the July 11 letter.

1. & 2. Basis of design for the proposed PHWW and MWW flows.

A Hydraulic Model for the City of Indianola's sanitary sewer collection system was developed. The model was developed from field collection of data from approximately 1,600 sanitary manholes in the collection system and then developing the SewerGEMS model from this data and finally plugging in actual rain events and flow data to calibrate the model. The Hydraulic Model was specifically developed so the City can look at individual sanitary sewer components to evaluate their effectiveness to convey sewer flows. The Hydraulic Model is intended to be a working model that will be updated when conditions change or improvements are made to the sanitary sewer collection system.

The PHWW flow being proposed was developed based off of Iowa DNR Design Chapter 14.4.5.1, the total maximum flow received during one hour when the groundwater is high, runoff is occurring, and the domestic, commercial, and industrial flows are at their peak. The runoff flow component shall be adjusted to the storm event of two inches of rainfall in one hour. According to *Rainfall Frequency Atlas of the Midwest*, a rain event with a peak two inches in one hour corresponds to a 10-year storm recurrence interval.

Two sanitary sewer flow hydrographs were developed from the hydraulic model to show the PHWW flow estimate and the MWW flow estimate for a 10-year and 25-year storm recurrence interval. These hydrographs are included. The PHWW and MWW flows corresponding to a 10-year storm are included in the revised Schedule G. The hydraulic model has been set up to mimic the groundwater, runoff, and precipitation conditions for these two storm events and accurately predicts the ultimate collection system flows (i.e. MWW and PHWW) that would be conveyed to the WWTP.

HRGreen.com

Phone 515.278.2913 Fax 515.278.1846 Toll Free 800.728.7805
5525 Merle Hay Road, Suite 200, Johnston, Iowa 50131

HRGreen

The Facility Plan submitted includes a new WWTP at the Farm Site that abandons the existing North Equalization Basin. A smaller 2.0 million gallon equalization tank is included in the proposal at the new Farm Site. The new WWTP will be sized to allow 6.0 mgd through the plant with peak flow treatment (screening, primary sedimentation, and disinfection) planned for an additional 10.0 mgd of influent wastewater flow. The peak flow treatment process was selected over the store and treat (equalization) process to help stabilize the secondary treatment process for total nitrogen (N) removal. Subsequently, the North Equalization basin was not considered when calculating the MWW and PHWW flows. Using this approach, the MWW and PHWW flows have been revised since the previous Design Schedule G forms that were submitted to the Department.

There are several reasons that the revised PHWW and MWW flows are less than the originally submitted flows. First, the flows received at the WWTP during the facility planning evaluation period (January 2010 – March 2015) used to identify MWW and PHWW generally exceeded the 10-year recurrence level. First, most of the high flows that the WWTP received were during calendar years 2010 and 2011, both of which were wet years in central Iowa. If the specific time intervals corresponding to these high flows are cross referenced with precipitation data, it is evident that these high flows correspond to storm events that exceed the 10-year recurrence interval hyetograph. Secondly, the City of Indianola has made significant improvements to their collection system during the period that flows were evaluated for the facility plan. The City was under an Administrative Consent Order that was satisfied in 2014. With the four-phased project complete, the City replaced or lined approximately 25% of their collection system sewers and replaced or repaired approximately 35% of their sewer manholes since 2008 along with EQ Basin and other miscellaneous improvements. The City has seen a significant decrease in excessive I&I and SSO's since these improvements were made.

Due to these reasons, the PHWW and MWW flows previously submitted in Design Schedule G were overstated, but the revised values are now in line with the design requirements of the Department. The revised design MWW flow is 9.10 mgd and PHWW flow is 14.41 mgd resulting in a PHWW/MWW ratio of 1.58.

3. *Hydrograph for MWW event at 2 inch per hour rain at high groundwater conditions with runoff occurring.*

The 10 and 25-year recurrence interval storm event hydrographs for MWW and PHWW flows are attached.

4. *Percentage of flow tributary to South Plant Flow Equalization*

Approximately 23% of the collection system is tributary to the South Plant Flow Equalization based on land area.

5. *South Plant Flow Equalization Hydrograph*

An evaluation of the existing South Plant's equalization basin and hydrograph was not included in the hydraulic model since the equalization basin was recently expanded as part of the Administrative Consent Decree in 2013. The expanded equalization basin allows the South Plant to function without sanitary sewer overflow (SSO) at a design storm event. The South Plant equalization capacity was increased based on a design by V&K as part of the collection system projects. V&K would likely have this hydrograph. The maximum flow

HRGreen

from the existing South Plant catchment of approximately 650 gpm (i.e. South Plant lift station's capacity) is the contributing flow during an event and critical to the hydraulic model.

Therefore, in response to the Department's question about the adequacy of the South Plant Flow Equalization, the PHWW and MWW flow as listed in the revised Design Schedule G assumes a maximum flow of 650 gpm from the South Plant catchment and assumes the South Plant equalization basin was sized in conjunction with the DNR's design guidelines as it received a DNR Construction Permit.

6. *Design Schedule G, present year "rated flows".*

   Agree, this has been revised in the revised Design Schedule G. There are no "rated flows" anticipated.

7. *Design Schedule G, rated flows.*

   Agree, changed in revised Design Schedule G. There are no "rated flows" anticipated.

8. *Design Schedule G, rated AWW-30 flow.*

   Revised, see new Design Schedule G.

9. *Design Schedule G, includes PFT.*

   Revised, see new Design Schedule G. The term PFT is an acronym for peak flow treatment. As stated above, the new WWTP will be sized to allow 6.0 mgd through the plant with peak flow treatment (screening, primary sedimentation, and disinfection) planned for an additional 10.0 mgd of influent wastewater flow. The peak flow treatment process was selected over the store and treat (equalization) process to help stabilize the secondary treatment process for total nitrogen (N) removal.

10. *Design Schedule G, high peak day TKN.*

    Revised, see new Design Schedule G.

11. *Design Schedule G, BOD_5 vs. $CBOD_5$.*

    All values in revised Schedule G are reported in BOD_5 and not $CBOD_5$. The City has done extensive testing to prove correlation between these two values.

12. *Ratio between TKN and ammonia.*

    Revised, see new Design Schedule G.

13. *Design Schedule G, $CBOD_5$ and TSS limits.*

    Revised, see new Design Schedule G.

Phone 515.278.2913 Fax 515.278.1846 Toll Free 800.728.7805
5525 Merle Hay Road, Suite 200, Johnston, Iowa 50131

HRGreen

Please review these influent design loads and flows and the attached revised Design Schedule G for this project. If you have additional questions, please don't hesitate to contact us.

Sincerely,

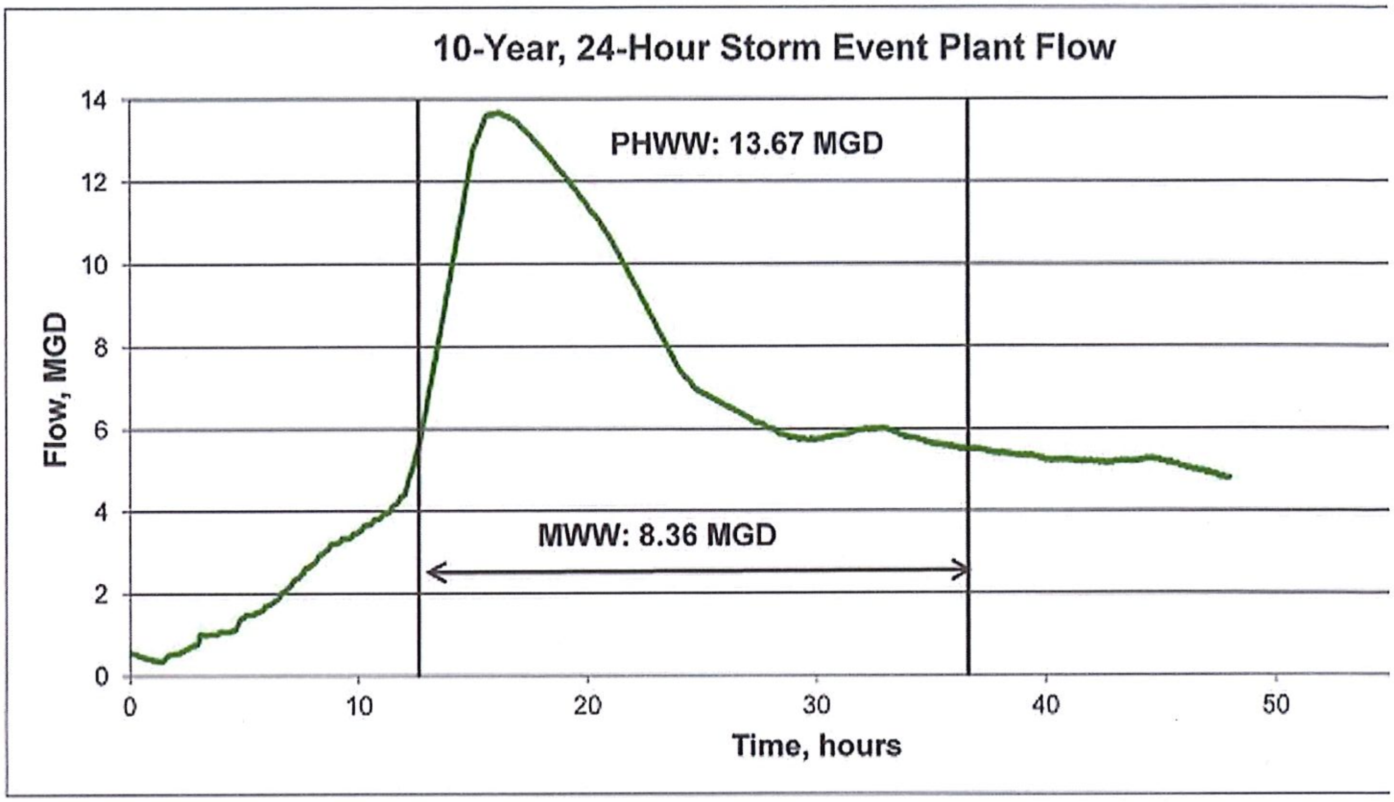
HR Green, Inc.

James R. Rasmussen

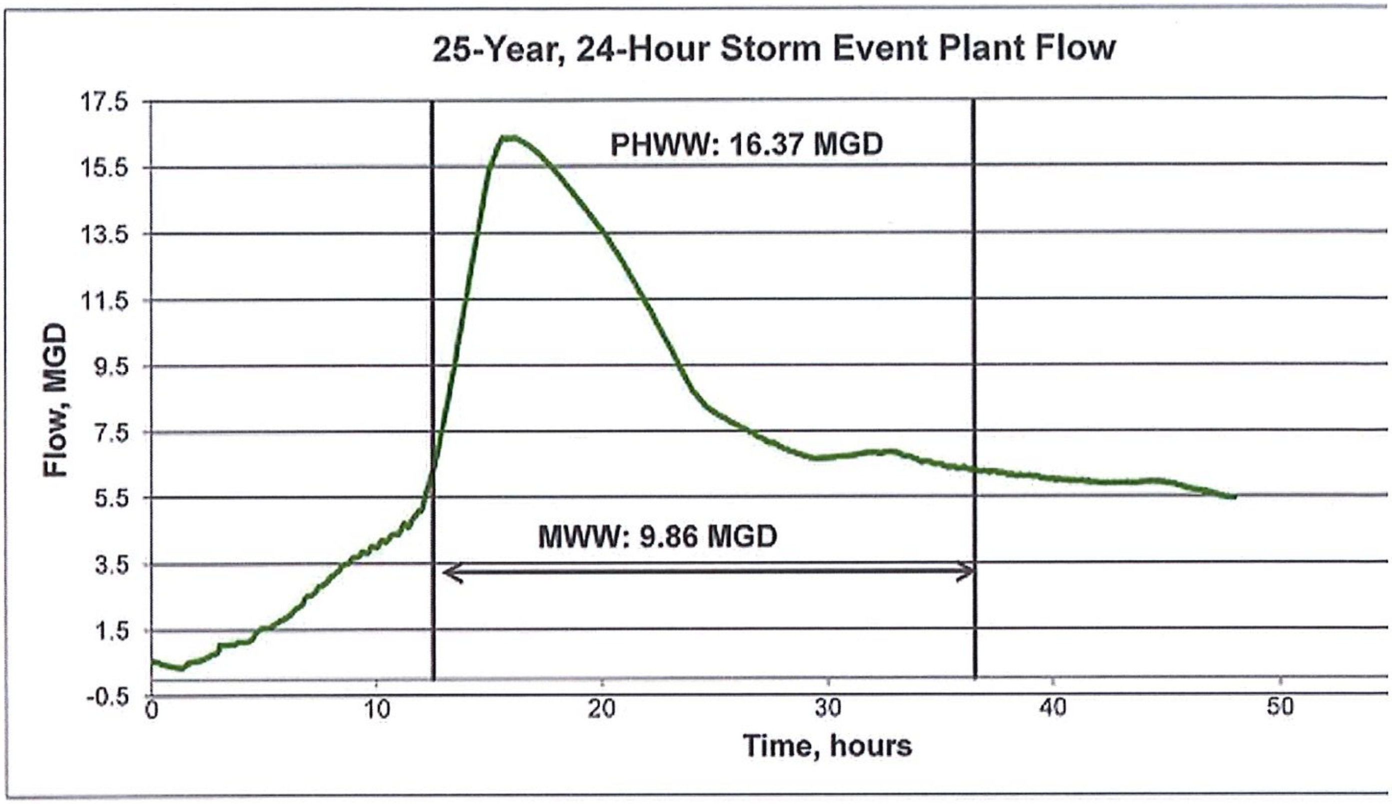
James R. Rasmussen, P.E.
Vice President

Enclosures: Revised Design Schedule G dated 02/24/17
10-year and 25-year recurrence interval storm event hydrographs
Letter from DNR dated July 11, 2016

cc: Ryan Waller, City Manager
Rick Graves, WWTF Superintendent
Tom Atkinson, IDNR FO5



10-Year, 24-Hour Storm Event Plant Flow
PHWW: 13.67 MGD
MWW: 8.36 MGD
Flow, MGD
Time, hours
0
2
4
6
8
10
12
14
0
10
20
30
40
50



25-Year, 24-Hour Storm Event Plant Flow
PHWW: 16.37 MGD
MWW: 9.86 MGD
Flow, MGD
Time, hours
17.5
15.5
13.5
11.5
9.5
7.5
5.5
3.5
1.5
-0.5
0
10
20
30
40
50

Fields of Opportunities

STATE OF IOWA

TERRY E. BRANSTAD, GOVERNOR
KIM REYNOLDS, LT. GOVERNOR

DEPARTMENT OF NATURAL RESOURCES
CHUCK GIPP, DIRECTOR

July 11, 2016

City of Indianola
Attention: Ryan Waller, City Manager
110 N. 1st Street, P.O. Box 299
Indianola, Iowa 50125

RE: City of Indianola WWTF - Design Schedule G dated 5/5/2016
Project No.: S2015-0386

Dear Mr. Waller,

On 10/20/2015, the Department concurred with the proposed ADW and AWW-30 flow estimates of 2.3 mgd and 5.91 mgd, respectively. Design Schedule G dated 8/19/2015 was revised on 10/22/2015. Soon after, additional guidance for completing Design Schedule G was given to the City. On 5/23/2016, the Department received Design Schedule G dated 5/5/2016. We offer the following comments on the enclosed construction permit application form:

1. To help us understand the basis of design for the proposed PHWW flow estimate of 17.11 mgd, please complete the following table for the alternative scenarios described below:

Design Schedule G PHWW Flow Estimates*

Source	New North Plant without South Plant Flow Equalization and without North Plant Flow Equalization (mgd)	New North Plant with South Plant Flow Equalization and without North Plant Flow Equalization (mgd)	New North Plant with South Plant Flow Equalization and with North Plant Flow Equalization (mgd)
Industrial	?	?	?
Domestic/Commercial	?	?	?
Maximum inflow from 2 inch/hour rain when the groundwater is high groundwater and runoff is occurring	?	?	?
Maximum infiltration rate at high groundwater conditions	?	?	?
Total	?	?	?

*PHWW is defined in Section 14.4.5.1 of Chapter 14.

502 EAST 9th STREET / DES MOINES, IOWA 50319-0034
PHONE 515-281-5918 FAX 515-281-8895 www.iowadnr.gov

2. To help us understand the basis of design for the proposed MWW flow estimate of 12.32 mgd and a peaking factor of only 1.39 (PHWW/MWW=1.39), please complete the following table for the alternative scenarios described below:

Design Schedule G MWW Flow Estimates*

Source	New North Plant without South Plant Flow Equalization and without North Plant Flow Equalization (mgd)	New North Plant with South Plant Flow Equalization and without North Plant Flow Equalization (mgd)	New North Plant with South Plant Flow Equalization and with North Plant Flow Equalization (mgd)
Industrial	?	?	?
Domestic/Commercial	?	?	?
Maximum infiltration/inflow during any 24 hour period	?	?	?
Total	?	?	?

*MWW is defined in Section 14.4.5.1 of Chapter 14.

3. Please submit hydrographs for the New North Plant depicting the anticipated diurnal variation in flow for a MWW flow event of 12.32 mgd and a 2 inch per hour rain at high groundwater conditions with runoff occurring.

4. What percentage of the collection system is tributary to the South Plant Flow Equalization?

5. Please submit the hydrographs used to determine adequacy of the South Plant Flow Equalization Basin. Will all flow to the South Plant Flow Equalization Basin be returned to the collection system for treatment within 24 hours?

6. Design Schedule G dated 5/5/2016, Item No. 2. The present year "rated flows" for the AWW-30 and MWW design parameters cannot be less than the averages for these design parameters. See Section 14.4.5.4 of Chapter 14 for additional guidance.

7. Design Schedule G dated 5/5/2016, Item No. 2. Why are rated flows presented for the plant when the industrial flow is not rated? What is the anticipated source of the rated flows cited in Design Schedule G dated 5/5/2016?

8. Design Schedule G dated 5/5/2016, Item No. 2. If the rated AWW-30 flow must be 6.0 mgd, the WLA for this project should be based on 6.0 mgd rather than 5.91 mgd.

9. Design Schedule G dated 5/5/2016, Item No. 2. What does "18.0 (includes PFT)" mean and/or represent?

10. Design Schedule G dated 5/5/2016, Item No. 3. At this time, the Department can concur with the proposed TSS and TKN design loadings of 5,778 lb/day and 820 lbs/day, respectively. However, the high peak day TKN loadings observed in 2013 have been noted and remain a possibility, especially during wet weather. This is reflected in the design estimate of 2,013 lbs TKN/day.

11. Design Schedule G dated 5/5/2016, Item No. 3. The design organic loadings presented must be revised to reflect the influent BOD_5 design loadings in terms of BOD_5 rather than $CBOD_5$. Previously, Design Schedule G dated 10/22/2015 presented influent BOD_5 design loadings.

12. From Table No. 2 of Ten States Standards, the ratio between the TKN and ammonia nitrogen is 1.6. This places the present influent TKN loading at approximately 528 lbs/day rather than 343 lbs/day. See Design Schedule G dated 10/22/2015.

13. Design Schedule G dated 5/5/2016, Item No. 4. If the AWW-30 design flow is 5.91 mgd, the applicable $CBOD_5$ and TSS limits for this project will be 1,232 lbs/day and 1,478 lbs/day, respectively, rather than 521 lbs/day and 626 lbs/day.

Following your response to the above questions and our concurrence of the influent design loadings per Step 12 of the wastewater construction permitting procedures, the City of Indianola must prepare an Antidegradation Alternatives Analysis for submittal to the Department for review and approval.

DNR's design standards, construction permit application forms, and wastewater construction permitting procedures are available at the following web site:
http://www.iowadnr.gov/InsideDNR/RegulatoryWater/WastewaterConstruction/ConstructionPermits.asp
x

Should you have any questions, please call.

Sincerely,

Terry Kirschenman

Terry L. Kirschenman, P.E.
Wastewater Engineering Section

Enclosure: Design Schedule G dated 5/5/2016

c: James R. Rasmussen, HR Green, Inc., 5525 Merle Hay Road, Suite 200, Johnston, IA 50131
Tom Atkinson, IDNR FO 5
Sewage File 6-91-33-0-01
Project File S2015-0386
Eric Wiklund, IDNR NPDES Permits Section

Iowa Department of Natural Resources
Wastewater Section
Construction Permit Application
SCHEDULE G, Treatment Project Design Data
Exhibit 11C

DNR USE ONLY
Project No.
Permit No.

Date Prepared	Project Identity
5/5/16	City of Indianola Wastewater Treatment Plant
Date Revised	

1. Project Description — Facility Plan Submittal for Indianola Wastewater Treatment Plant

2. Design Flows	Present Year (2015)		Design Year (2040)	
Design Condition →	AWW (MGD)	MWW (MGD)	AWW (MGD)	MWW (MGD)
Domestic/Commercial Flow	2.02	2.02	2.70	2.70
Industrial				
Flow			0.21	0.21
Rated Flow				
Other Flow (specify)				
Infiltration/Inflow	3.15	9.56	3.0	9.41
Total				
Flow	5.17	11.58	5.91	12.32
Rated Flow	4.32	4.32	6.0	18.0 (includes PFT)

Average Dry Weather Flow (ADW):	Peak Hourly Wet Weather Flow (PHWW):	Demographic Data:
1.56 MGD (present year)	16.37 MGD (present year)	Population 15000 (present year)
2.30 MGD (design year)	17.11 MGD (design year)	Population 20491 (design year)

3. Organic Design Loadings		Present Year (2015)		Design Year (2040)	
	Design Condition →	Max. 30 day (#/day)	Max. Day (#/day)	Max. 30 day (#/day)	Max. Day (#/day)
Domestic/Commercial	BOD_5	2437 CBOD	3952 CBOD	3262 CBOD	5289 CBOD
	TSS	3859	6529	5165	8738
	TKN	343	932	725	1919
Industrial	BOD_5			525 CBOD	525 CBOD
	TSS			613	613
	TKN			94	94
Other (Specify)	BOD_5				
	TSS				
	TKN				
Total	BOD_5	2437 CBOD	3952 CBOD	3787 CBOD	5815 CBOD
	TSS	3859	6529	5778	9351
	TKN	343	932	820	2013

4. Effluent Limitations		BOD_5		TSS		NH_3-N (most stringent month)		Other		Other	
		Avg	Max	Avg	Max	Avg	Max	Avg	Max	Avg	Max
Operation Permit Effluent Limits*	mg/l	25.0	40.0	30.0	45.0	1.0	1.8	389	629	1514	1514
	#/day	521.0	834.0	626.0	938.0	48.6	85.6	1915 6	3099 6	74609	7460 9

*Date of Waste Load Allocation (WLA) determination: January 21, 2016

**Effluent Limitations entered shall be the more stringent value between the existing NPDES Permit and the WLA or an approved antidegradation analysis

5. Major Industrial/Commercial contributors or Significant Industrial User: N/A

Waste Contributors	Pre-Treat (Y/N)	Operation		Design Loadings						
				Flow		BOD5 #/day	Susp. Solids #/day	TKN #/day	Oil & Grease #/day	#/day
		Hrs/ Day	Days/ Week	Ave. MGD	Max. MGD					

6. SCHEDULE G SUPPLEMENTAL CHECKLIST MUST ACCOMPANY THIS FORM

APPENDIX C

IDNR FLOWS & LOADS CONCURRENCE LETTER

Fields of Opportunities

STATE OF IOWA

TERRY E. BRANSTAD, GOVERNOR
KIM REYNOLDS, LT. GOVERNOR

DEPARTMENT OF NATURAL RESOURCES
CHUCK GIPP, DIRECTOR

July 13, 2017

City of Indianola
Attention: Ryan Waller, City Manager
110 N. 1st Street, P.O. Box 299
Indianola, Iowa 50125

RE: City of Indianola WWTF - Design Schedule G dated 2/24/2017
Project No.: S2015-0386

Dear Mr. Waller,

The Iowa Department of Natural Resources concurs with the following design loadings per Step 12 of the wastewater construction permitting procedures. An estimate for the MSDWW flow was provided on 5/1/2017.

Design Flows and Loadings (Design Year 2040)

Design Flows (mgd)		Maximum 30 Day Design Loadings (lbs/day)	
ADW	2.3	BOD5	4,707
AWW-30	5.91	TSS	5,778
Maximum Seven Day Wet Weather*	8.23	TKN	820
Maximum Wet Weather	9.10		
Peak Hourly Wet Weather	14.41		

*Maximum Seven Day Wet Weather flow is the daily average flow for the wettest seven (7) consecutive days.

DNR's design standards, construction permit application forms, and wastewater construction permitting procedures are available at the following web site:
http://www.iowadnr.gov/Environmental-Protection/Water-Quality/Wastewater-Construction/Construction-Permits

Nothing in this letter shall be construed as an approval to reject wet weather flow equalization as a viable alternative for handling excess flows while meeting the limits in the 1/21/2016 Waste Load Allocation. Should you have any questions, please call. My telephone number is 515-725-8422.

Sincerely,

Terry Kirsch

Terry L. Kirschenman, P.E.
Wastewater Engineering Section

Enclosure: Design Schedule G dated 2/24/2017

c: Joe Frankl, HR Green, Inc., 5525 Merle Hay Road, Suite 200, Johnston, IA 50131
Tom Atkinson, IDNR FO 5
Sewage File 6-91-33-0-01
Project File S2015-0386

Iowa Department of Natural Resources
Wastewater Section
Construction Permit Application
SCHEDULE G, Treatment Project Design Data
Exhibit 11C

DNR USE ONLY
Project No. ____________
Permit No. ____________

Date Prepared	Project Identity
5/5/16	City of Indianola Wastewater Treatment Plant
Date Revised 02/24/17	

1. Project Description — Facility Plan Submittal for Indianola Wastewater Treatment Plant

2. Design Flows	Present Year (2020)		Design Year (2040)	
Design Condition →	AWW (MGD)	MWW (MGD)	AWW (MGD)	MWW (MGD)
Domestic/Commercial Flow	1.56	1.56	2.09	2.09
Industrial				
Flow			0.21	0.21
Rated Flow			0.21	0.21
Other Flow (specify)				
Infiltration/Inflow	3.61	6.80	3.61	6.80
Total				
Flow	5.17	8.36	5.91	9.10
Rated Flow	5.17	8.36	5.91	9.10

Average Dry Weather Flow (ADW): 1.56 MGD (present year); 2.30 MGD (design year)

Peak Hourly Wet Weather Flow (PHWW): 13.67 MGD (present year); 14.41 MGD (design year)

Demographic Data: Population 15000 (present year); Population 20491 (design year)

3. Organic Design Loadings		Present Year (2015)		Design Year (2040)	
Design Condition→		Max. 30 day (#/day)	Max. Day (#/day)	Max. 30 day (#/day)	Max. Day (#/day)
Domestic/Commercial	BOD_5	3124	5067	4181	6782
	TSS	3859	6529	5165	8738
	TKN	343	932	706	1919
Industrial	BOD_5			525	525
	TSS			613	613
	TKN			94	94
Other (Specify)	BOD_5				
	TSS				
	TKN				
Total	BOD_5	3124	5067	4707	7307
	TSS	3859	6529	5778	9351
	TKN	528	932	820	2013

4. Effluent Limitations		BOD_5		TSS		NH_3-N (most stringent month)		Other		Other	
		Avg	Max	Avg	Max	Avg	Max	Avg	Max	Avg	Max
Operation Permit	mg/l	25.0	40.0	30.0	45.0	1.0	1.8				
Effluent Limits*	#/day	1232		1478		48.6	85.6				

*Date of Waste Load Allocation (WLA) determination: January 21, 2016

**Effluent Limitations entered shall be the more stringent value between the existing NPDES Permit and the WLA or an approved antidegradation analysis

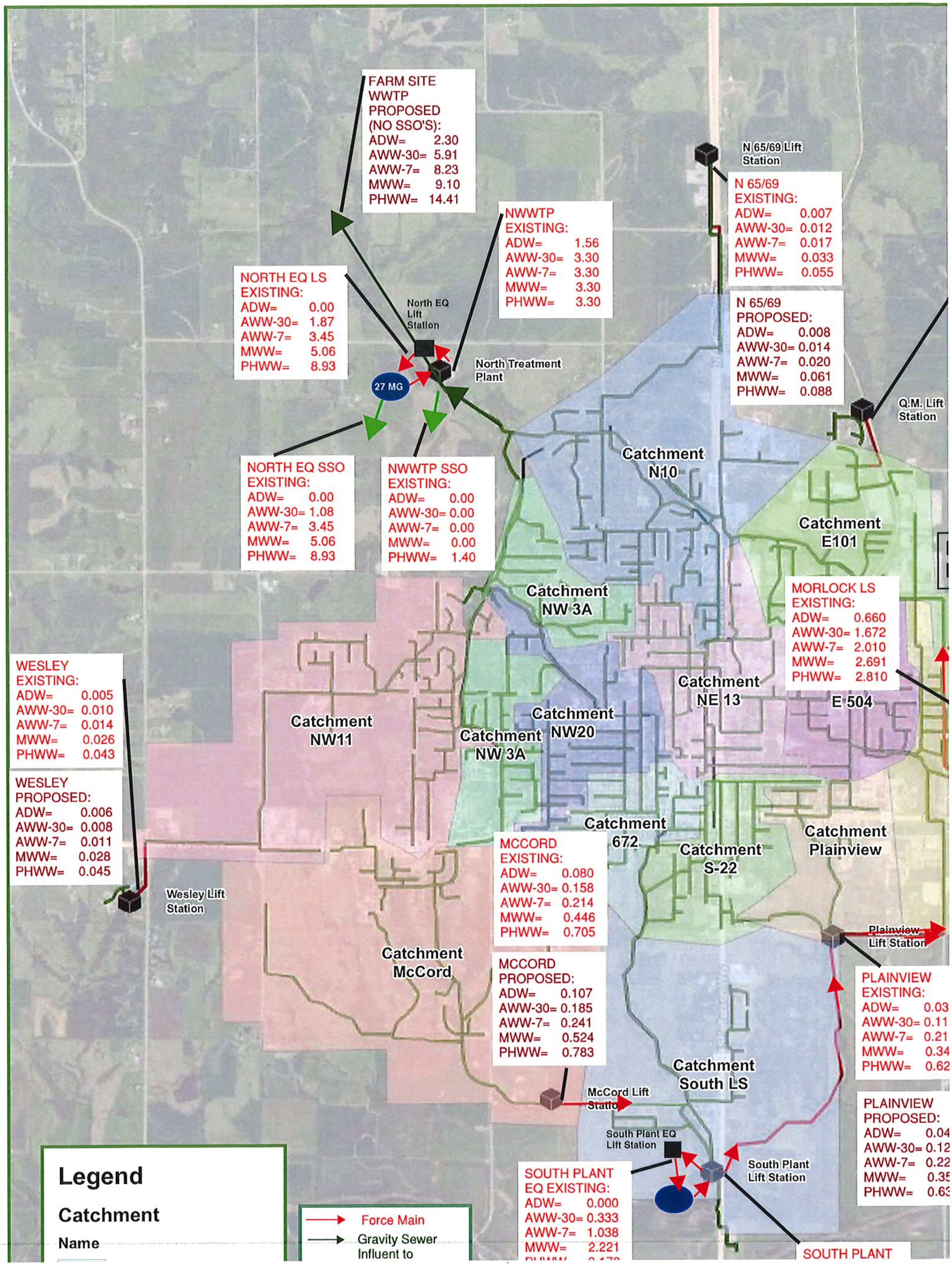
5. Major Industrial/Commercial contributors or Significant Industrial User: N/A

Waste Contributors	Pre-Treat (Y/N)	Operation		Design Loadings						
				Flow		BOD5 #/day	Susp. Solids #/day	TKN #/day	Oil & Grease #/day	#/day
		Hrs/ Day	Days/ Week	Ave. MGD	Max. MGD					

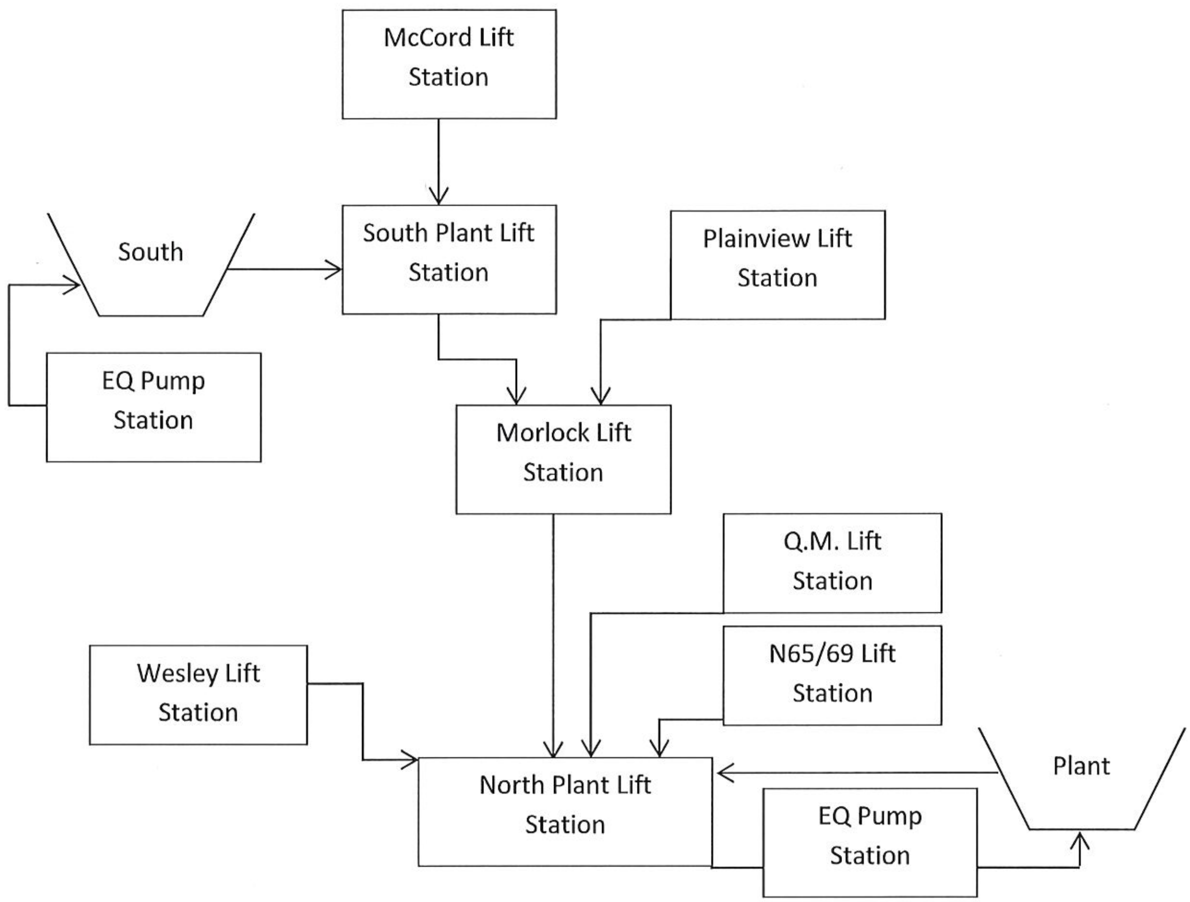
6. SCHEDULE G SUPPLEMENTAL CHECKLIST MUST ACCOMPANY THIS FORM

APPENDIX D

INDIANOLA ENTIRE SYSTEM FLOW BREAKDOWN SCHEMATIC



FARM SITE WWTP PROPOSED (NO SSO'S):
ADW= 2.30
AWW-30= 5.91
AWW-7= 8.23
MWW= 9.10
PHWW= 14.41
N 65/69 Lift Station
N 65/69 EXISTING:
ADW= 0.007
AWW-30= 0.012
AWW-7= 0.017
MWW= 0.033
PHWW= 0.055
NWWTP EXISTING:
ADW= 1.56
AWW-30= 3.30
AWW-7= 3.30
MWW= 3.30
PHWW= 3.30
NORTH EQ LS EXISTING:
ADW= 0.00
AWW-30= 1.87
AWW-7= 3.45
MWW= 5.06
PHWW= 8.93
North EQ Lift Station
N 65/69 PROPOSED:
ADW= 0.008
AWW-30= 0.014
AWW-7= 0.020
MWW= 0.061
PHWW= 0.088
North Treatment Plant
27 MG
Q.M. Lift Station
NORTH EQ SSO EXISTING:
ADW= 0.00
AWW-30= 1.08
AWW-7= 3.45
MWW= 5.06
PHWW= 8.93
NWWTP SSO EXISTING:
ADW= 0.00
AWW-30= 0.00
AWW-7= 0.00
MWW= 0.00
PHWW= 1.40
Catchment N10
Catchment E101
Catchment NW 3A
MORLOCK LS EXISTING:
ADW= 0.660
AWW-30= 1.672
AWW-7= 2.010
MWW= 2.691
PHWW= 2.810
WESLEY EXISTING:
ADW= 0.005
AWW-30= 0.010
AWW-7= 0.014
MWW= 0.026
PHWW= 0.043
Catchment NE 13
E 504
Catchment NW11
Catchment NW20
Catchment NW 3A
WESLEY PROPOSED:
ADW= 0.006
AWW-30= 0.008
AWW-7= 0.011
MWW= 0.028
PHWW= 0.045
Catchment 672
MCCORD EXISTING:
ADW= 0.080
AWW-30= 0.158
AWW-7= 0.214
MWW= 0.446
PHWW= 0.705
Catchment S-22
Catchment Plainview
Wesley Lift Station
Plainview Lift Station
Catchment McCord
MCCORD PROPOSED:
ADW= 0.107
AWW-30= 0.185
AWW-7= 0.241
MWW= 0.524
PHWW= 0.783
PLAINVIEW EXISTING:
ADW= 0.03
AWW-30= 0.11
AWW-7= 0.21
MWW= 0.34
PHWW= 0.62
Catchment South LS
McCord Lift Station
PLAINVIEW PROPOSED:
ADW= 0.04
AWW-30= 0.12
AWW-7= 0.22
MWW= 0.35
PHWW= 0.63
South Plant EQ Lift Station
South Plant Lift Station
SOUTH PLANT EQ EXISTING:
ADW= 0.000
AWW-30= 0.333
AWW-7= 1.038
MWW= 2.221
SOUTH PLANT
Legend
Catchment
Name
Force Main
Gravity Sewer Influent to



Lift Station Flow Diagram

Appendix C - Revised Ammonia Loads

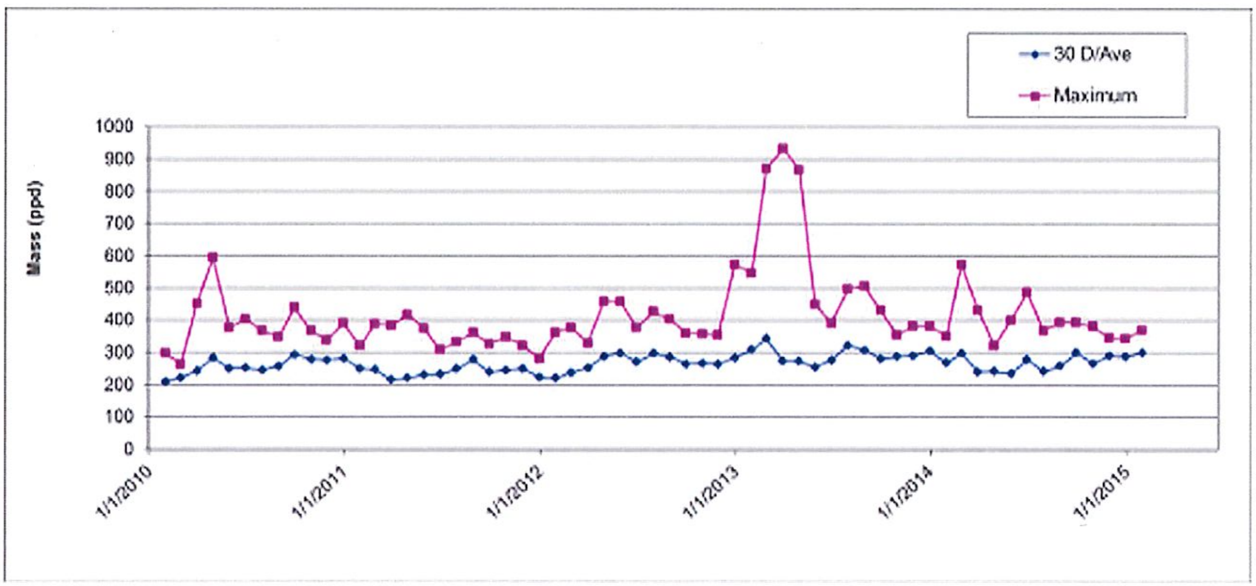
HRGreen

MEMO

To:	Iowa DNR
From:	Joe Frankl, P.E. - HR Green
Subject:	Indianola Wastewater Facility Planning – Revised Ammonia Loadings
Project No.	40150016
Date:	September 2017

Background

After careful analysis and review, the determination has been made that the ammonia design loadings proposed in the Indianola Facility Plan for Wastewater Treatment Facilities dated April 2016 are too high and non-representative of actual expected design conditions. Figure 3-7 below shows the 30 day average and maximum day ammonia mass loadings during the evaluation period (January 2010 – March 2015).



Ammonia Spikes

Three maximum day ammonia loadings during the months of April, May, and June are very easily identifiable on this figure. Ammonia loadings for these three months were reported as 871, 932 and 866 ppd, respectively, which are all nominally 50% higher than other data reported for the 2010-2015 evaluation period.

HR Green went back and evaluated these three data points. The first assumption was that these spikes may be related to storm events and that faulty calculation methods for ammonia total loadings created the spikes. However, this assumption proved to be incorrect as the spikes in ammonia loading for May and June did not correspond to the flow events. The second assumption was that the spikes in ammonia concentrations may be somewhat related to return from the EQ lagoon, but that also did not prove out. The May and June spikes in concentration did occur during periods of return flow from the lagoon, but there were other periods of higher return rate that had much lower ammonia concentrations.

So a more detailed evaluation of the time frame in question was made and the City was consulted. After discussions with the City, it became clear that the likely cause of the three spikes was due to trouble with the ammonia-selective electrode that is used by staff to determine the ammonia content of samples. Past operational reports indicate that staff was having troubles with the electrode during all three of the high readings. Some of the specific observations and notes made in the past operational reports indicated that plant staff was frequently replacing membrane and solution of the electrode and that accurate calibration was not possible during this three-month stretch. A service repair technician was eventually called out to repair the electrode. Further evidence that the ammonia results were false readings was the fact that there were no BOD5 spikes in concentration or loading that would correlate to the high ammonia readings. If the spikes in ammonia loading were true, there would be a corresponding spike in BOD5 loading that would be evident. Although these three spike readings were written down and skew the data, they are false readings and should be discarded from the evaluation results.

<u>Supernatant Decant</u>

It was also discovered that influent ammonia readings have generally been artificially high due to the influence of a supernatant flow stream from the plant's biosolids storage tank. Anaerobic digested sludge creates a reject water stream that at typical POTW's can contribute 15 to 20% of the ammonia load to a wastewater treatment plant. The supernatant flow from Indianola's biosolids storage tank is routed to a plant sewer that directs flow back to the Primary Pump Station near the head of the plant. Plant staff typically take composite samples from a splitter structure just upstream of the primary clarifiers, but downstream of the Primary Pump Station. Therefore, the majority of influent ammonia results used in the evaluation period were artificially too high due to the presence of this ammonia-rich supernatant flow stream. Additional testing has shown that the ammonia results are higher when decanting (estimated to be about two-thirds of the time) than without decanting. In addition to the supernatant from the biosolids storage tank, there is also a supernatant line from the anaerobic digesters that decants less frequently but also contributes to superficial ammonia readings. To demonstrate a typical daily contribution while the decant supernatant is flowing, the typical decant flow rate multiplied by the typical decant ammonia concentration while converting to pounds per day is roughly 39 ppd (0.012mgd x 388 mg/L NH_3 x 8.34 = 39 pounds/day of NH_3-N). As compared to the average month ammonia loading of 266 ppd, the supernatant loading is approximately 15%.

In order to find the true ammonia loading in the City's raw wastewater, typical per capita values of domestic wastewater can be applied. It should be noted that Indianola is generally a light to moderately loaded community with no significant industrial contribution. For example, the average BOD per capita loading for the community is 0.15 ppd/capita; the typical BOD per capita loading and Iowa DNR design guideline is 0.17 ppd/capita. Meltcalf and Eddy reports that a general BOD per capita loading range of 0.11-0.26 ppd/capita is common with 0.18 ppd/capita is the most typical value. Metcalf and Eddy also reports that typical ammonia per capita loading ranges between 0.011-0.026 while 0.017 ppd/capita is the most typical value. Since Indianola is a lightly loaded community, it can be assumed that the per capita ammonia loading is approximately 0.015 ppd/capita for an average loading of **245 ppd** of ammonia. This is about 9% less than the current average ammonia value of 266 ppd which is listed in the facility plan. This decrease would be in line with subtracting typical anaerobic digestion decant from the influent loading.

To establish the max month and max day ammonia loadings, typical peaking factors can be assumed. Metcalf and Eddy Figure 3-8 gives typical information on the ratio of averaged peak and low-constituent mass loadings to average mass loadings. Typical ammonia peaking ratios for max day to average and for max 30 day to average are 2.0 to 1 and 1.5 to 1, respectively. Therefore, the current max day ammonia load can be taken as **490 ppd** and **368 ppd**. The 2040 Design Flows and Loadings Table 5-2 can be revised as shown below. As stated in the facility plan, these design flows and loadings include future contribution from commercial, residential, and industrial growth. The revised ammonia loadings also affect the TKN loadings as shown below.

Conclusion

These revised design values below will be incorporated into a revised Antidegradation Alternatives Analysis as well as a revised Facility Plan.

Table 3-6 Indianola North WWTF Historic Loads 2010-2015

Parameter	Value	Per Capita (Est)
Ammonia-N		
Average	266 lbs/day	0.017 lbs/cap/day
Max Month	343 lbs/day	
Max Day	932 lbs/day	

Table 3-6 Indianola North WWTF Historic Loads 2010-2015 (Revised)

Parameter	Value	Per Capita (Est)
Ammonia-N		
Average	240 lbs/day	0.016 lbs/cap/day
Max Month	368 lbs/day	
Max Day	490 lbs/day	

Table 5-2 2040 Design Flows and Loadings

Parameter	**Residential**	**Industrial**	**Total**
Ammonia-N (lbs/day)			
Ave Month	356	61	417
Max Month	472	61	533
Max Day	1247	61	1309
TKN (lbs/day)			
Average	548	94	642
Max Month	725	94	820
Max Day	1919	94	2013

Table 5-2 2040 Design Flows and Loadings (Revised)

Parameter	Residential	Industrial	Total
Ammonia-N (lbs/day)			
Ave Month	321	61	383
Max Month	493	61	554
Max Day	656	61	717
TKN (lbs/day)			
Average	494	94	588
Max Month	758	94	852
Max Day	1009	94	1103

Appendix D - South Plant Equalization Capacity

MEMO

To:	Iowa DNR
From:	Joe Frankl, P.E. - HR Green
Subject:	Indianola Wastewater Facility Planning – South WWTP Equalization Capacity
Project No.	40150016
Date:	April 2018

Background

The South Wastewater Plant (WWTP) in Indianola was taken out of service in the 1990's and converted to earthen equalization basin and a sanitary lift station. The earthen equalization basin capacity was approximately 9.0 million gallons to equalize peak flows from the south collection system. The South WWTP Lift Station pumped flows to the Morlock Lift Station and the Morlock Lift Station pumped the sanitary sewer flows on to the North Wastewater Treatment Plant (NWWTP).

As part of the Administrative Order in 2009 to make improvements to the collection system, the South WWTP Lift Station equalization basins were expanded in 2013 to approximately 13.0 million gallons. Other improvements such as new splitter box, new influent sewer, new EQ Basin pumps and controls, and flow meters were also part of the improvements project. The additional equalization basin volume and other improvements were intended to eliminate SSO's from the South Lift Station.

As the City and HR Green developed the Morlock Lift Station Improvements project, it was determined that during peak flow events when Morlock was surcharged, the South Lift Station was actually shut off from continuing to send flow to the Morlock catchment. Obviously, this operational configuration put more stress on the equalization volume at the South Lift Station and there was a higher risk of SSO's at that lift station and EQ Basin.

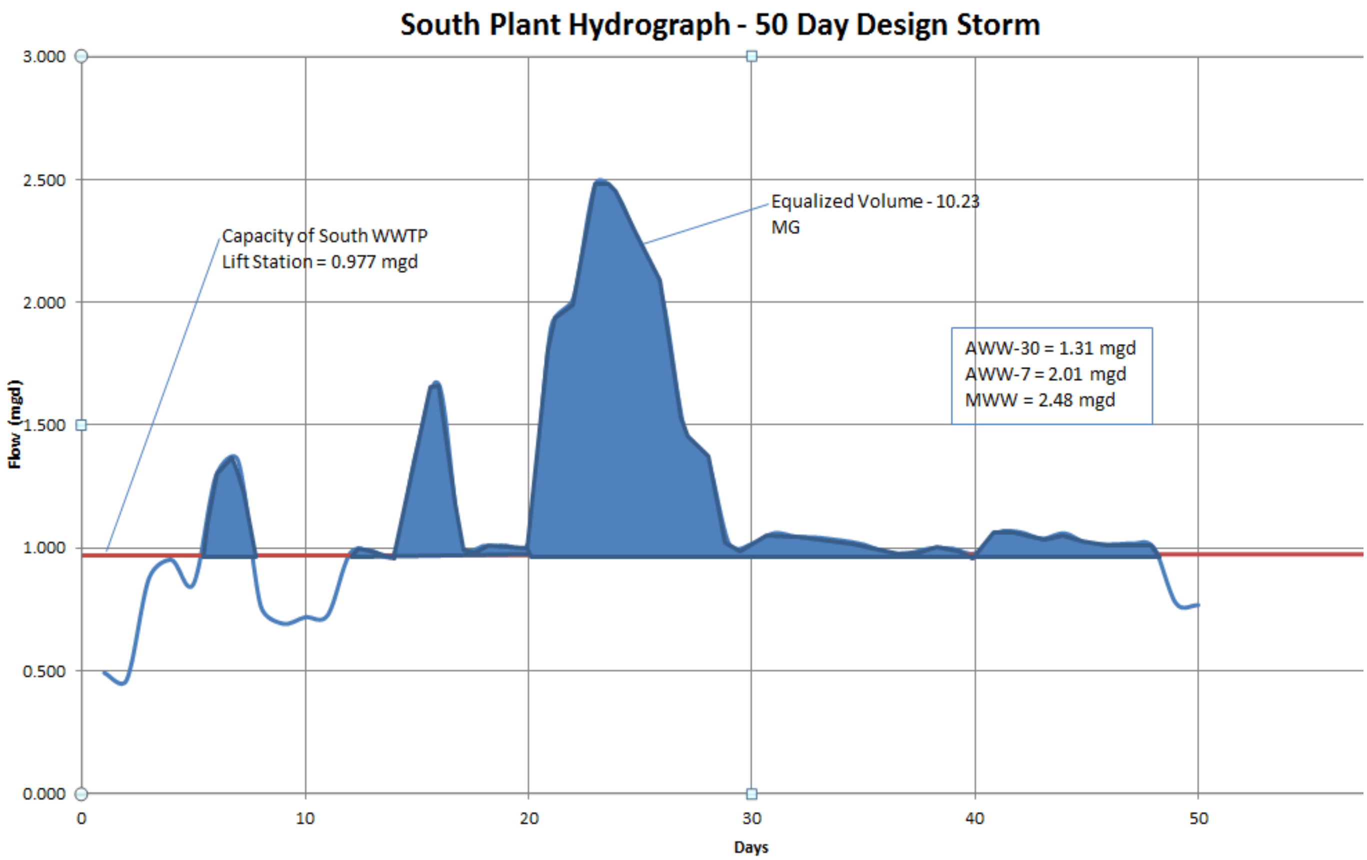
When the current Morlock Lift Station Improvements are complete (planned for July of 2018) the new Morlock Lift Station will be able to handle a 25 year peak flow event without shutting flows off from the South WWTP Lift Station and overloading the 13.0 million gallon equalization basins. The purpose of this Technical Memorandum is to analyze the storage capacity at the South WWTP Lift Station, Equalization Basin, and Lagoon Pump Station.

Evaluation

See the attached Appendix for current and design flows of the Indianola collection system. The South WWTP Lift Station conveys flow from the South LS catchment as well as the McCord catchment. The discharge from the South WWTP Lift Station Force Main discharges into the Morlock catchment and runs parallel to the Plainview discharge force main. The capacity of the South WWTP Lift Station is reportedly approximately 0.98 mgd while the capacity of the Lagoon Pump Station is approximately 5.76 mgd.

In order to evaluate the adequacy of the South WWTP Lift Station, a 50-day design storm event was considered. The AWW-30, AWW-7, MWW, and PHWW flows were all based on this design storm. See the figure below for the hydrograph associated with this event. Based on the South WWTP Lift

Station operating at its capacity of 0.977 mgd, the flow equalization storage of 10.23 million gallons is needed to attenuate the flow and avoid SSO's. As illustrated below, the 50-day storm includes several rainfall events that occur along with high groundwater and runoff occurring. The blue shaded area on the hydrograph represents the total volume of wastewater diverted to the equalization basin. This combined equalization volume is 10.23 million gallons.

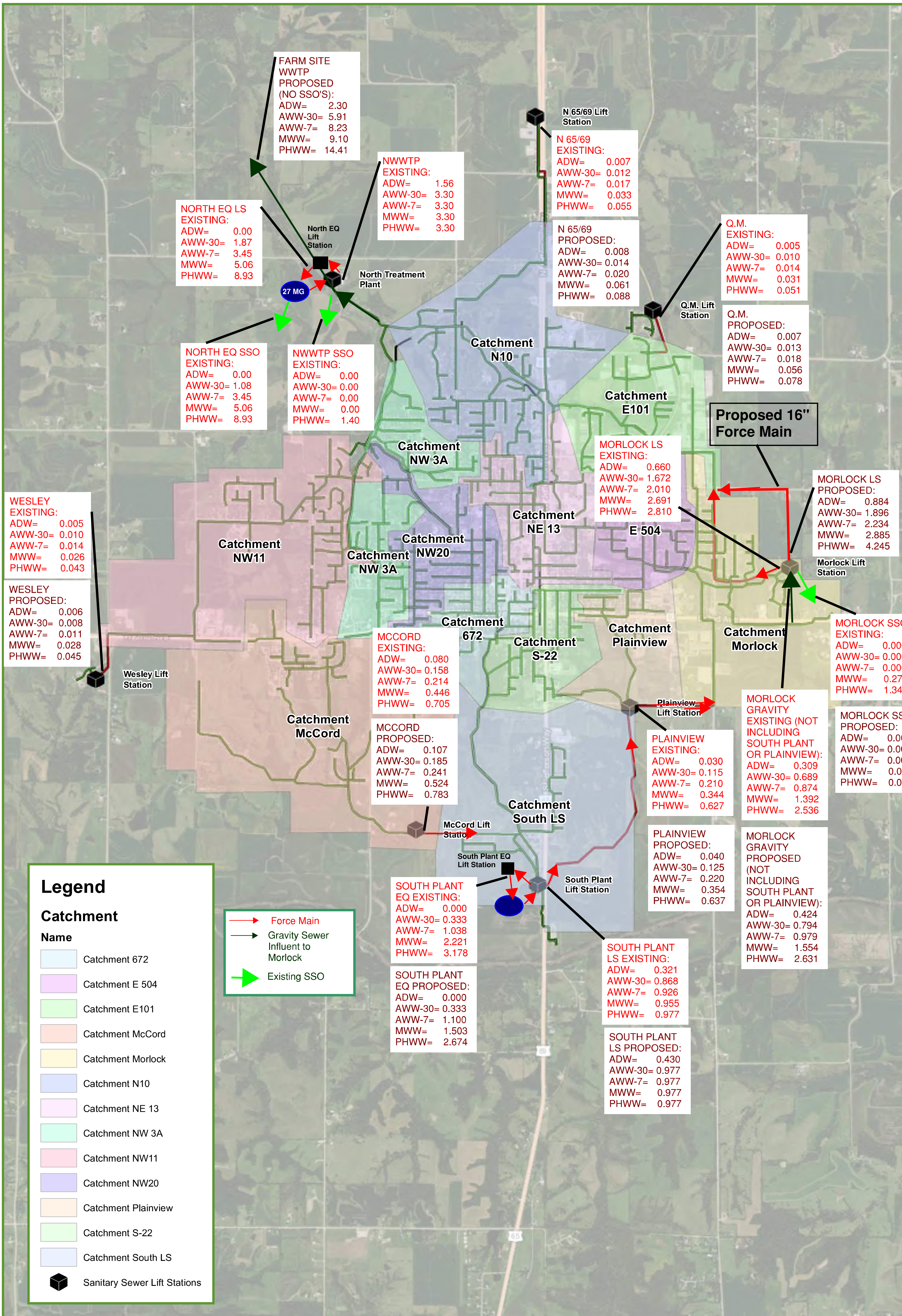


<u>Conclusion</u>

Based off of the above analysis, the 13.0 million gallon equalization basin has adequate capacity to attenuate the design flows at the South WWTP site. Additionally, the Lagoon Pump Station capacity of 5.76 mgd is adequate to pump flow into the equalization basin. The PHWW flow needed at the Lagoon Pump Station is 2.67 mgd.

The improvements at the Morlock Lift Station are a major factor in the adequacy of the South WWTP equalization capacity. Continued pumping from the South WWTP Lift Station during wet weather events will be a significant relief on the equalization basin and Lagoon Pump Station.

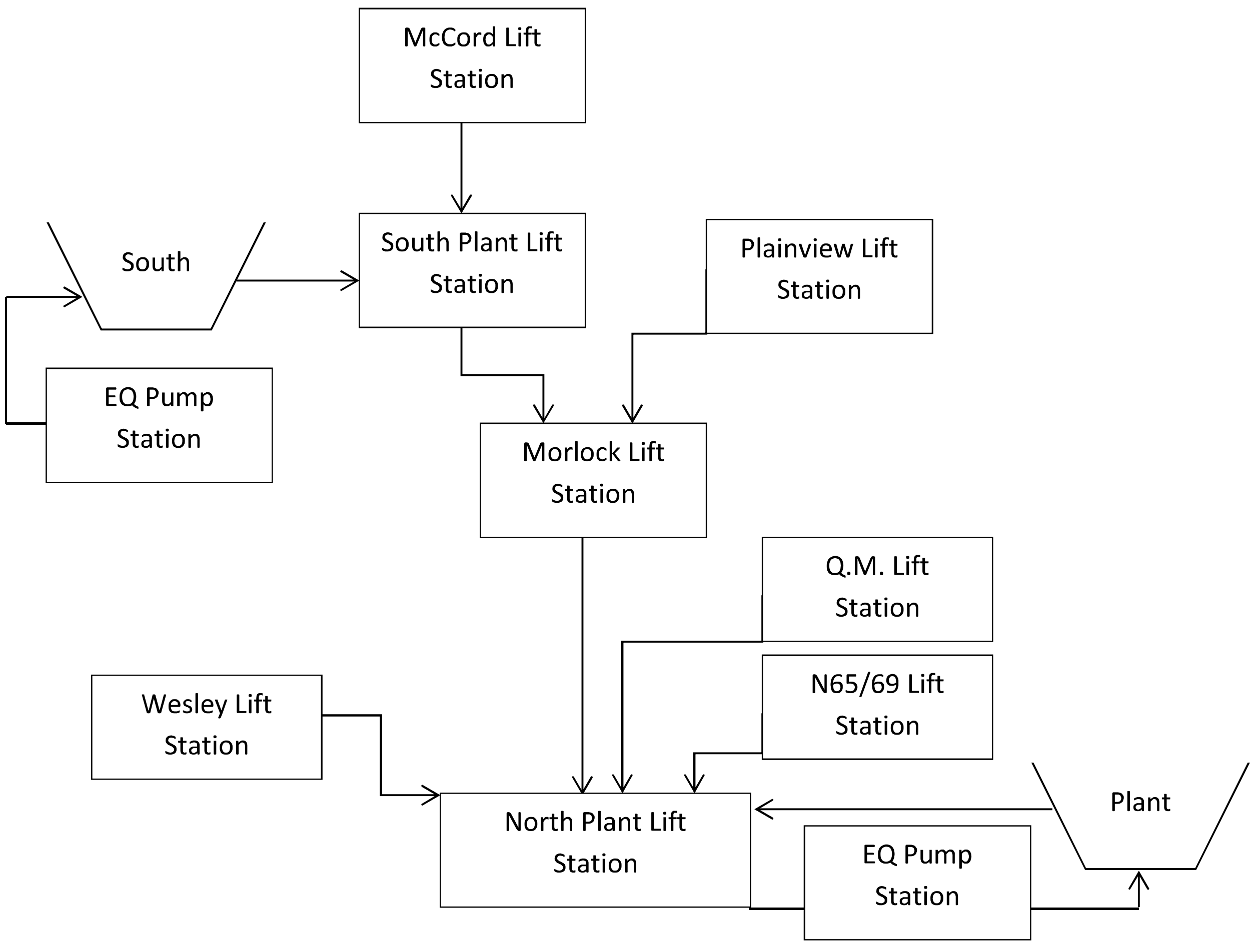
There is currently no need to evaluate increased pumping capacity and downstream conveyance at the South WWTP Lift Station.



HRGreen

Appendix A

Indianola, Iowa



Lift Station Flow Diagram

Appendix E - Waste Load Allocation – Middle River

Indianola, City of STP (North)

Proposed new outfall on the Middle River, ***variance required***

(Please do not microfiche this document.)

This Package Contains

WASTELOAD ALLOCATION CALCULATIONS & NOTES

Please Do Not Separate

ENVIRONMENTAL SERVICES DIVISION
WATER QUALITY BASED PERMIT LIMITS

SECTION VI: WATER QUALITY-BASED PERMIT LIMITS

Facility Name: Indianola, City of STP (North) Sewage File Number: 6-91-33-0-01

Parameters	Ave. Conc. (mg/l)	Max Conc. (mg/l)	Ave. Mass (lbs/d)	Max Mass (lbs/d)	Sampling Frequency
Outfall No. 001	**ADW = 2.30 mgd & AWW = 5.91 mgd**				
CBOD5	Secondary Treatment Levels Will Not Violate WQS				--
Total D.O.	Minimum Concentration (mg/l)				
January – December	5.0				--
Ammonia – Nitrogen*					
January	9.1	15.4	330.7	752.8	--
February	10.3	14.5	372.9	704.7	--
March	4.7	14.9	175.3	727.4	--
April	3.5	15.8	129.4	**697.8**	--
May	3.0	**15.0**	112.1	**480.3**	--
June	2.1	**10.3**	80.1	**334.9**	--
July	2.0	**8.1**	70.9	**263.6**	--
August	1.8	**8.2**	64.6	**260.3**	--
September	2.3	**9.6**	89.1	**308.4**	--
October	4.8	15.9	176.0	**481.3**	--
November	5.9	14.8	215.1	**716.7**	--
December	6.9	16.1	251.0	789.9	--
Bacteria	Geometric Mean (#org/100 ml)		March 15th – November 15th		--
*E. coli***	126				
Chloride	434	634	20,026	31,100	--
Sulfate	1,527	1,527	74,863	74,863	--
TRC***	0.223	0.319	11.0	15.7	--
pH	6.5 - 9.0 Standard Units				--

For the major facility acute WET testing, use 99.1% of effluent and 0.9% of dilution water

Stream Network/Classification of Receiving Stream: The Middle River (A1, B(WW-1) HH) to the Des Moines River (A1, B(WW-1) HH and Class C at the Ottumwa Water Works intake)

Date Done: Aug. 11, 2017

Annual critical low flow in the Middle River at the proposed outfall location ***(variance required)***
30Q10 flow 3.02 cfs, 7Q10 flow 1.82 cfs, 1Q10 flow 1.30 cfs, harmonic mean flow 20.5 cfs

Annual critical low flow in the Des Moines River at the Ottumwa Water Works intake
30Q10 flow 333 cfs, 7Q10 flow 267 cfs, 1Q10 flow 237 cfs, harmonic mean flow 1,792cfs

Excel Spreadsheet calculations [X] Qual II E Model [ ] Qual II E Modeling date[ ]

Performed by: Collin Klingbeil Approved by: Connie Dou

* **Bold** values are governed by CBOD5/DO modeling, while the others are based on ammonia nitrogen toxicity
** Des Moines River bacteria TMDL based limit
*** TRC limits are provided, but are not necessary unless chlorination is used.

Antidegradation Review Requirement

A tier II antidegradation review is required. See Section 2 for details.

Please note that the antidegradation review conducted in this WLA is based on the current information available. Antidegradation could also be triggered during the NPDES permitting process based on new information.

ENVIRONMENTAL SERVICES DIVISION
WATER QUALITY BASED PERMIT LIMITS

SECTION VI: WATER QUALITY-BASED PERMIT LIMITS (Cont'd)

Facility Name: Indianola, City of STP (North) Sewage File Number: 6-91-33-0-01

Parameters	Ave. Conc. (mg/l)	Max Conc. (mg/l)	Ave. Mass (lbs/d)	Max Mass (lbs/d)	Sampling Frequency
Outfall No. 001	**ADW = 2.30 mgd & AWW = 5.91 mgd**				
Toxics					
1,1,1-Trichloroethane	1.521E+01	2.664E+01	2.977E+02	1.306E+03	--
1,1,2-Trichloroethane	3.028E+00	3.028E+00	5.826E+01	5.826E+01	--
1,1-Dichloroethylene	1.733E+01	5.449E+01	5.461E+02	2.671E+03	--
1,2,4-Trichlorobenzene	5.323E+00	5.323E+00	1.042E+02	1.042E+02	--
1,2-Dichloroethane	9.029E-01	5.954E+01	2.846E+01	2.918E+03	--
1,2-Dichloropropane	3.661E-01	3.661E-01	1.154E+01	1.154E+01	--
2,3,7,8-TCDD (Dioxin)	1.245E-10	1.245E-10	3.923E-09	3.923E-09	--
2,4,5-TP (Silvex)	7.604E-01	7.604E-01	1.489E+01	1.489E+01	--
2,4-D	7.604E+00	7.604E+00	1.489E+02	1.489E+02	--
3,3-Dichlorobenzidine	6.833E-04	6.833E-04	2.154E-02	2.154E-02	--
4,4' DDT	1.128E-06	1.110E-03	5.174E-05	5.441E-02	--
Alachlor	1.521E-01	1.521E-01	2.977E+00	2.977E+00	--
Aldrin	1.220E-06	3.027E-03	3.846E-05	1.484E-01	--
Aluminum	9.813E-02	7.569E-01	4.502E+00	3.710E+01	--
Antimony	4.258E-01	1.110E+01	8.337E+00	5.441E+02	--
Arsenic (III)	9.084E-02	3.431E-01	1.748E+00	1.682E+01	--
Asbestos	5.323E-01	5.323E-01	1.042E+01	1.042E+01	--
Atrazine	2.281E-01	2.281E-01	4.466E+00	4.466E+00	--
Barium	7.604E+01	2.069E+02	1.489E+03	1.014E+04	--
Benzene	1.245E+00	1.665E+01	3.923E+01	8.162E+02	--
Benzo(a)Pyrene	4.393E-04	4.393E-04	1.385E-02	1.385E-02	--
Beryllium	3.042E-01	5.046E-01	5.955E+00	2.473E+01	--
Bromoform	5.369E-02	5.369E-02	1.692E+00	1.692E+00	--
Cadmium	3.417E+00	3.417E+00	1.077E+02	1.077E+02	--
Carbofuran	5.101E-04	4.355E-03	2.340E-02	2.135E-01	--
Carbon Tetrachloride	3.042E+00	3.042E+00	5.955E+01	5.955E+01	--
Chlordane	3.905E-02	2.175E+01	1.231E+00	1.066E+03	--
Chloride	4.850E-06	2.422E-03	2.225E-04	1.187E-01	--
Chlorobenzene	4.34E+02	6.34E+02	2.0026E+04	3.1100E+04	--
Chlorodibromomethane	1.805E+00	1.625E+01	8.279E+01	7.964E+02	--
Chloroform	3.172E-01	3.172E-01	9.999E+00	9.999E+00	--
Chloropyrifos	1.147E+01	1.147E+01	3.615E+02	3.615E+02	--
Chromium (VI)	4.624E-05	8.376E-05	2.121E-03	4.106E-03	--
cis-1,2-Dichloroethylene	1.241E-02	1.615E-02	5.692E-01	7.914E-01	--
Copper	5.323E+00	5.323E+00	1.042E+02	1.042E+02	--
Cyanide	1.849E-02	2.711E-02	8.624E-01	1.330E+00	--
Dalapon	5.865E-03	2.220E-02	2.691E-01	1.088E+00	--
Di(2-ethylhexyl)adipate	1.521E+01	1.521E+01	2.977E+02	2.977E+02	--
Bis(2-ethylhexyl)phthalate	3.042E+01	3.042E+01	5.955E+02	5.955E+02	--
Dibromochloropropane	1.521E-02	1.521E-02	2.977E-01	2.977E-01	--
Dichlorobromomethane	4.149E-01	4.149E-01	1.308E+01	1.308E+01	--

ENVIRONMENTAL SERVICES DIVISION
WATER QUALITY BASED PERMIT LIMITS

SECTION VI: WATER QUALITY-BASED PERMIT LIMITS (Cont'd)

Facility Name: Indianola, City of STP (North) Sewage File Number: 6-91-33-0-01

Parameters	Ave. Conc. (mg/l)	Max Conc. (mg/l)	Ave. Mass (lbs/d)	Max Mass (lbs/d)	Sampling Frequency
Outfall No. 001	**ADW = 2.30 mgd & AWW = 5.91 mgd**				
Toxics					
Dichloromethane	3.802E-01	3.802E-01	7.444E+00	7.444E+00	--
Dieldrin	1.318E-06	2.422E-04	4.154E-05	1.187E-02	--
Dinoseb	5.323E-01	5.323E-01	1.042E+01	1.042E+01	--
Diquat	1.521E+00	1.521E+00	2.977E+01	2.977E+01	--
Endosulfan	6.316E-05	2.220E-04	2.898E-03	1.088E-02	--
Endothall	7.604E+00	7.604E+00	1.489E+02	1.489E+02	--
Endrin	4.060E-05	8.679E-05	1.863E-03	4.254E-03	--
Ethylbenzene	2.369E+00	2.286E+01	1.087E+02	1.120E+03	--
Ethylene dibromide	3.802E-03	3.802E-03	7.444E-02	7.444E-02	--
Fluoride	8.151E+00	8.151E+00	3.995E+02	3.995E+02	--
gamma-Hexachlorocyclohexane (Lindane)	9.587E-04	9.587E-04	4.699E-02	4.699E-02	--
Glyphosate	5.323E+01	5.323E+01	1.042E+03	1.042E+03	--
Heptachlor	1.928E-06	5.247E-04	6.077E-05	2.572E-02	--
Heptachlor epoxide	9.517E-07	5.247E-04	3.000E-05	2.572E-02	--
Hexachlorobenzene	7.077E-06	7.077E-06	2.231E-04	2.231E-04	--
Hexachlorocyclopentadiene	1.241E+00	1.241E+00	5.692E+01	5.692E+01	--
Iron	1.009E+00	1.009E+00	4.946E+01	4.946E+01	--
Lead	8.677E-03	1.992E-01	3.980E-01	9.765E+00	--
Mercury (II)	1.692E-04	1.655E-03	7.761E-03	8.112E-02	--
Methoxychlor	7.604E+00	7.604E+00	1.489E+02	1.489E+02	--
Nickel	1.058E-01	8.510E-01	4.852E+00	4.172E+01	--
Nitrate as N	3.229E+02	3.229E+02	1.489E+04	1.583E+04	--
Nitrate+Nitrite as N	3.229E+02	3.229E+02	1.489E+04	1.583E+04	--
Nitrite as N	7.604E+01	7.604E+01	1.489E+03	1.489E+03	--
o-Dichlorobenzene	4.562E+01	4.562E+01	8.932E+02	8.932E+02	--
Oxamyl (Vydate)	1.521E+01	1.521E+01	2.977E+02	2.977E+02	--
para-Dichlorobenzene	2.143E-01	2.018E+00	9.831E+00	9.893E+01	--
Parathion	1.466E-05	6.559E-05	6.727E-04	3.215E-03	--
Pentachlorophenol (PCP)	2.062E-02	2.405E-02	9.460E-01	1.179E+00	--
Phenols	5.639E-02	2.523E+00	2.587E+00	1.237E+02	--
Picloram	3.802E+01	3.802E+01	7.444E+02	7.444E+02	--
Polychlorinated Biphenyls (PCBs)	1.562E-06	2.018E-03	4.923E-05	9.893E-02	--
Polynuclear Aromatic Hydrocarbons (PAHs)	3.384E-05	3.027E-02	1.552E-03	1.484E+00	--
Selenium	5.639E-03	1.948E-02	2.587E-01	9.547E-01	--
Silver	3.835E-03	3.835E-03	1.880E-01	1.880E-01	--
Simazine	3.042E-01	3.042E-01	5.955E+00	5.955E+00	--
Styrene	7.604E+00	7.604E+00	1.489E+02	1.489E+02	--
Sulfate	1.527E+03	1.527E+03	7.4863E+04	7.4863E+04	--

ENVIRONMENTAL SERVICES DIVISION WATER QUALITY BASED PERMIT LIMITS					
SECTION VI: WATER QUALITY-BASED PERMIT LIMITS (Cont'd) Facility Name: Indianola, City of STP (North)					Sewage File Number: 6-91-33-0-01
Parameters	Ave. Conc. (mg/l)	Max Conc. (mg/l)	Ave. Mass (lbs/d)	Max Mass (lbs/d)	Sampling Frequency
Outfall No. 001	**ADW = 2.30 mgd & AWW = 5.91 mgd**				
Toxics					
Tetrachloroethlyene	8.053E-02	8.053E-02	2.538E+00	2.538E+00	--
Thallium	5.301E-04	6.035E-01	2.432E-02	2.958E+01	--
Toluene	5.639E-02	2.523E+00	2.587E+00	1.237E+02	--
Total Residual Chlorine (TRC)	2.23E-01	3.19E-01	1.10E+01	1.57E+01	--
Toxaphene	2.256E-06	7.367E-04	1.035E-04	3.611E-02	--
trans-1,2-Dichloroethylene	1.579E-01	1.579E-01	7.244E+00	7.244E+00	--
Trichloroethylene (TCE)	9.023E-02	4.037E+00	4.139E+00	1.979E+02	--
Trihalomethanes (total)	6.083E+00	6.083E+00	1.191E+02	1.191E+02	--
Vinyl Chloride	5.857E-02	5.857E-02	1.846E+00	1.846E+00	--
Xylenes (Total)	7.604E+02	7.604E+02	1.489E+04	1.489E+04	--
Zinc	2.175E-01	2.175E-01	1.066E+01	1.066E+01	--

WLA/permit limits for the City of Indianola's Mechanical Plant

These wasteload allocations and water quality based permit limitations are for the City of Indianola's wastewater discharge. The wasteload allocations/permit limits are based on the Water Quality Standards (IAC 567.61) and 'Supporting Document for Iowa Water Quality Management Plans,' Chapter IV, November 11, 2009. The chloride allocation/permit limits are based on the criteria that became effective on November 11, 2009.

The water quality based limits in this WLA are calculated to meet the surface water quality criteria to protect downstream uses. There could be technology based limits applicable to this facility that are more stringent than the water quality based limits shown in this WLA. The technology based limits could be derived from either federal guidelines based on different industrial categories or permit writer's judgment.

1. BACKGROUND: The City of Indianola is proposing to discharge treated domestic wastewater from a new mechanical (activated sludge) wastewater treatment facility. This wasteload allocation is for a proposed outfall into the Middle River (at 41° 25' 14" N, 93° 36' 26" W).

Route of Flow and Use Designations:
The Middle River is an A1, B(WW-1) HH designated use waterbody from the proposed outfall to the mouth. Downstream of the mouth of the Middle River, the Des Moines River is an A1, B(WW-1) HH designated use waterbody and also has a Class C use at the Ottumwa Municipal Water Works intake. The designations have been adopted in Iowa's state rule described in the rule referenced document of Surface Water Classification effective on June 17, 2015. Based on the pollutants of concern, the use designations of stream segments further downstream will not impact the resulting limits for this facility.

Critical Low Flow Determination:
The annual critical low flows in the Middle River at the proposed discharge point are estimated based on the 2012 USGS Low-Flow Study "Methods for Estimating Selected Low-Flow Frequency Statistics and Harmonic Mean Flows for Streams in Iowa" (hereafter "USGS 2012 Low-Flow Study" as revised in 2013) in lieu of the March 1979 methodology. There is a USGS gage about 1 mile below the proposed outfall of this facility on the Middle River (05486490), thus the weighted drainage area ratio (WDAR) method is used.

Please note that a variance is required to use the flows and corresponding limits shown in this report.

Table 1a: Annual Critical Low Flows in the Middle River

Location	Drainage Area (mi^2)	Harmonic Mean (cfs)	Annual critical low flows (cfs)		
			1Q10	7Q10	30Q10
The Middle River at the proposed outfall location	486	20.5$^{@}$	1.30$^{@}$	1.82$^{@}$	3.02$^{@}$

$^{@}$: Estimated based on the 2012 USGS Low-Flow Study

Downstream of the mouth of the Middle River, there is a Class C use of the Des Moines River at the Ottumwa Water Works intake. The annual critical low flows in the Des Moines River at the Ottumwa Water Works intake are determined so that the limits for the protection of the Class C use of the Des Moines River can be calculated. The annual critical low flows are estimated based on the drainage area ratio method and flow statistics obtained at USGS gage station 05488500, located on the Des Moines River near Tracy, Iowa.

Table 1b: Annual Critical Low Flows in the Des Moines River

Location	Drainage Area (mi^2)	Harmonic Mean (cfs)	Annual critical low flows (cfs)		
			1Q10	7Q10	30Q10
USGS Gage (05488500)	12,479	$1{,}670^{\$}$	$221^{\$}$	$249^{\$}$	$310^{\$}$
The Des Moines River at the Ottumwa Water Works intake	13,393	$1{,}792^{@}$	$237^{@}$	$267^{@}$	$333^{@}$

$^{\$}$: USGS gage station statistic data
$^{@}$: Estimated based on the drainage area ratio method

2. ANTIDEGRADATION REVIEW REQUIREMENT:

According to the Iowa Antidegradation Implementation Procedure, effective February 17, 2010 (IAC 567-61.2(2).e), all new or expanded regulated activities (with limited exceptions, such as unsewered communities) are subject to antidegradation review requirements.

Table 2: Antidegradation Review Analysis

Item #	Factor or Scenario	Antidegradation Determination	Analysis/Comments
1	Design Capacity Increase	Yes ☒, No ☐, or Not Applicable ☐	
2	Significant Industrial Users (SIU) Contributing New Pollutant of Concern (POC)	Yes ☐, No ☒, or Not Applicable ☐	As indicated in the request form
3	New Process Contributing New Pollutant of Concern (POC)	Yes ☐, No ☒, or Not Applicable ☐	As indicated in the request form
4	Less Stringent Water Quality Based Limits?	Yes ☒, No ☐, or Not Applicable ☐	1: Current limits sheet attached
5	Outfall Location Change	Yes ☒, No ☐, or Not Applicable ☐	1: Move outfall to the Middle River
Conclusion and discussion: Due to Items 1, 4, and 5, a tier II antidegradation review is required. Please note that the antidegradation review conducted in this WLA is based on the current information available. Antidegradation could also be triggered during the NPDES permitting process based on new information.			

3. TOTAL MAXIMUM DAILY LOAD (TMDL) LIMITATIONS:

The following stream segments in the discharge route are on the 2014 impaired waters list:

- The Middle River for aquatic life – biological (IBI) and primary contact – indicator bacteria
- The Des Moines River for primary contact – indicator bacteria, aquatic life – biological (other), and aquatic life – biological (fish kill: unknown toxicity)

In 2009, a TMDL was completed for five segments of the Des Moines River in Polk, Warren, and Marion Counties for pathogen indicators (*E. coli*). In the TMDL, the Indianola wastewater treatment facility was assigned *E. coli* wasteload allocations, as discussed in the *E. coli* section below. TMDLs for the other impairments in the route of flow downstream from the proposed outfall of this facility have not been completed at this time.

Please note that the results presented in this report are wasteload allocations based on meeting the State's current water quality standards in the receiving waterbody. Additional and/or more stringent effluent limits may be applicable to this discharge based on approved TMDLs for impaired waterbodies, which may provide watershed based wasteload allocations. Information on impaired streams in Iowa and approved TMDLs can be found at the following website: http://www.iowadnr.gov/Environmental-Protection/Water-Quality/Watershed-Improvement/Impaired-Waters

4. CALCULATIONS: The wasteload allocations / permit limits for this outfall are calculated based on the facility's proposed Average Dry Weather (ADW) design flow of 2.30 mgd and its proposed Average Wet Weather (AWW) design flow of 5.91 mgd.

Please note that only wasteload allocations/permit limits (water quality based effluent limits) calculated using DNR approved design flows can be applied in NPDES permits. Water quality based effluent limits calculated using proposed flows that have not been approved by the DNR for permitting and compliance may be used for informational purposes only.

The water quality based permit concentration limits are derived using the allowed stream flow and the ADW design flow, while loading limits are derived using the allowed stream flow and the AWW design flow.

Toxics: The toxics wasteload allocations will consider the procedures included in the 2000 revised WQS and the 2007 chemical criteria. TRC limits are provided, but are not necessary unless chlorination is used.

To protect the aquatic life use:
Important to the toxics is the use of the 1Q10 stream flow in association with the acute wasteload allocation calculations. The chronic WLA will continue to use the 7Q10 stream flow in its calculations. In this case, 25% of the 7Q10 flow and 2.5% of the 1Q10 flow in the Middle River at the proposed outfall are used as the Mixing Zone (MZ) and Zone of Initial Dilution (ZID), respectively.

To protect the Class HH use:
For pollutants that are non-carcinogenic and have criteria for human health protection, the criteria apply at the end of the MZ, which in this case is 25% of the 7Q10 flow in the Middle River at the proposed outfall location.

For pollutants that are carcinogenic and have criteria for human health protection, the criteria apply at the end of the MZ, which in this case is 25% of the harmonic mean flow in the Middle River at the proposed outfall location.

To protect the downstream Class C use:
The Middle River enters the Des Moines River over 30 miles upstream of the Class C use of the Des Moines River; therefore, the effluent is expected to be completely mixed with the Des Moines River flows at that point.

For pollutants that are non-carcinogenic and have criteria for maximum contaminant level (MCL), the criteria apply at the end of the MZ, which in this case is 100% of the 7Q10 flow in the Des Moines River at the Ottumwa Water Works intake.

For pollutants that are carcinogenic and have criteria for maximum contaminant level (MCL), the criteria apply at the end of the MZ, which in this case is 100% of the harmonic mean flow in the Des Moines River at the Ottumwa Water Works intake.

Final limits:
The maximum limits are those calculated for the protection of the aquatic life use and the average limits are the most stringent between those for the protection of the aquatic life use, those for the protection of the Class HH use, and those for the protection of the Class C use.

Please note that the TRC limits are based on a sampling frequency of 5/week based on a population equivalent (PE) of 28,186. The limits for the other toxics are based on a sampling frequency of 1/week.

Ammonia Nitrogen: Standard stream background temperatures, pH, and concentrations of NH3-N are mixed with the discharge from the facility's effluent pH and temperature values to calculate the applicable instream WQS criteria for the protection of the Middle River.

Based on the ratio of the stream flow to the discharging flow, 5% of the 1Q10 and 100% of the 30Q10 flow are used as the ZID and the MZ, respectively. The Middle River is a B(WW-1) stream; therefore, early life protection will begin in March and run through September.

The monthly background temperatures, pH, and NH3-N concentrations shown in Table 3 are used for the wasteload allocation/permit limits calculations based on the Year 2000 ammonia nitrogen criteria. Table 4 shows the statewide monthly effluent pH and temperature values for mechanical facilities. Table 5a shows the calculated toxicity based ammonia nitrogen wasteload allocations for this facility. Additionally, Table 5b shows the final WLAs for ammonia nitrogen with reductions from the CBOD5/DO modeling.

Table 3: Background pH, Temperature, and NH3-N Concentrations For Use with Year 2000 Ammonia Nitrogen Criteria

Months	pH	Temperature (°C)	NH_3-N (mg/l)
January	7.8	0.6	0.5
February	7.7	1.2	0.5
March	7.9	4.3	0.5
April	8.1	11.7	0.5
May	8.1	16.6	0.5
June	8.1	21.4	0.5
July	8.1	24.8	0.0
August	8.2	23.8	0.0
September	8.0	22.2	0.5
October	8.0	12.3	0.5
November	8.1	6.0	0.5
December	8.0	1.6	0.5

Table 4: Standard Effluent pH & Temperature Values for Mechanical Facilities

Months	pH	Temperature (°C)
January	7.67	12.4
February	7.71	11.3
March	7.69	13.1
April	7.65	16.2
May	7.67	19.3
June	7.7	22.1
July	7.58	24.1
August	7.63	24.4
September	7.62	22.8
October	7.65	20.2
November	7.69	17.1
December	7.64	14.1

Table 5a: Toxicity Based Wasteload Allocations for Ammonia Nitrogen for the Protection of Aquatic Life

Months	ADW-Based*		AWW-Based**	
	Acute (mg/l)	Chronic (mg/l)	Acute (mg/l)	Chronic (mg/l)
January	15.4	9.1	15.3	6.7
February	14.5	10.3	14.3	7.6
March	14.9	4.7	14.8	3.6
April	15.8	3.5	15.8	2.6
May	15.3	3.0	15.2	2.3
June	14.6	2.1	14.5	1.6
July	17.8	2.0	17.6	1.4
August	16.4	1.8	16.3	1.3
September	16.7	2.3	16.6	1.8
October	15.9	4.8	15.8	3.6
November	14.8	5.9	14.7	4.4
December	16.1	6.9	16.0	5.1

*: bases for concentration limits; **: bases for mass loading limits

Table 5b: Final Wasteload Allocations for Ammonia Nitrogen for the Protection of Aquatic Life after CBOD5/DO Modeling*

Months	ADW-Based**		AWW-Based***	
	Acute (mg/l)	Chronic (mg/l)	Acute (mg/l)	Chronic (mg/l)
January	15.4	9.1	15.3	6.7
February	14.5	10.3	14.3	7.6
March	14.9	4.7	14.8	3.6
April	15.8	3.5	**14.2**	2.6
May	**15.0**	3.0	**9.7**	2.3
June	**10.3**	2.1	**6.8**	1.6
July	**8.1**	2.0	**5.3**	1.4
August	**8.2**	1.8	**5.3**	1.3
September	**9.6**	2.3	**6.3**	1.8
October	15.9	4.8	**9.8**	3.6
November	14.8	5.9	**14.5**	4.4
December	16.1	6.9	16.0	5.1

*: **Bold** values are governed by CBOD5/DO modeling, while the other values are based on ammonia nitrogen toxicity protection for aquatic life
**: bases for concentration limits
***: bases for mass loading limits

CBOD5/Total Dissolved Oxygen: Streeter-Phelps DO Sag Model is used to simulate the decay of CBOD and dispersion of total Dissolved Oxygen (DO) in the receiving water downstream from the outfall. The criterion is that the discharge cannot cause the DO level in the receiving stream (warm waters) to be below 5.0 mg/l.

The parameter values used in the modeling are listed below:

Background: The temperature and ammonia nitrogen levels are shown in Table 3. The ultimate CBOD and DO levels are assumed to be 8.0 mg/l and 6.0 mg/l, respectively.

Effluent: The temperatures are shown in Table 4. The CBOD5 level used in the modeling is 40 mg/l, which is the technology based maximum limit for standard secondary treatment. The ammonia nitrogen values used in the modeling are the calculated acute wasteload shown in Table 5a. Both ADW and AWW flows and the ammonia nitrogen allocations associated with them are used in the modeling.

Receiving stream parameters: There is an average water channel slope of approximately 0.00048 (the water channel elevation changes from 784 ft to 760 ft over a distance of approximately 50,200 ft), estimated based on the GIS LiDAR 2-ft contour coverage.

USGS gage 05486490, located on the Middle River near Indianola, IA, had field measurement data, such as stream flow, cross section area, stream width and velocity. The stream depth is not reported, however, can be derived using the following equation:

Depth = Cross Section Area / Width

Regression equations of Ln(Velocity) vs. Ln (Flow) and Ln(Depth) vs. Ln (Flow) were established with acceptable R-squared values.

Ln (Velocity) = 0.2925*Ln(Flow) – 1.1752 R-squared = 0.7763
Ln (Depth) = 0.457*Ln(Flow) – 1.8957 R-squared = 0.8223

Width = Flow / Velocity / Depth

The gage station is about 1 mile downstream of the proposed discharge location. Therefore in the absence of other data that could be used to estimate stream width, depth and velocity, it is assumed that the above regression equations are valid at the outfall.

Table 6: Stream Width, Depth and Velocity

Flow condition	Flow (cfs)	Width (ft)	Depth (ft)	Velocity (fps)
7Q10 + ADW	5.378	32.9	0.32	0.51
7Q10 + AWW	10.963	39.3	0.45	0.62

Reaeration: UAA data noted that the Middle River had steep banks and described the Middle River downstream of the proposed outfall as a run. Therefore, the USGS channel-control model (Melching and Flores 1999) is used in the modeling.

Discussion and Conclusion: The modeling results show that the effluent, which could have an allowed maximum effluent CBOD5 level of 40 mg/l (technology based limits for secondary treatment) and a minimum DO level of 5.0 mg/l will not cause the DO level in the receiving stream below 5.0 mg/l at any time; however, some of the calculated water quality based ammonia nitrogen wasteload allocations, as shown in Table 5a, need to be reduced. The final ammonia nitrogen limits are shown in Table 5b and on Page 1 of this report.

*E. coli***:** The proposed discharge is into a Class (A1) water body. The water quality standard for *E. coli* in a Class (A1) water body is a Geometric Mean of 126 org./100 ml and a Sample Maximum of 235 org./100 ml from March 15th through November 15th. The criteria apply at "end-of-pipe".

A 2009 TMDL for five segments of the Des Moines River for *E. coli* assigned the Indianola wastewater treatment facility a geometric mean of 126 org./100 ml and a sample maximum of 235 org./100 ml from March 15th through November 15th. The criteria apply at "end-of-pipe". These values are identical to those for the protection of a Class (A1) water body; therefore, they govern the final limits.

However, 567 IAC 62.8(2) states that "the daily sample maximum criteria for *E. coli* set forth in Part E of the 'Supporting Document for Iowa Water Quality Management Plans' shall not be used as an end-of-pipe permit limitation." Therefore, only the geometric mean limit of 126 org./100 mL applies.

Chloride and Sulfate: The new chloride and sulfate criteria became effective on Nov. 11, 2009. The default hardness for background and effluent has been changed from 100 mg/l to 200 mg/l, effective on Nov. 11, 2009.

Chloride criteria are functions of hardness and sulfate concentration, shown as follows:

$$\text{Acute criteria} = 287.8*(\text{Hardness})^{0.205797}*(\text{Sulfate})^{-0.07452}$$
$$\text{Chronic criteria} = 177.87*(\text{Hardness})^{0.205797}*(\text{Sulfate})^{-0.07452}$$

The criteria apply to all Class B waters.

Sulfate criteria, shown in Table 7, are functions of hardness and chloride concentration.

Table 7: Sulfate Criteria

Hardness (mg/l as CaCO3)	Sulfate Criteria (mg/l)		
	Chloride < 5 mg/l	5 mg/l <= Chloride < 25 mg/l	25 mg/l <= Chloride < 500 mg/l
< 100	500	500	500
100<=H<=500	500	(-57.478+5.79*H+54.163*Cl)*0.65	(1276.7+5.508*H-1.457*Cl)*0.65
H> 500	500	2,000	2,000

The criteria defined in Table 7 serve as both acute and chronic criteria and apply to all Class B waters.

The acute criteria apply at the end of the ZID, and the chronic criteria apply at the end of the MZ. In this case, 25% of the 7Q10 flow and 2.5% of the 1Q10 flow in the Middle River are used as the MZ and the ZID, respectively.

The default chloride concentration for both background water and effluent is 34 mg/l, while the default sulfate concentration for both background water and effluent is 63 mg/l. The limits for chloride and sulfate are calculated based on an assumed sampling frequency of 1/week.

Iron: The current iron criteria are defined in the 2005 issue paper entitled "Iron Criteria and Implementation for Iowa's Surface Waters (December 5, 2005)". An iron criterion of 1 mg/l applies at the end of the ZID for designated streams. In this case, the ZID is 2.5% of the 1Q10 at the discharging point.

pH: Iowa Water Quality Standards (IAC 567.61.3.(3).a.(2) and IAC 567.61.3.(3).b.(2)) require that pH in Class A or Class B waters "Shall not be less than 6.5 nor greater than 9.0". The criteria apply at the end of the ZID. In this case, the ZID is 2.5% of the 1Q10 at the discharging point.

TDS: Effective Nov. 11, 2009, the site-specific TDS approach is no longer applicable; instead the new chloride and sulfate criteria became applicable. However, the TDS level should be controlled to a level such that the narrative criteria stated in IAC 567.61.3.(2) be fulfilled.

Major Facility Acute WET testing Ratio: Use 99.1% of effluent and 0.9% of dilution water for the testing. The ratio is calculated using ADW design flow and 2.5% of 1Q10 as the ZID.

5. PERMIT LIMITATIONS: - *Based on the Year 2006 Water Quality Standards & 2002 Permit Derivation Procedure.*

The acute and chronic WLAs are used as the values for input into the current permit derivation procedure. Under the 2002 permit derivation procedure, only for toxic parameters is the monitoring frequency considered in the calculation of final limits. The water quality based limits are shown on Pages 1 – 4 of this report.

Appendix F - Indianola Hydraulic Model Summary

SANITARY SEWER MODEL REPORT

FOR

CITY OF INDIANOLA, IOWA

JUNE 2014

INDIANOLA, IOWA

40130054

Prepared by: Matthew J. Wildman

CERTIFICATION

SANITARY SEWER MODEL REPORT

INDIANOLA, IOWA

JUNE 2014

LICENSED PROFESSIONAL ENGINEER
MATTHEW J.
WILDMAN
17910
IOWA

I hereby certify that this engineering document was prepared by me or under my direct personal supervision and that I am a duly licensed Professional Engineer under the laws of the State of Iowa.

_______________________________ Date:__________

MATTHEW J. WILDMAN, P.E.

License No. **17910**

My renewal date is **December 31, 2015**

Pages or sheets covered by this seal:

Entire Report

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I. EXECUTIVE SUMMARY

Purpose

The City of Indianola has a known issue of inflow and infiltration in the existing sanitary sewer system. Due to the limited amount of data available on the existing system and the uncertainty regarding the accuracy of the existing data, the primary focus of this work was to examine the existing sanitary sewer system and establish a hydraulic model that can be utilized as a planning tool for future growth and design as more data is collected and input. The hydraulic model was developed to delineate problem areas by evaluating both the dry and wet weather conditions for the existing system. The model was then used to evaluate the adequacy of collection and conveyance systems for existing and future flows.

Method

The first step in the development of the model was to collect physical attributes of the manholes and pipes. This included GPS data as well as a brief condition assessment. Hourly and 15-minute incremental flow data was provided by the City for time periods after September 2013. Daily flow data was also collected from the City's monthly operating reports as needed. The average baseline flow, or the portion of flow caused solely by sanitary use, was determined to be approximately 1.2 MGD. The diurnal pattern associated with this baseline flow was utilized as a template for sanitary loadings to individual utility structures throughout the system.

The wet weather flow was modeled using a storm event occurring on April 13, 2014. The rainfall event was assumed as 2.65 inches based on nearby recorded rainfall information obtained from the National Climatic Data Center (NCDC). During wet weather, the initial response seen at the plant is typically due to inflow into the system. This is identifiable by the quick increase of the flowrate. The flowrate is typically increased in proportion with the amount of rain that falls. Once the rain ceases, the flow due to inflow will decrease quickly.

Findings

Following calibration, four rainfall events were simulated within the model including the Base Flow Condition. The flow data generated by the model for the various scenarios can be found in Table 1 below.

Table 1: Summary of Model Output for Various Storm Events

Event	Rainfall (in)	Maximum Average Daily Flow (MGD)	Peak Daily Flow (MGD)
Dry Weather (base flow)	0.0	1.20	1.55
10-yr, 24-hr Storm	4.54	8.11	12.45
25-yr, 24-hr Storm	5.59	9.36	14.51
100-yr, 24-hr Storm	7.5	11.51	18.21

The model indicates that the existing piping is sized correctly to handle the dry weather base line flows. Under these dry weather conditions the model indicates that no pipes will surcharge and that no backups will occur.

The system model indicates that during high rain events sewers in many of the catchment areas will start to surcharge and cause backups. These issues can generally be solved by either increasing the size of the collection system or decreasing the demand on the system by reducing I&I. Typically, eliminating inflow from the system is a more cost effective alternative then increasing the size of piping and utility structures and is the first choice of action. Based on the model results, a relatively small reduction in inflow would allow the system to accommodate a 100-year, 24-hour storm event without producing backups or overflowing any manholes in the collection system.

Recommendations

Further calibration of the model is recommended in the future to ensure accurate model results. This can easily be completed with additional flow data including substantial rainfall events. Also, the current model uses rainfall data from monitoring stations in nearby towns. To increase accuracy of the model, rainfall monitors should be installed in multiple locations around the City. This ensures the accuracy of rainfall data which is crucial to correct model calibration. To fully calibrate the model, flow monitoring should be done throughout the system to pinpoint areas contributing excessive amounts of I&I. The current model distributes I&I relatively evenly over each catchment area due to lack of known I&I locations. In reality, certain sections of piping likely contribute significantly more I&I compared to others. These sections will likely result in surcharging manholes and backups not identified within this report.

The most cost effective way to reduce inflow is smoke testing and home inspections. This will allow the City to identify and eliminate storm connections from directly connecting to the sanitary system. The next step after inflow has been addressed will be to determine the locations of greatest infiltration. This can either be completed using flow monitoring or televising. Once problem lines are determined, the pipes could be lined or replaced.

II. INTRODUCTION

The City of Indianola has a known issue of inflow and infiltration in the existing sanitary sewer system. HR Green was recently contracted by the City to survey existing utilities and develop a conveyance system model to pinpoint areas of concern within the collection system. Due to the limited amount of data available on the existing system and uncertainty regarding the accuracy of the existing data, the primary focus of this work was to examine the existing sanitary sewer system and establish a hydraulic model that can be utilized as a planning tool for future growth and design as more data is collected and input.

The hydraulic model was developed to delineate problem areas by evaluating both the dry and wet weather conditions for the existing system. The model was then used to evaluate the adequacy of collection and conveyance systems for existing and future flows. By evaluating the existing flows and system responses to storm events, the model will provide assistance in the prioritization of maintenance on the existing sanitary sewer system. The model can also be used as a tool when investigating options for updating the wastewater treatment plant to meet new and upcoming regulations or to assist the City in determining capacity within the sanitary sewer system for future development. By narrowing down the most apparent problem areas for inflow and infiltration and providing the proper maintenance, the City could reduce the cost of construction for the additional wastewater treatment infrastructure by reducing the required overall size.

The purpose of this report is to summarize assumptions made, as well as detail and summarize the findings of the modeling process. The goals and objectives are detailed below:

1. Evaluate the availability of adequate collection and conveyance of wastewater for existing and future flows during both dry and wet weather conditions.
2. Assist in supporting the level of service expected by customers to avoid system surcharges that may lead to basement or service back-ups and sanitary sewer overflow events.
3. Control wet weather effects on operations of system facilities such as the treatment plant.
4. Develop a hydraulic model that serves as a key tool for assisting in prioritizing maintenance for sanitary sewer system assets.
5. Use this hydraulic model for assisting in management of the sanitary sewer collection system, for resolving issues with the current system, and planning for future development and economic growth.

III. BACKGROUND AND SYSTEM INFORMATION

The City of Indianola's sanitary sewer system consists of approximately 83 miles of sanitary sewer, 1560 manholes, 8 lift stations, 2 equalization basins and a wastewater treatment plant. Sanitary sewer sizes range from 6" to 36" and materials commonly range from Vitrified Clay (VCP), Polyvinyl Chloride (PVC) to Truss piping. Flows from all users are routed through the various lift stations and a mixture of gravity and forcemain piping to the wastewater treatment plant located northwest of the city.

IV. DATA COLLECTION

Initially, GPS data was collected for all manholes and piping in town. This data included a condition assessment of all utility structures as displayed in Figure 1 below. The system's physical attributes were then imported into SewerGEMS V8i software. The software automatically generated sewer pipes and manholes within the model. Under various circumstances, manhole and pipe characteristics were unable to be collected, located or measured in the field. In these scenarios, unknown manhole and pipe characteristics were assigned using known upstream and downstream utility data.

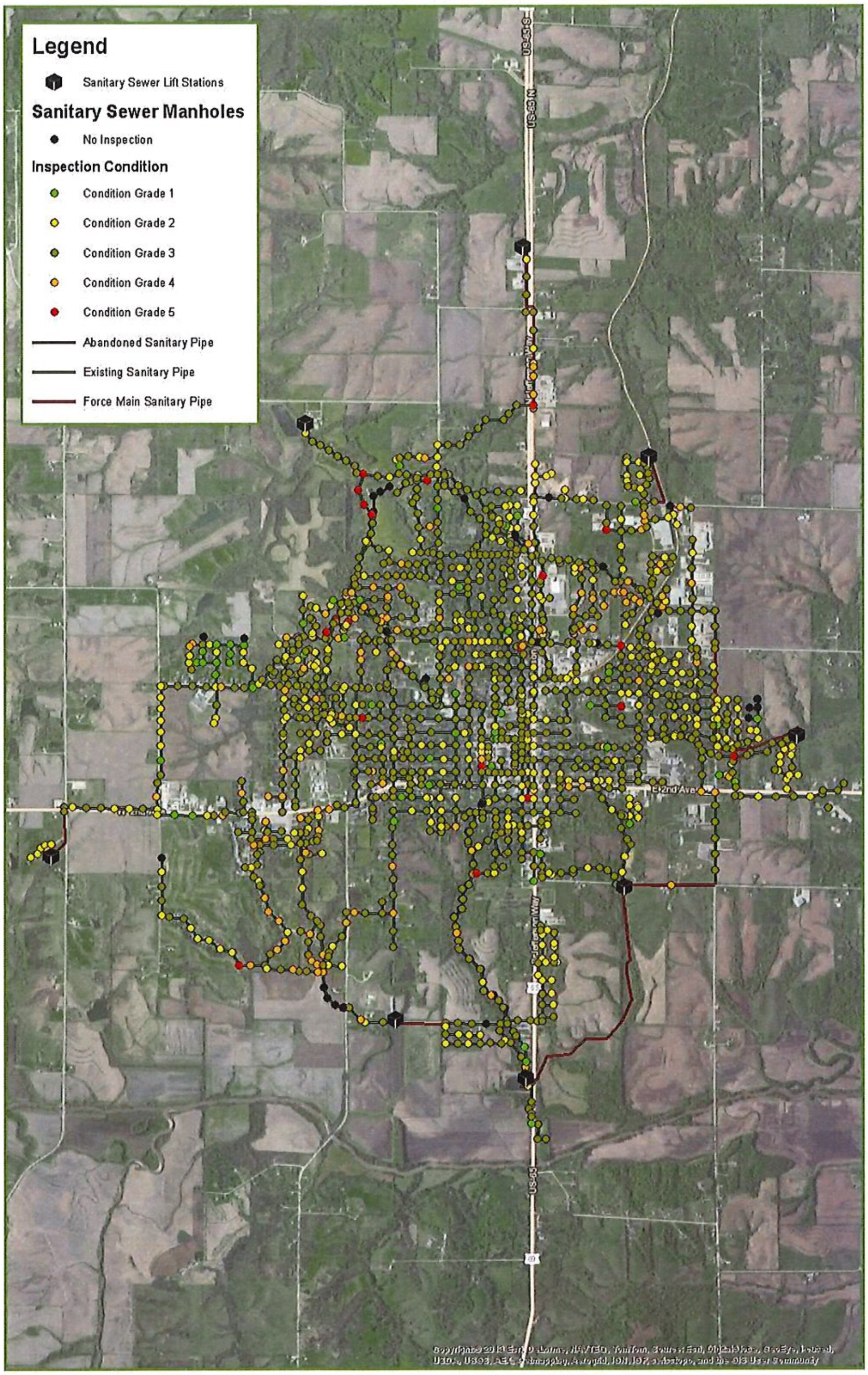


Figure 1: Manhole Condition Assessment Map

A mixture of hourly, 15-minute, and 1-minute incremental flow data was provided by the City for time periods later than September 2013. Daily flow data was also obtained from the City's monthly operating reports (MORs) as needed. Hourly rainfall data was collected from the NCDC website for nearby locations such as Knoxville, Osceola, and the Des Moines International Airport. Rainfall data from these cities was used due to the absence of incremental rainfall records for the City of Indianola. Because storms can differ substantially between small geographic areas, NOAA total rainfall maps were utilized to compare recorded rainfall totals from Indianola to the three cities listed above. Based on these NOAA maps, all rainfall data not representative of storms seen in Indianola were excluded.

V. DRY WEATHER FLOW CALIBRATION

The hydraulic model was set up by first dividing the collection system into eight catchment areas based on the number of lift stations present within the system. The eight catchments were labeled North Plant, South Plant, Morlock, McCord, Plainview, N 65/69, Q.M. and Wesley according to each catchments associated lift station. Catchments were defined as all piping and utility structures upstream of the associated lift station up to either the termination of piping or a junction with an upstream lift station.

After setting up the catchments, each manhole within the system was assigned a sanitary load based on the number of nearby residential, commercial and/or industrial properties as determined using aerial imagery. When running the model, these sanitary loads are then multiplied by a pattern (typically diurnal) to determine influent flows to each manhole at each time step throughout the day. For example, assuming the use of a typical diurnal pattern and a manhole with a sanitary loading of 10 gpd, this manhole may see an influent flow flowrate of 2 gpd at 1:00 am when persons in nearby houses are sleeping. At 8:00 am, the same manhole would likely see an influent flowrate around 15 gpm when persons in nearby houses are preparing for work.

The next step in setting up the model involves defining a representative flow pattern typical for the City of Indianola. This was completed by using historical flow data provided by the City. A December 10, 2014 North Lift Station flow of 1.2 MGD was selected for use as the baseline flow for the conveyance system. This flow occurred during a very dry period and in which inflow and infiltration were assumed to be negligible. The diurnal curve associated with this event was then used to create a unitless diurnal flow pattern which was then input into the model to be multiplied by the assigned sanitary loadings as previously discussed.

As baseline flow patterns will vary slightly between each lift station the peak and trough diurnal pattern multipliers used were adapted slightly to fit observed influent flow patterns recorded at the various lift stations. The adapted diurnal pattern can be seen in Figure 2 below. The selected base flow pattern indicates a peak flow occurring in the morning around 8:30 AM when residential users are typically preparing for the day. The second peak occurs around 8:00 PM when residential users are typically preparing for bed. After this time the flow reduces which represents the minimal activity that occurs throughout the night.

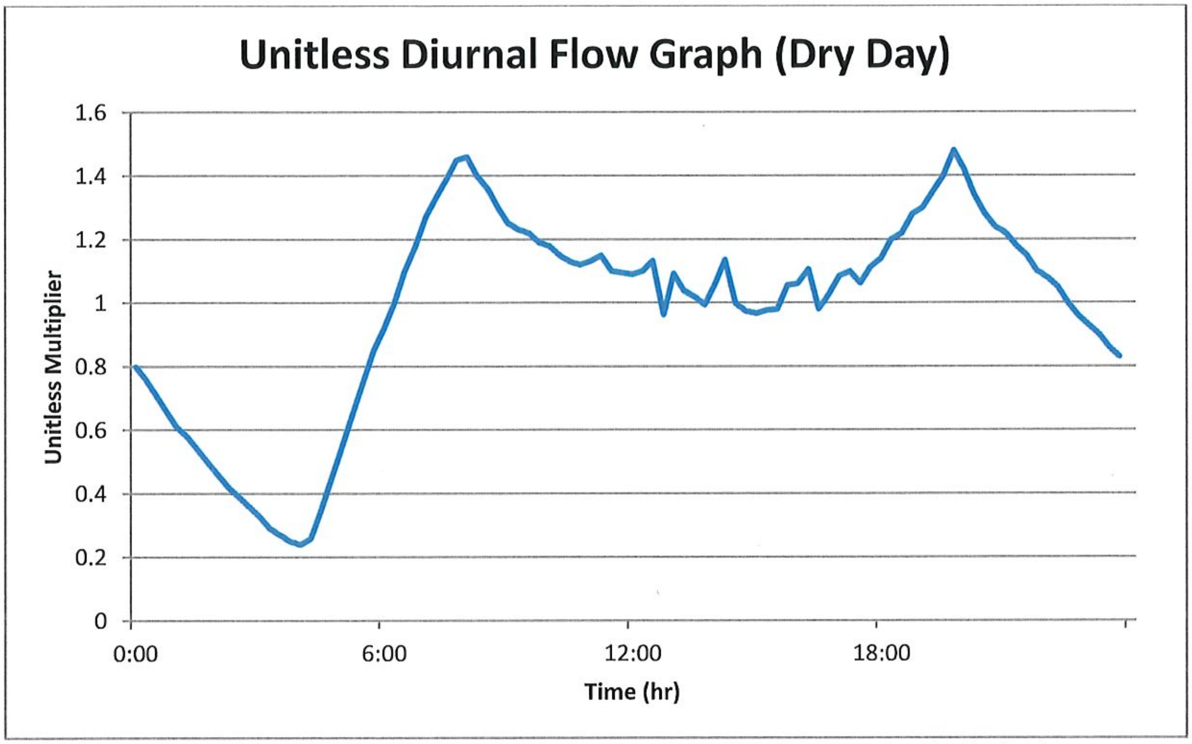


Figure 2: Adapted Unitless Diurnal Flow Pattern (Dry Day) – 12/10/2013

As can be seen Figure 3 and Table 2, modeled lift station influent flows resultant of the sanitary loading process discussed above result in pump station influent flows nearly identical to actual flows observed at the various lift stations. The overall peak dry weather flow for the pattern was observed at approximately 1,073 gpm and occurred at approximately 9:00 p.m.

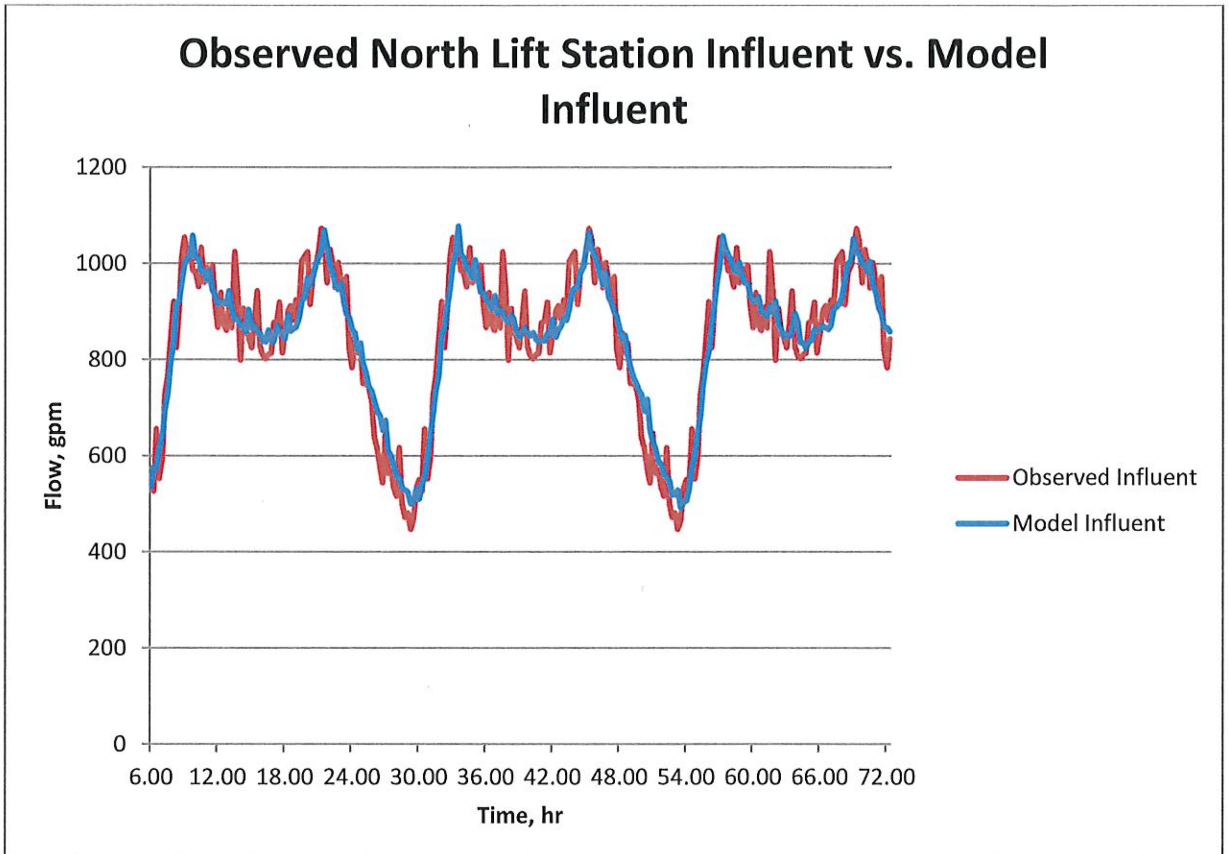


Figure 3: North Plant Influent Model Flows vs. North Plant Influent Observed Flow (Base Flow)

Table 2: Modeled vs. Observed Total Influent at Each Lift Station (Base Flow)

Lift Station Flows (Dry Weather)			
Lift Station	**Observed**	**Model**	**Error**
North Plant	1185000	1192000	-0.6%
Morlock	395000	404000	-2.3%
South Plant	220000	224000	-1.8%
McCord	65000	66000	-1.5%
Plainview	28000	28000	0.0%
N 65/69	7000	7000	0.0%
Q.M.[1]	5000	5000	0.0%
Wesley[1]	5000	5000	0.0%

*Observed flow data not provided. Assumed based on similar sized lift stations

In summary, the model indicates that the system is sized correctly to handle dry weather flow events. Under dry weather conditions, the model also indicates that no pipes will surcharge and no backups will occur. The model results are shown in Figure 4. The green pipes and structures indicate adequate capacity in the sewer pipe to transport wastewater flow.

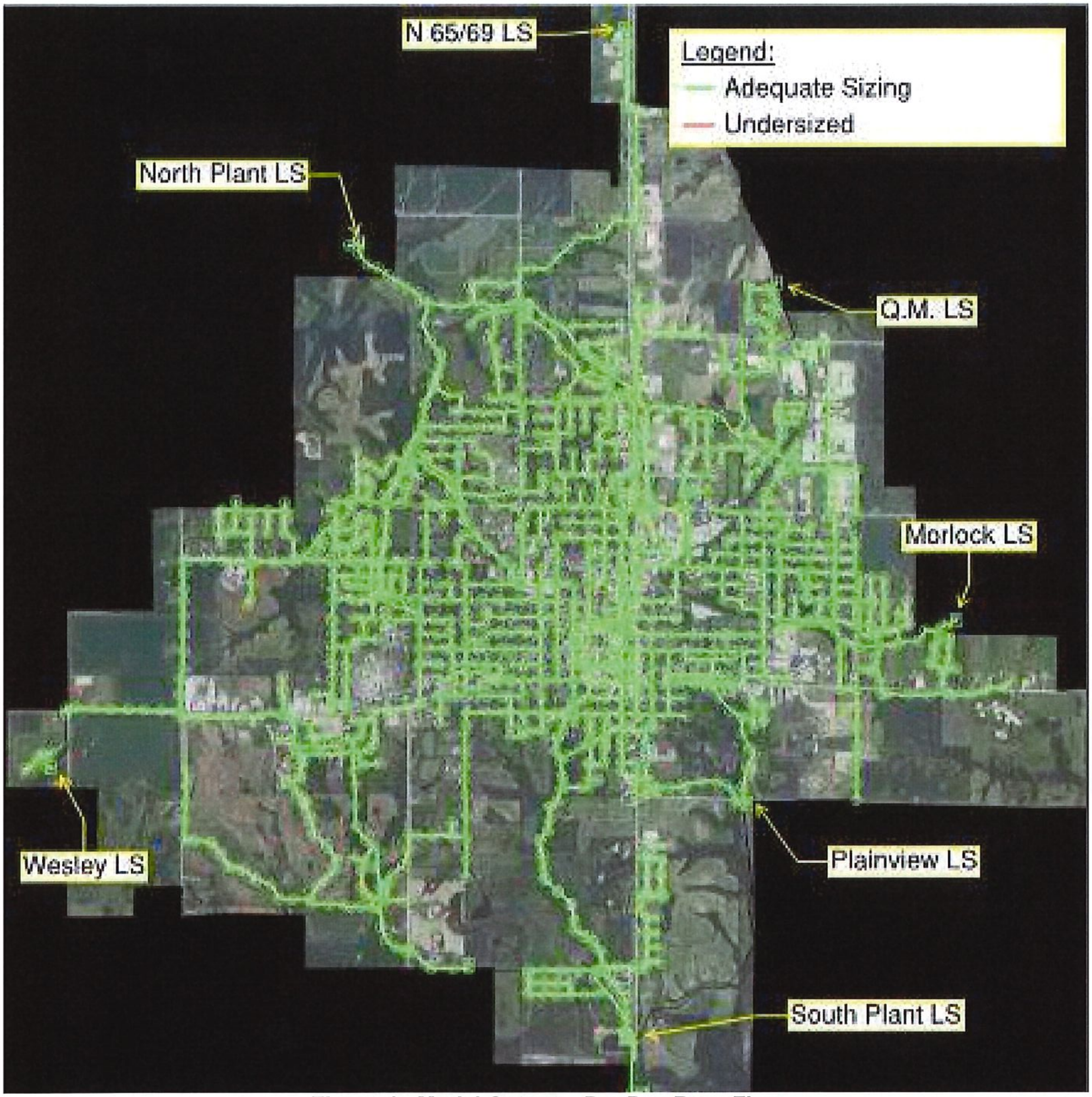


Figure 4: Model Output – Dry Day Base Flow

VI. WET WEATHER FLOW CALIBRATION

The wet weather flow was calibrated using a storm event occurring on April 13, 2014. The NOAA recorded the event as a 2.65 inch rainfall with no significant rainfall events within 12 days prior to this storm. A comparison of North Plant lift station model effluent versus observed flows is provided in Figure 5 below. The model was calibrated using this rainfall event to evaluate system performance. It should be noted that further calibration is recommended to improve performance of the model. This was not possible due to the fact that only one other significant rainfall event was recorded during the time period of observed flow data provided. In an attempt to simulate this storm event within the model, significant correlation errors between NCDC recorded rainfall events from nearby monitoring stations and recorded periods of high sewer flows were discovered. Therefore, this attempt was abandoned in lieu of further flow data to avoid calibration inaccuracy.

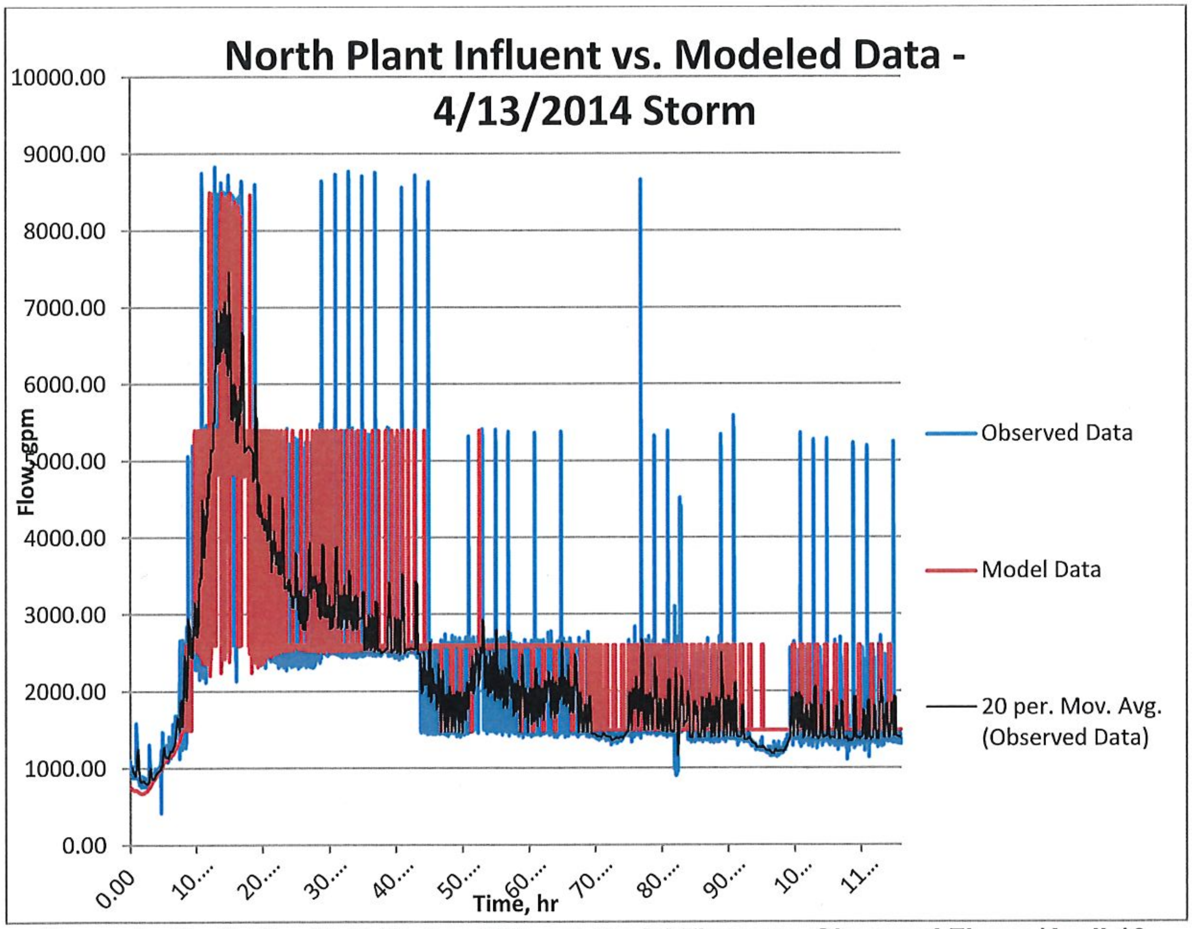


Figure 5: North Plant Lift Station Effluent Model Flows vs. Observed Flows (April 13, 2014 Rainfall Event)

In reference to the above figure, the initial response seen at the plant is typically due to inflow into the system. This is identifiable by the rapid increase in plant influent flowrate. The flowrate is typically increased in proportion with the amount of rain that falls. Once the rain ceases the inflow associated flows will decrease quickly. Inflow is typically due to cross connections with storm sewer, illegal sump pump connections or tile lines connected directly to the sanitary system instead of the storm sewer system. After this initial response, flow rates may remain higher than normal due to moderate and slow infiltration. This type of infiltration is caused by leaking and broken pipes. Water enters the system due to surface water seepage through soils to sewer services and mains and will recede as the water infiltrates deeper into the ground or when soils drop below saturation limits and the water quits moving through the soil. As can also be seen in Figure 5, there are multiple outliers or peaks within the observed data that do not show up within the model output. These peaks represent a very miniscule volume in comparison to total volumes leaving the system and should be ignored. They are a common result of small differences between model and actual calculation time steps, head conditions and/or pump settings.

Table 3 below provides a comparison of total lift station storm effluent to observed effluent volumes for the April 13, 2014 storm event. The similarity between modeled and observed flows to each lift station indicates the model is correctly calibrated to represent the conveyance system during a storm event of this caliber.

Table 3: Total Lift Station Effluent vs. Observed Effluent (April 13, 2014 Rainfall Event)

Lift Station Flows (4/13/2014 Storm Event)			
Lift Station	**Observed**	**Model**	**Error**
North Plant	17,700,000	18,300,000	3%
Morlock	5,000,000	5,100,000	2%
South Plant	3,500,000	3,500,000	0%
McCord	900,000	840,000	-7%
Plainview	420,000	410,000	-2%

As can be seen in the April 13, 2014 storm event model results shown in Figure 6 below, no surcharging is present within the system. Surcharging manholes and lift stations are indicated in red where present.

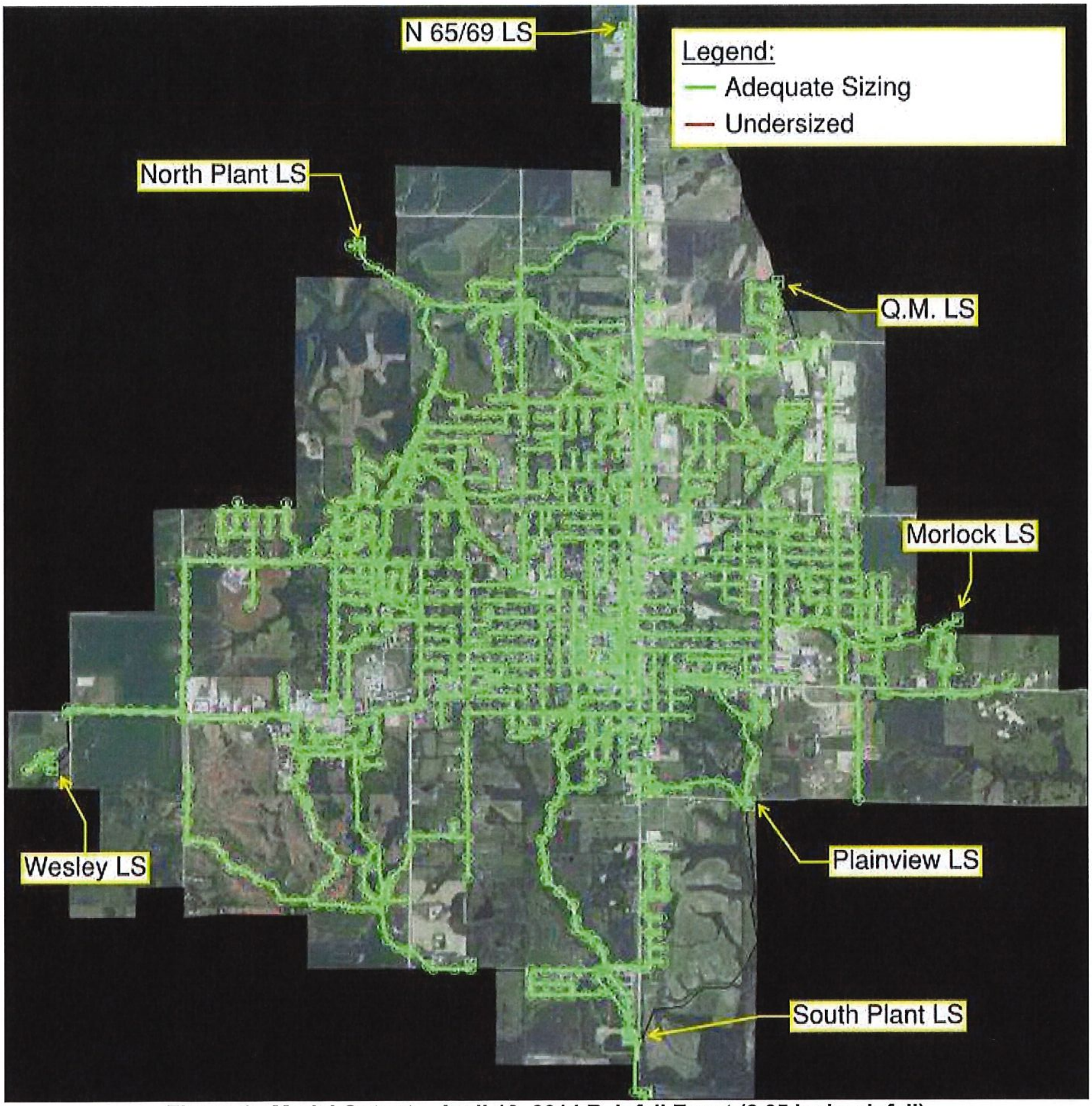


Figure 6: Model Output – April 13, 2014 Rainfall Event (2.65 inch rainfall)

VII. WET WEATHER FLOW EVALUATION

Three design rainfall events were modeled following the calibration process mentioned in previous sections. These design rainfall events were obtained directly from the NOAA website and are as follows:

1. 24 Hour Rain Event with a 10 Year Return Period (4.54 inch rainfall)
2. 24 Hour Rain Event with a 25 Year Return Period (5.59 inch rainfall)
3. 24 Hour Rain Event with a 100 Year Return Period (7.50 inch rainfall)

Table 4 below provides additional information and modeled results at the treatment plant for each of the design storm events listed above as well as the base flow event discussed in previous sections. The provided Maximum Average Daily Flows and Peak Daily Flows to the treatment plant assume no improvements to the existing collection system have been made. Thus, flows to the treatment plant during the storm events listed will increase slightly if surcharges within the system are eliminated. Table 6, discussed later in the section, provides expected flows to the treatment plant assuming all surcharges to the system have been eliminated.

Table 4: Summary of Model Output for Various Storm Events – Existing System

Event	Rainfall (in)	Maximum Average Daily Flow (MGD)	Peak Daily Flow (MGD)
Dry Weather (base flow)	0.0	1.20	1.55
10-yr, 24-hr Storm	4.54	8.11	12.45
25-yr, 24-hr Storm	5.59	9.36	14.51
100-yr, 24-hr Storm	7.5	11.51	18.21

Lift Station Improvements:

Upon running the design storm events listed above, each lift station was analyzed to identify all improvements necessary for proper function of the lift station during each event. Figure 7, Figure 8 and Figure 9 below indicate surcharging lift stations, shown in red, during these design storm events.

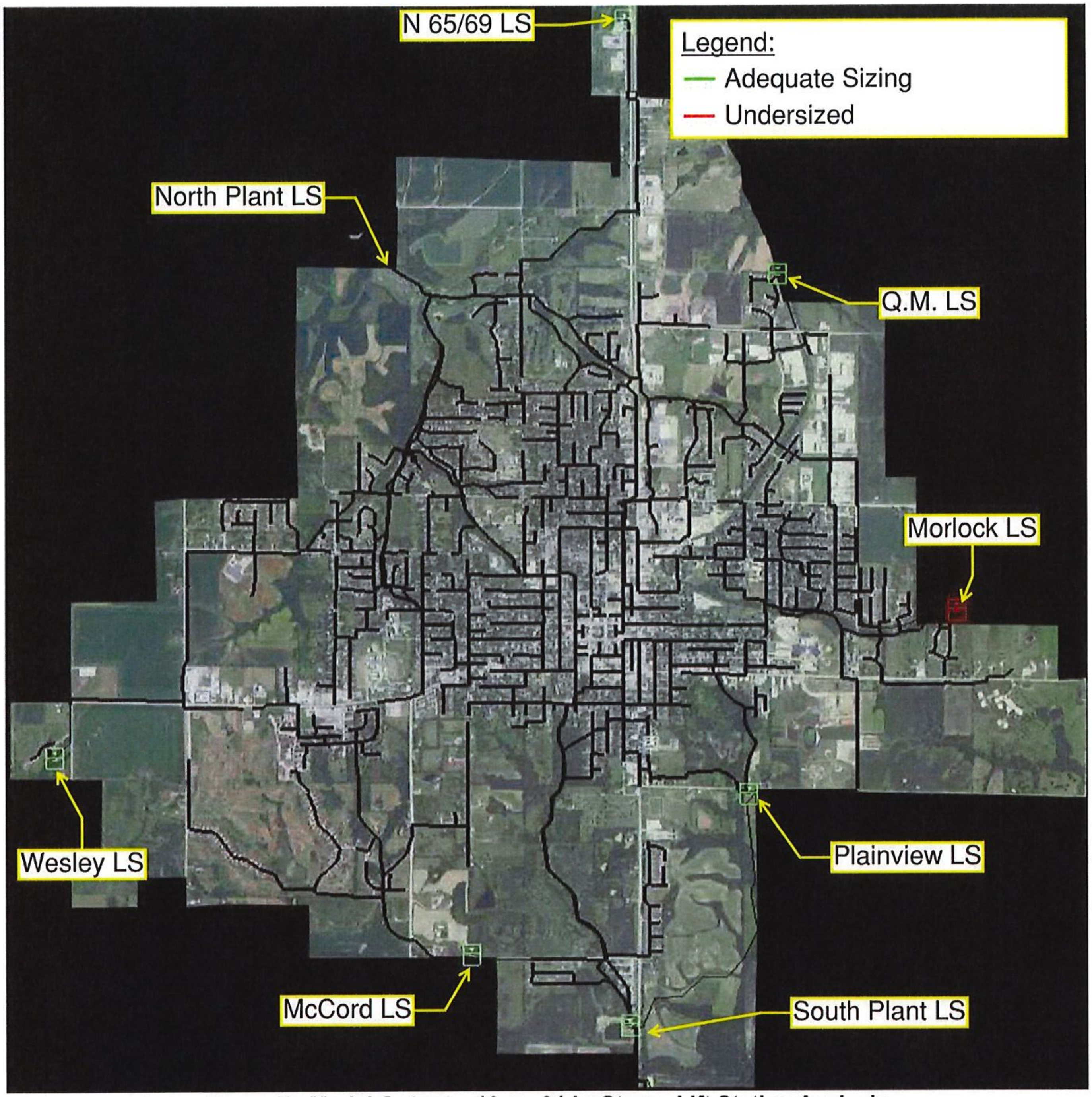


Figure 7: Model Output – 10-yr, 24-hr Storm, Lift Station Analysis

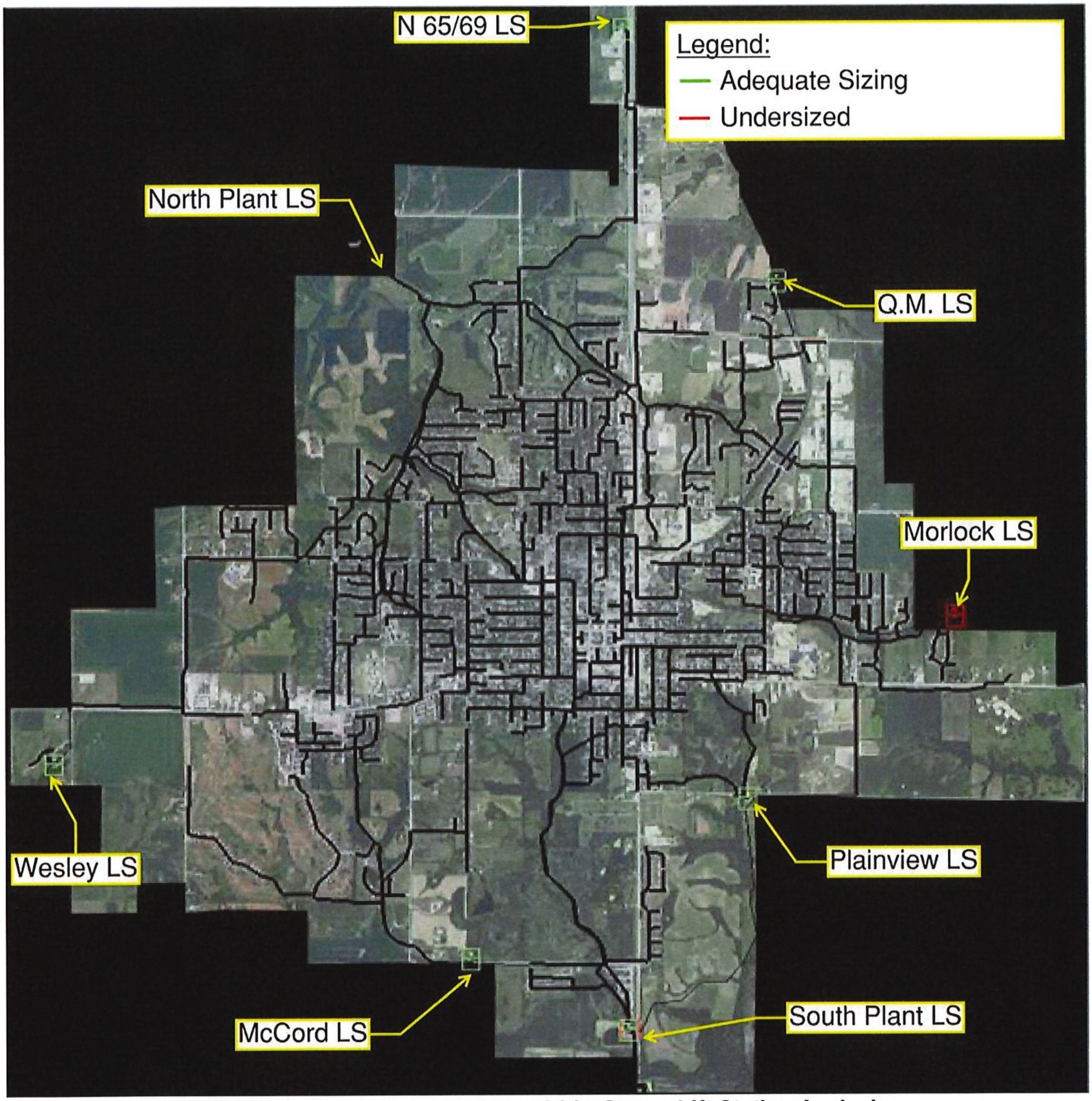


Figure 8: Model Output – 25-yr, 24-hr Storm, Lift Station Analysis

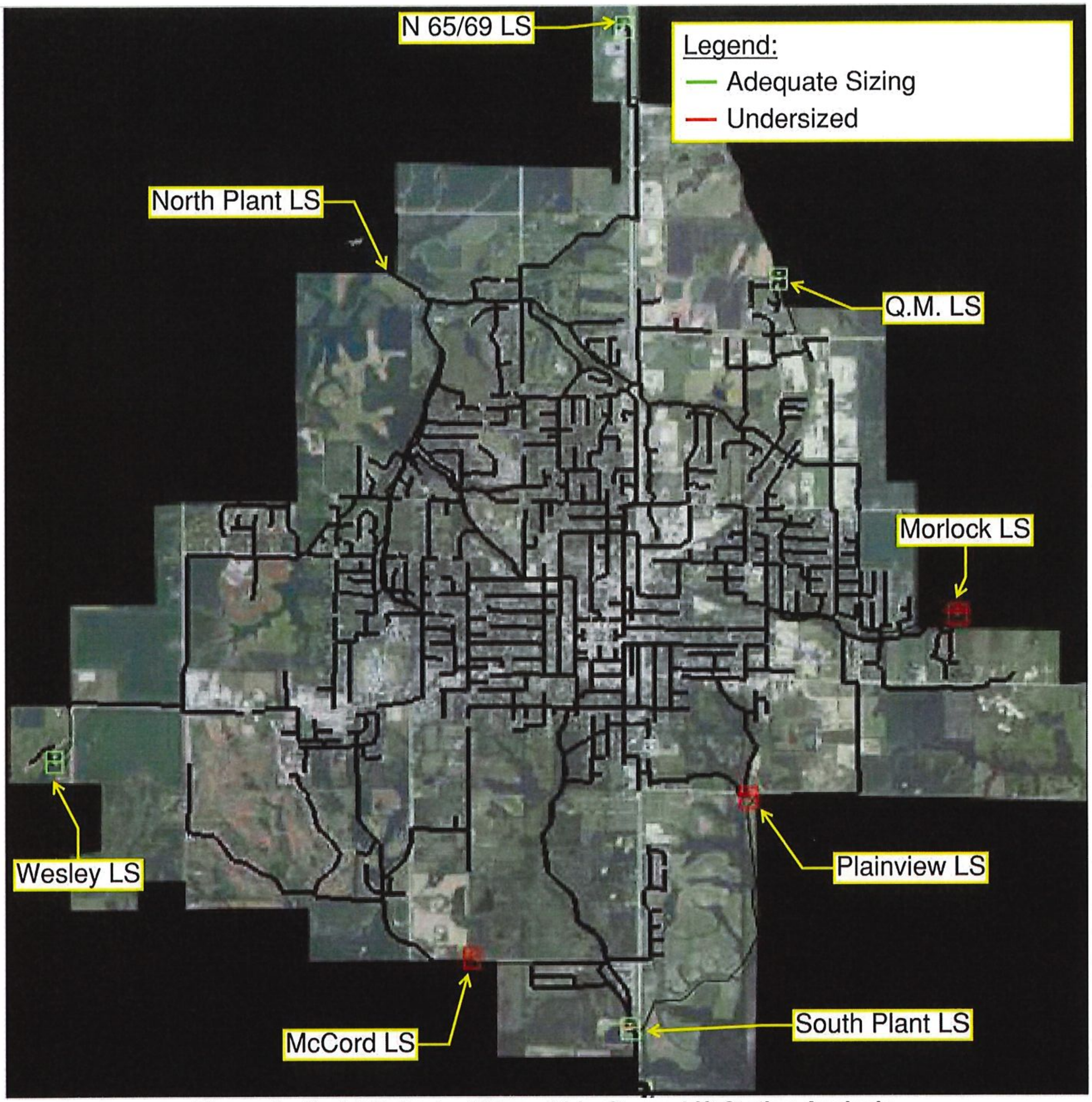


Figure 9: Model Output – 100-yr, 24-hr Storm, Lift Station Analysis

As can be seen in the figures above, multiple lift stations within the system were found to be undersized to handle certain storm events. Table 5 provides existing surcharged lift station capacities as well as the capacities required to handle each of the modeled design storm events. It should be noted that existing South Lagoon Lift Station capacities are directly tied to the capacities of the South Plant Lift Station. Thus, South Plant Lift Station capacities could be increased while South Lagoon Lift Station capacities could remain the same.

Table 5: Current Versus Required Lift Station Capacities

Event	Surcharging Lift Station	Current Capacity (All Pumps) (gpm)	Required Capacity (All Pumps) (gpm)
10-yr, 24-hr Storm			
	Morlock	1950	2900
25-yr, 24-hr Storm			
	Morlock	1950	3340
100-yr, 24-hr Storm			
	McCord	1900	2060
	South Lagoon	2000	3710
	Plainview	614	720
	Morlock	1950	4250

Due to surcharging lift stations within the system during large storm events, as seen in the previous figures, a percentage of sanitary sewer flow is not conveyed directly to the treatment plant. Thus, peak daily flows provided in Table 4 do not represent the potential peak daily flows to the system if all surcharges are eliminated. In order to determine the expected treatment plant flows if all surcharged are eliminated, the necessary improvements were made within the model to eliminate these losses. Table 6 below provides model output data summarizing the potential treatment plant flows if all influent to the conveyance system is delivered to the treatment plant.

Table 6: Summary of Model Output for Various Storm Events – Surcharges Eliminated

Event	Rainfall (in)	Maximum Average Daily Flow (MGD)	Peak Daily Flow (MGD)
10-yr, 24-hr Storm	4.54	8.36	13.67
25-yr, 24-hr Storm	5.59	9.86	16.37
100-yr, 24-hr Storm	7.5	12.55	21.28

<u>Conveyance System Improvements:</u>

Using data from the three design storm alternatives, each catchment was broken out and modeled separately to locate bottlenecks within the system. The peak daily flowrate from each catchments downstream lift station was distributed amongst the manholes in the catchment area. Manholes in higher populated areas were assigned larger loadings than in less populated areas. Model outputs for all major catchment areas for each design storm alternative are provided in the figures below along with further explanation. Unless otherwise mentioned, a green coloration within these figures indicates adequately sized utilities while red indicates undersized utilities. These figures assume all lift station surcharges within the system have been eliminated. Model output for the Q.M. and Wesley lift stations were not included below as flow meter data was not provided for these structures. The N 65/69 Lift station is also excluded due to obvious inconsistences between flow meter data provided for the April 13 calibration storm and obtained rainfall data. Thus, flows from this lift station should be assumed approximate. Due to the relatively small size of this lift station compared to the rest of the system, errors to downstream segments resulting from the approximate nature of these flows will be negligible.

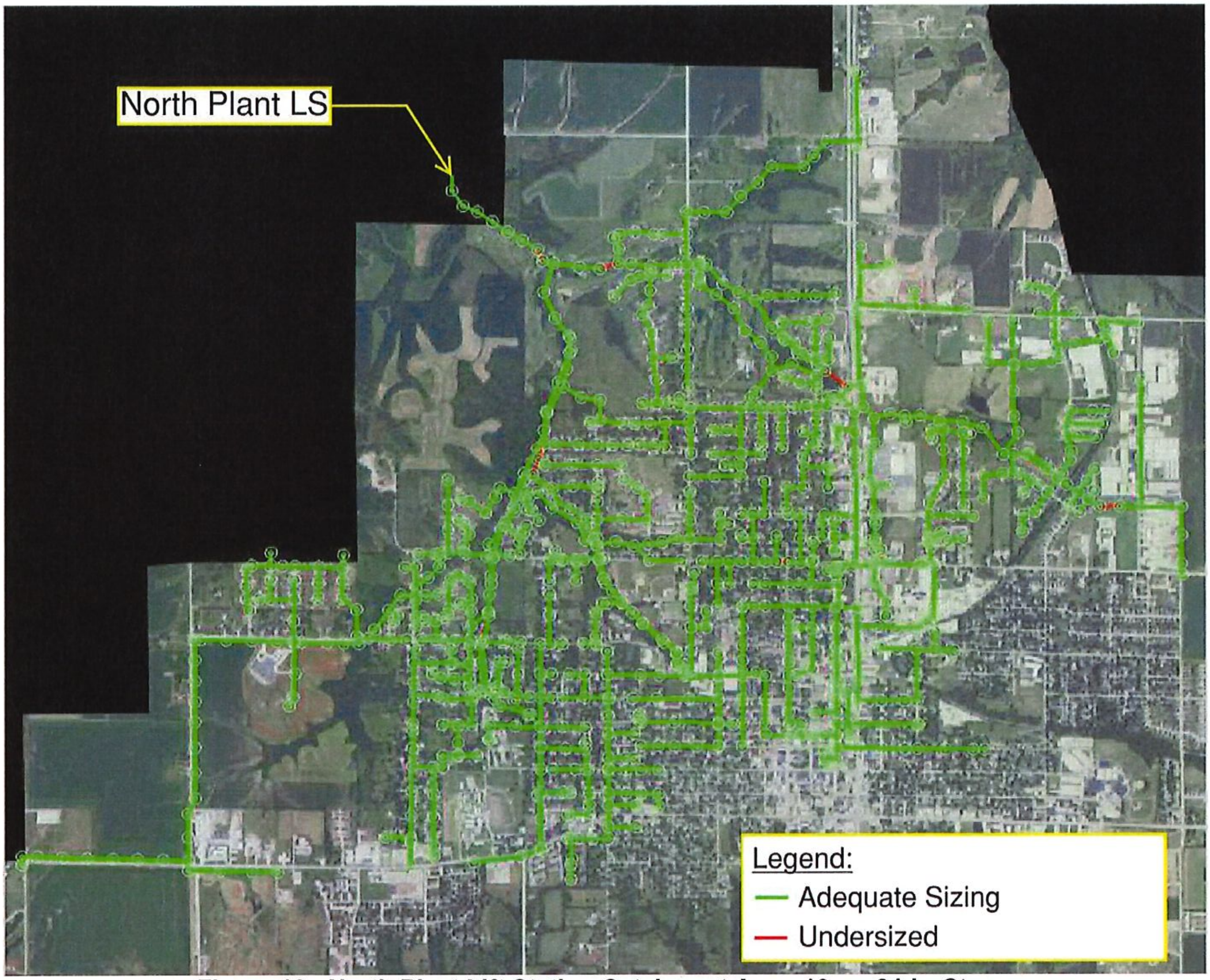


Figure 10: North Plant Lift Station Catchment Area, 10-yr, 24-hr Storm

Figure 10 above provides model output for the North Plant lift station catchment area during a 10-yr, 24-hr design storm. All manholes and piping within the catchment area were color coded green, where adequately sized, and red, where undersized. Figure 11, Figure 12, Figure 13 and Figure 14 below, provide identical model output information for the remaining lift stations. As is shown in the aforementioned figures, the system is sized to adequately handle the 10-yr, 24-hr design storm without surcharging any manholes. In a few cases, pipe flows were found to exceed pipe carrying capacities which could potentially result in limited basement back-ups.

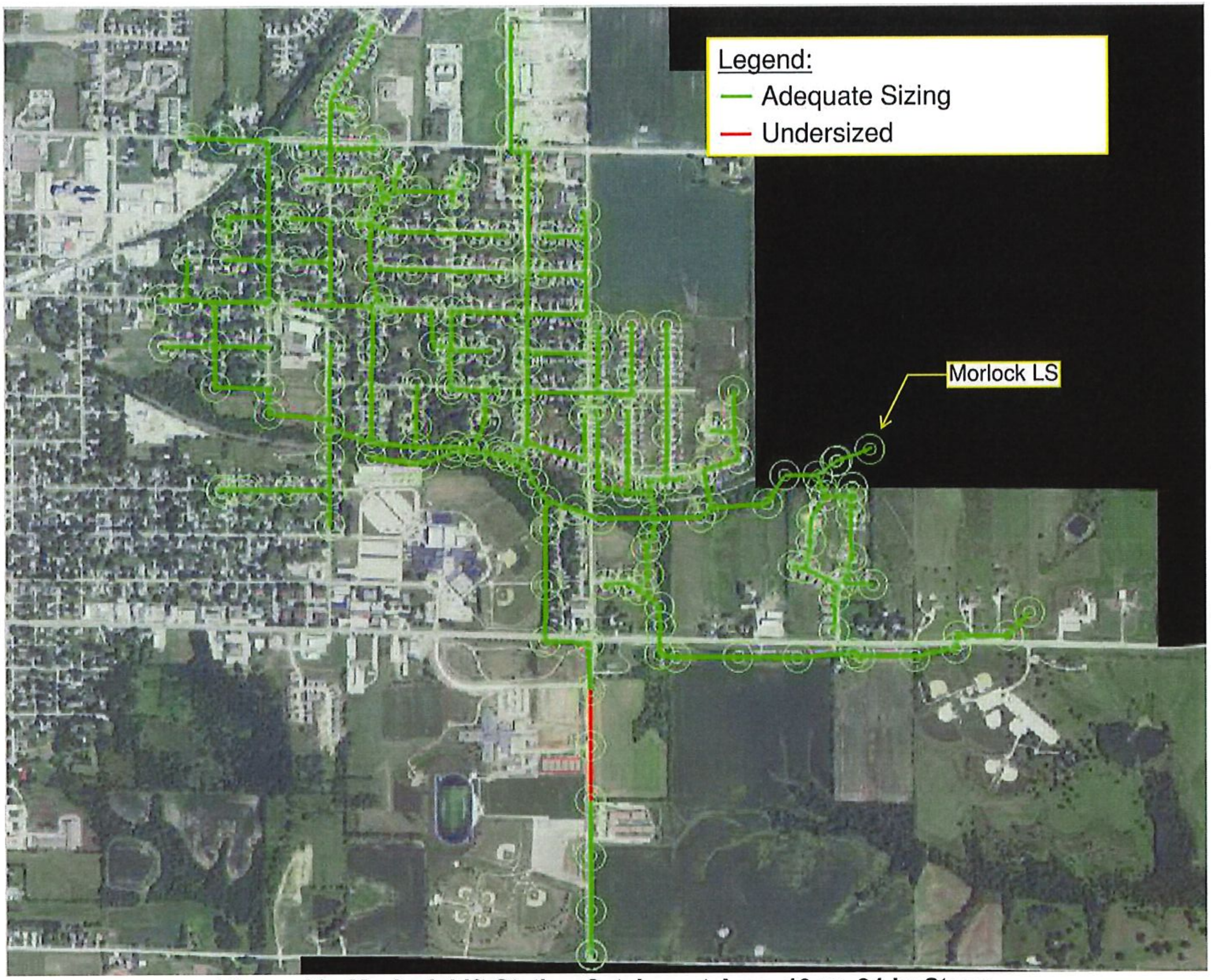


Figure 11: Morlock Lift Station Catchment Area, 10-yr, 24-hr Storm

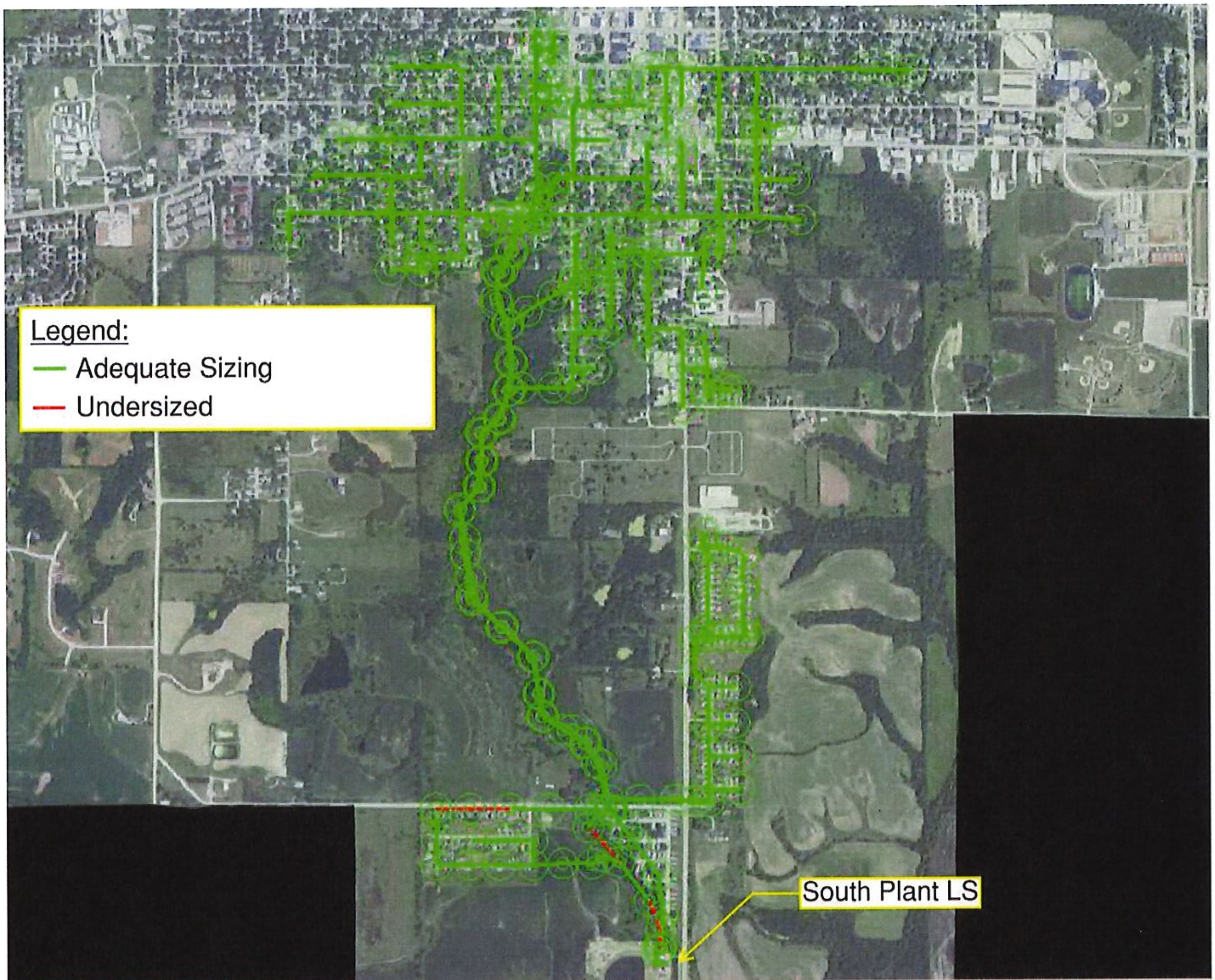


Figure 12: South Plant Lift Station Catchment Area, 10-yr, 24-hr Storm

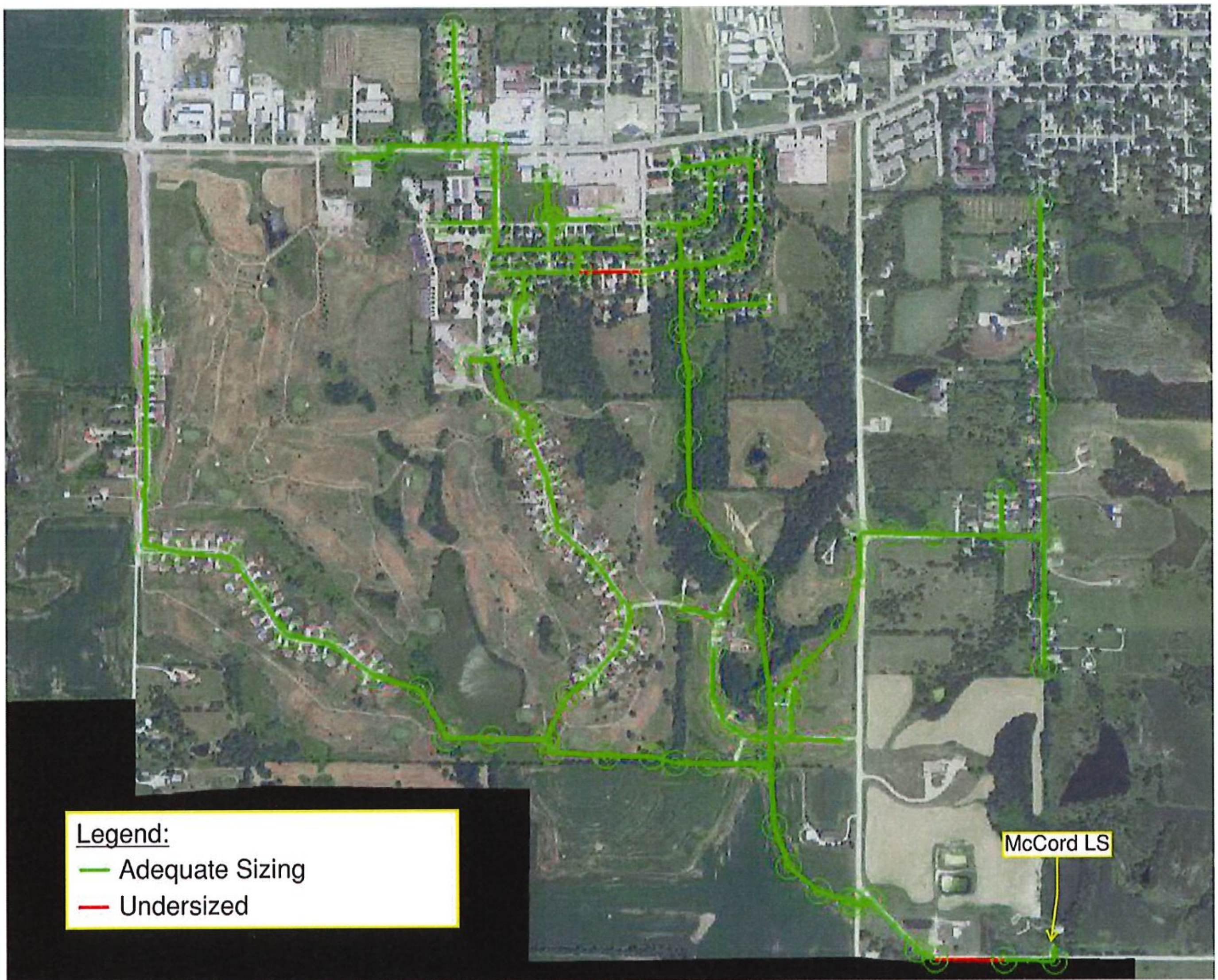


Figure 13: McCord Lift Station Catchment Area, 10-yr, 24-hr Storm

Figure 14: Plainview Lift Station Catchment Area, 10-yr, 24-hr Storm

Figure 15, Figure 18, Figure 19, Figure 20 and Figure 21 below provide model output data for all lift station catchment areas during a 25-yr, 24-hr design storm. All manholes and piping within the catchment area were color coded green, where adequately sized, and red, where undersized. As is shown in the aforementioned figures, the system is sized to adequately handle the 25-yr, 24-hr design storm without surcharging any manholes. Again, multiple pipe flows were found to exceed pipe carrying capacities which could potentially result in limited basement back-ups.

As sewer conveyance systems are commonly designed to handle a 25-yr, 24-hr storm, improvements to the system, as provided in Table 7 through Table 11, are based on this design storm event.

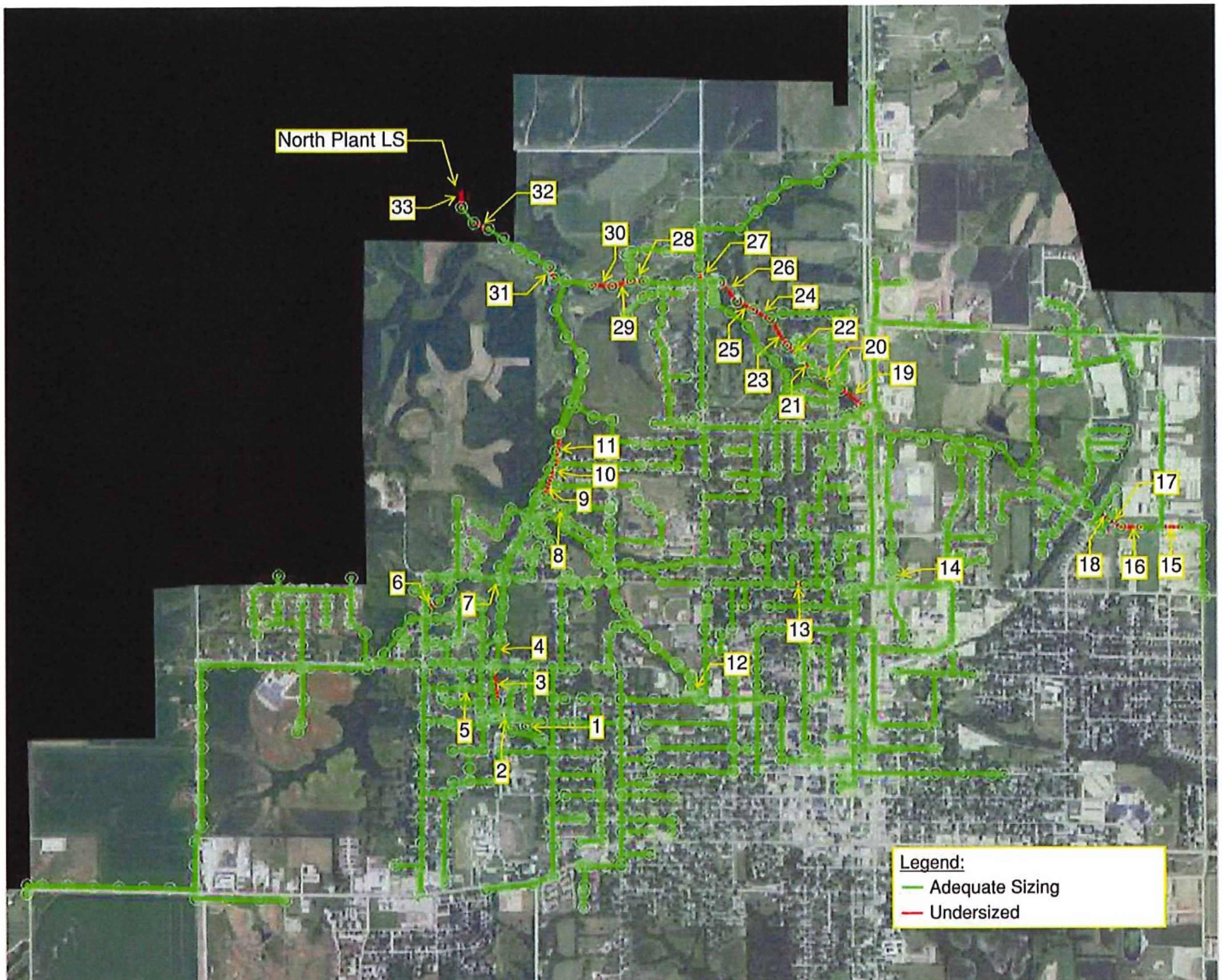


Figure 15: North Plant Lift Station Catchment Area, 25-yr, 24-hr Storm

Table 7: North Plant Lift Station Catchment Repair Recommendations, 25-yr, 24-hr Storm

Description	Type	Issue	Issue During 10-Yr Storm Event	Improvements Recommended
1	Pipe	Pipe Cap.	Yes (Minor)	Minor surcharging, no repairs recommended
2	Pipe	Pipe Cap.	Yes	Increase to 12" piping from MH 25 to MH NW-19A
3	Pipe	Pipe Cap.	No	Increase to 12" piping from MH 25 to MH NW-19A
4	Pipe	Pipe Cap.	Yes	Increase to 12" piping from MH 25 to MH NW-19A
5	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended
6	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended

7	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended
8	Pipe	Pipe Cap.	Yes (Minor)	Minor surcharging, no repairs recommended
9	Pipe	Pipe Cap.	Yes (Minor)	Increase to 15" piping from MH 14 to MH 11
10	Pipe	Pipe Cap.	Yes (Minor)	Increase to 15" piping from MH 14 to MH 11
11	Pipe	Pipe Cap.	Yes (Minor)	Increase to 15" piping from MH 14 to MH 11
12	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended
13	Pipe	Pipe Cap.	Yes (Minor)	Minor surcharging, no repairs recommended
14	Pipe	Pipe Cap.	Yes (Minor)	Minor surcharging, no repairs recommended
15	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended
16	Pipe	Pipe Cap.	Yes (Minor)	Minor surcharging, no repairs recommended
17	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended
18	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended
19	Pipe	Pipe Cap.	Yes (Minor)	Increase to 24" piping from MH NE9 to MH NE1
20	Pipe	Pipe Cap.	No	Increase to 24" piping from MH NE9 to MH NE1
21	Pipe	Pipe Cap.	No	Increase to 24" piping from MH NE9 to MH NE1
22	Pipe	Pipe Cap.	No	Increase to 24" piping from MH NE9 to MH NE1
23	Pipe	Pipe Cap.	No	Increase to 24" piping from MH NE9 to MH NE1
24	Pipe	Pipe Cap.	No	Increase to 24" piping from MH NE9 to MH NE1
25	Pipe	Pipe Cap.	No	Increase to 24" piping from MH NE9 to MH NE1
26	Pipe	Pipe Cap.	No	Increase to 24" piping from MH NE9 to MH NE1
27	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended
28	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended
29	Pipe	Pipe Cap.	Yes (Minor)	Minor surcharging, no repairs recommended
30	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended
31	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended
32	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended

33	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended

Pipe sections with less than two feet of surcharge, as demonstrated by Figure 16, were classified as minor surcharge events and no improvements were recommended. This is based on the assumption that pipe water levels must exceed two feet above the top of pipe before basement flooding becomes a likely issue. Improvement recommendations were provided for all pipes exceeding two feet of surcharge, as demonstrated by Figure 17. Improvement recommendations were not provided for manhole structures unless overflowing. Figure 16 and Figure 17 were included in the report to provide an example of the process used to identify potential issues related to surcharging in the sewer system.

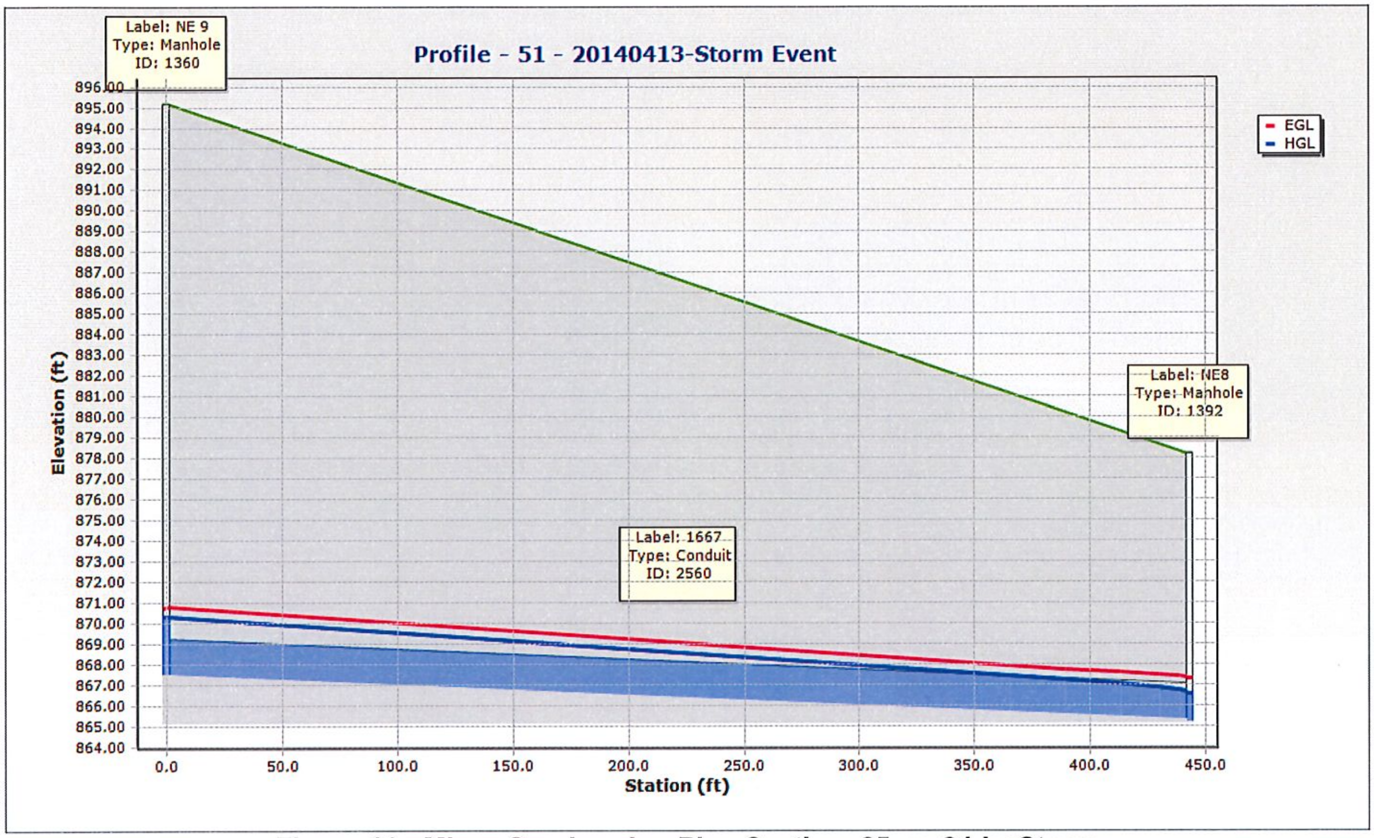


Figure 16: Minor Surcharging Pipe Section, 25-yr, 24-hr Storm

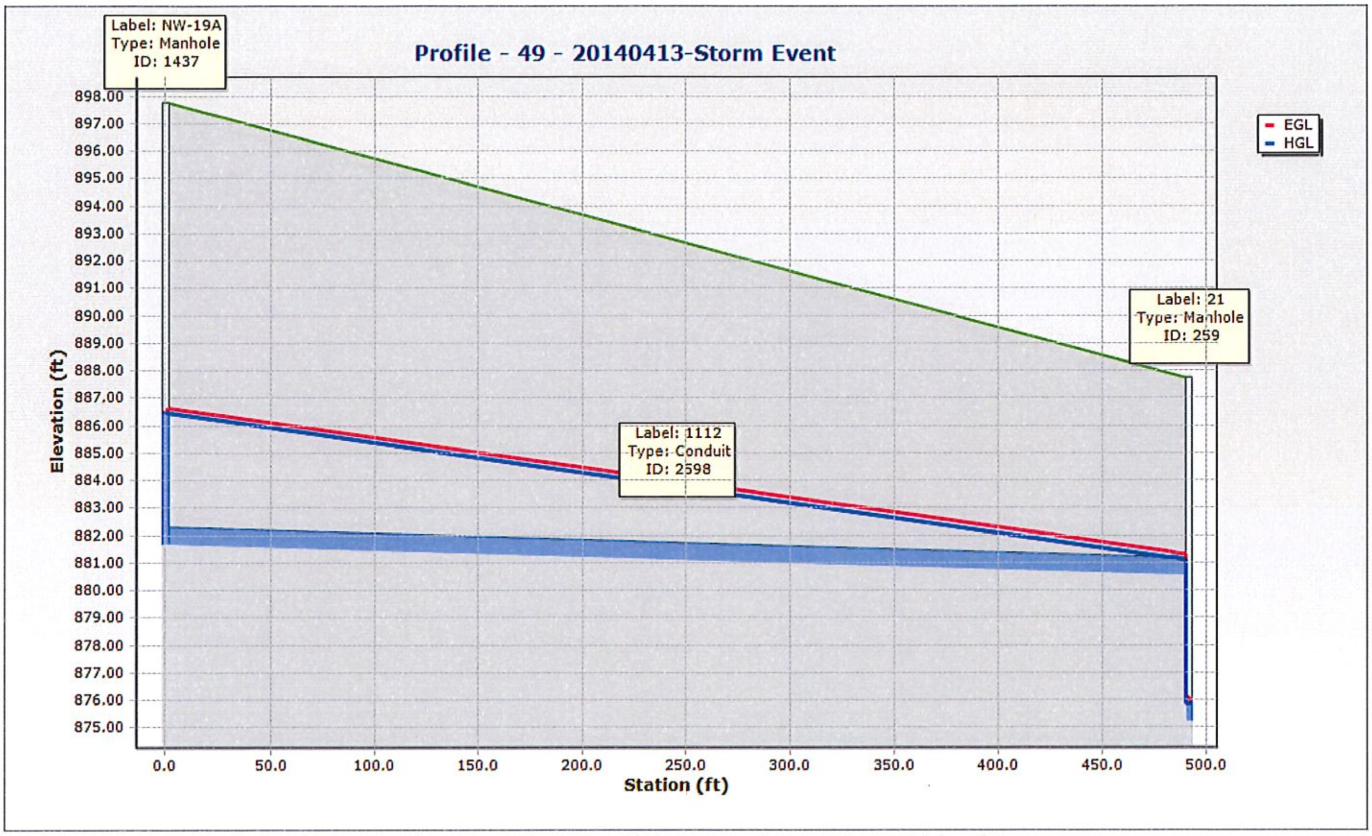


Figure 17: Surcharging Pipe Section, 25-yr, 24-hr Storm

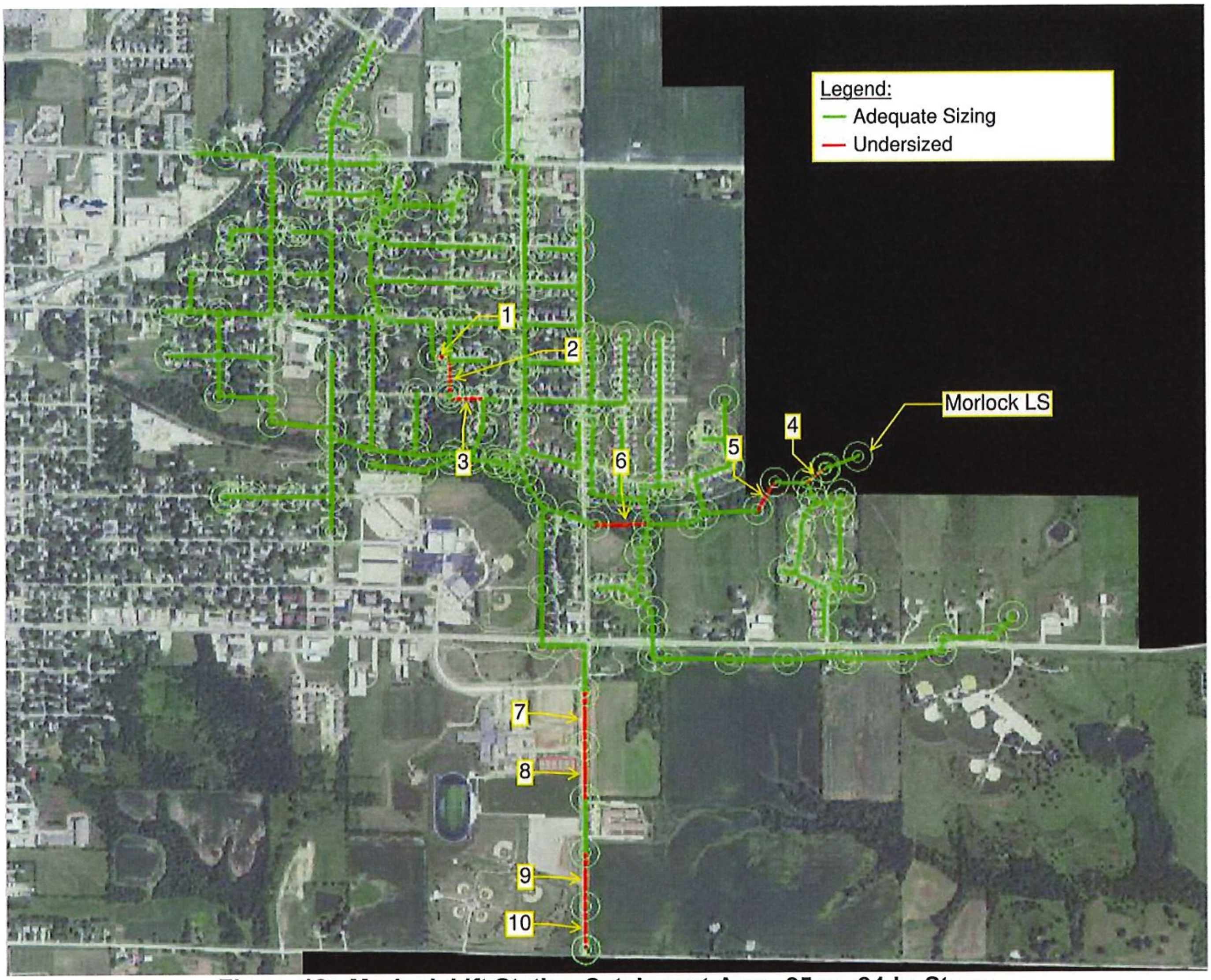


Figure 18: Morlock Lift Station Catchment Area, 25-yr, 24-hr Storm

Table 8: Morlock Lift Station Catchment Repair Recommendations, 25-yr, 24-hr Storm

Description	Type	Issue	Issue During 10-Yr Storm Event	Improvements Recommended
1	Pipe	Pipe Cap.	No	Increase to 10" piping from MH 750 to MH 507
2	Pipe	Pipe Cap.	No	Increase to 10" piping from MH 750 to MH 507
3	Pipe	Pipe Cap.	No	Increase to 10" piping from MH 750 to MH 507
4	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended
5	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended
6	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended

7	Pipe	Pipe Cap.	Yes (Minor)	Minor surcharging, no repairs recommended
8	Pipe	Pipe Cap.	Yes (Minor)	Minor surcharging, no repairs recommended
9	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended
10	Pipe	Pipe Cap.	No	Minor surcharging, no repairs recommended

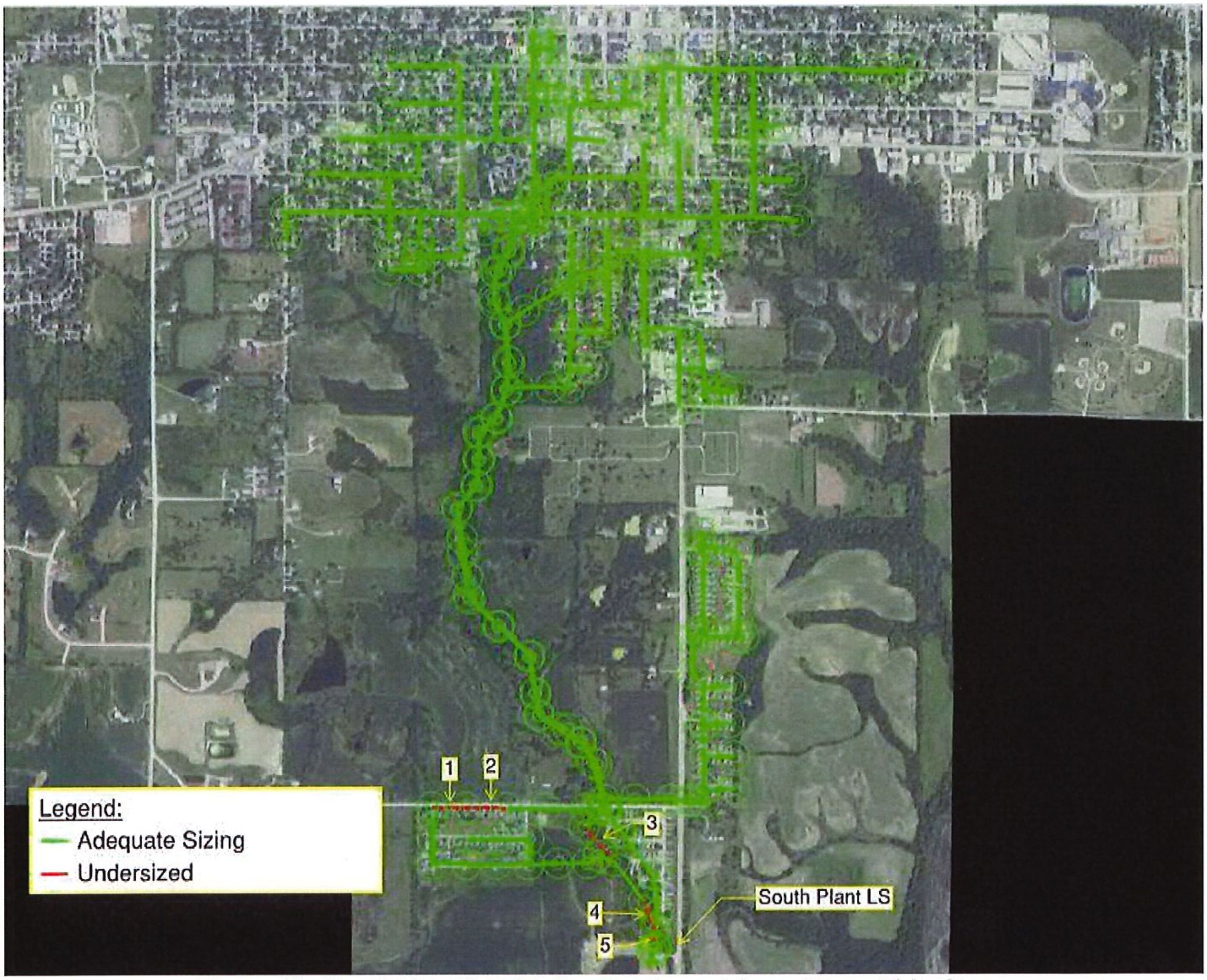


Figure 19: South Plant Lift Station Catchment Area, 25-yr, 24-hr Storm

Table 9: South Plant Lift Station Catchment Repair Recommendations, 25-yr, 24-hr Storm

Description	Type	Issue	Issue During 10-Yr Storm Event	Improvements Recommended
1	Pipe	Pipe Cap.	Yes	Increase to 21" piping from MH S105 to MH S103
2	Pipe	Pipe Cap.	Yes	Increase to 21" piping from MH S105 to MH S103
3	Pipe	Pipe Cap.	Yes (Minor)	Minor surcharging, no repairs recommended
4	Pipe	Pipe Cap.	Yes (Minor)	Minor surcharging, no repairs recommended
5	Pipe	Pipe Cap.	Yes (Minor)	Minor surcharging, no repairs recommended

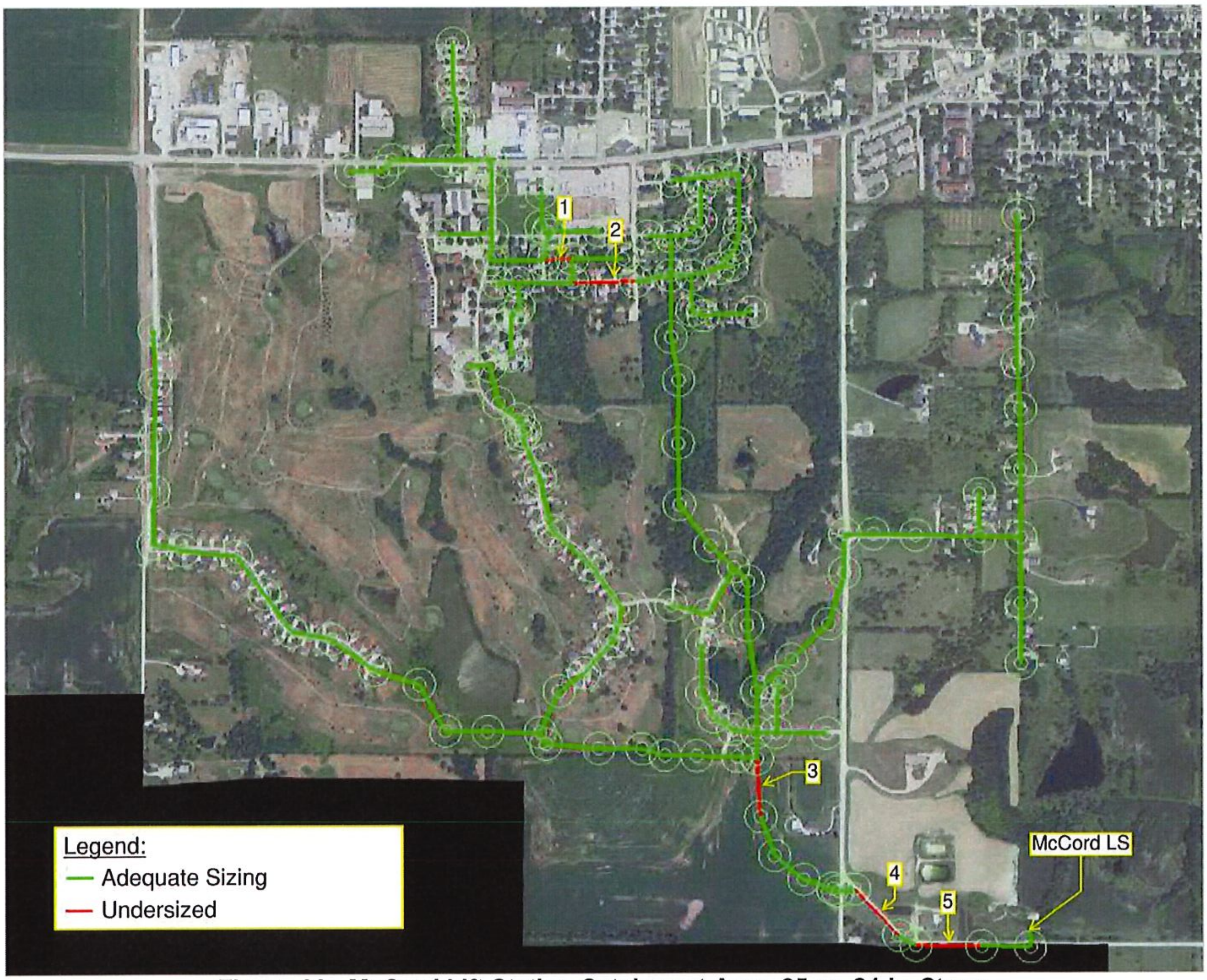


Figure 20: McCord Lift Station Catchment Area, 25-yr, 24-hr Storm

Table 10: McCord Lift Station Catchment Repair Recommendations, 25-yr, 24-hr Storm

Description	Type	Issue	Issue During 10-Yr Storm Event	Improvements Recommended
1	Pipe	Pipe Cap.	No	Increase to 10" piping from MH 56 to MH 50
2	Pipe	Pipe Cap.	Yes (Minor)	Increase to 10" piping from MH 56 to MH 50
3	Pipe	Pipe Cap.	No	Minor surcharging, no repairs required
4	Pipe	Pipe Cap.	No	Increase to 18" piping from MH S205 to MH S202
5	Pipe	Pipe Cap.	Yes (Minor)	Increase to 18" piping from MH S205 to MH S202

Figure 21: Plainview Lift Station Catchment Area, 25-yr, 24-hr Storm

Table 11: Plainview Lift Station Catchment Repair Recommendations, 25-yr, 24-hr Storm

Description	Type	Issue	Issue During 10-Yr Storm Event	Improvements Recommended
No Improvements recommended				

Figure 22 through Figure 36 below provide model output data for all lift station catchment areas during a 100-yr, 24-hr design storm. There are three figures provided for each catchment area. The first figure for each area identifies all undersized manholes and piping within the existing system. The second figure for each area identifies surcharging manholes within the system. The third figure for each area identifies all undersized manholes and piping within the system assuming all of the 25-yr, 24-hr design storm improvement recommendations are completed. All manholes and piping within the catchments were color coded green, where adequately sized, and red, where undersized.

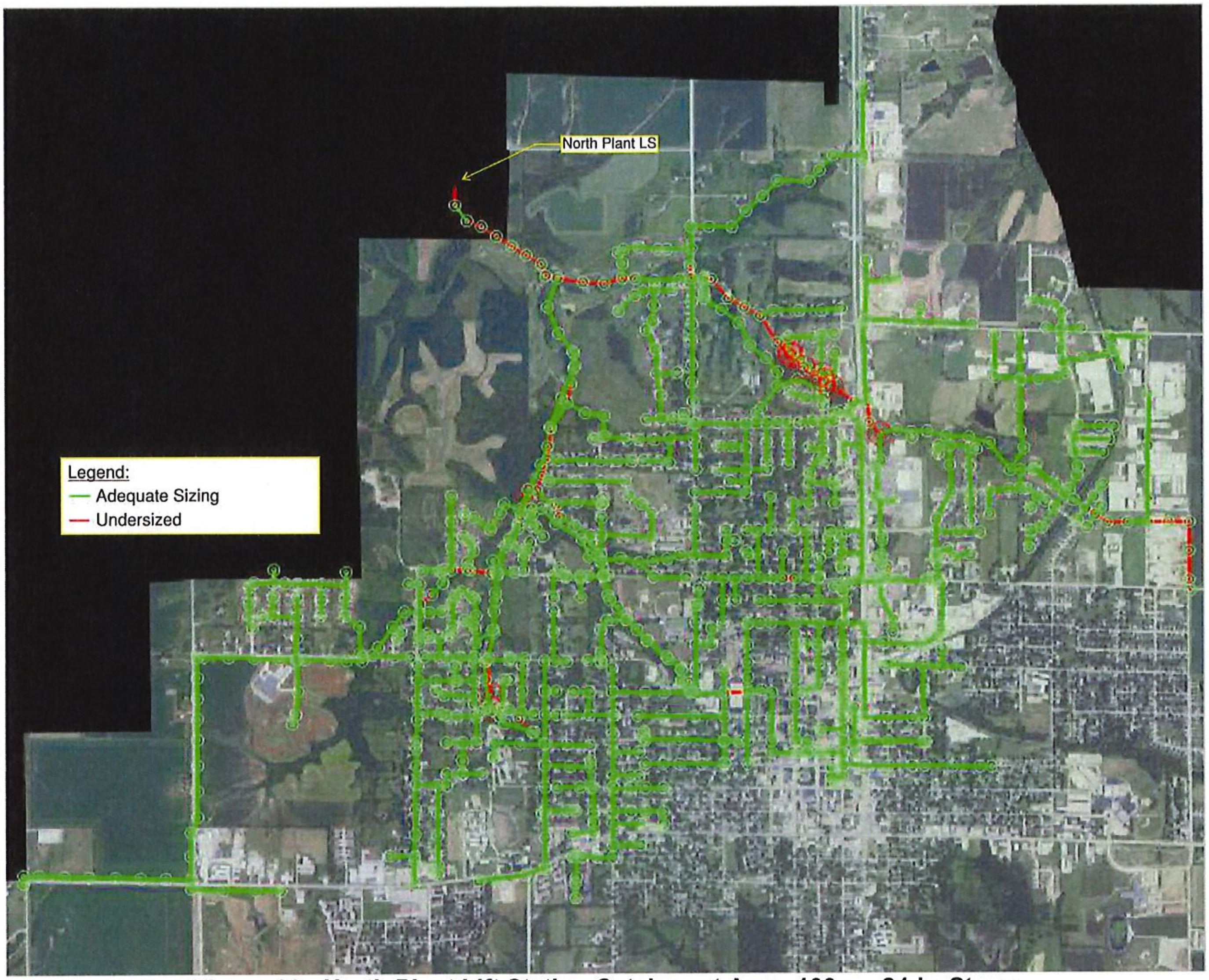


Figure 22: North Plant Lift Station Catchment Area, 100-yr, 24-hr Storm

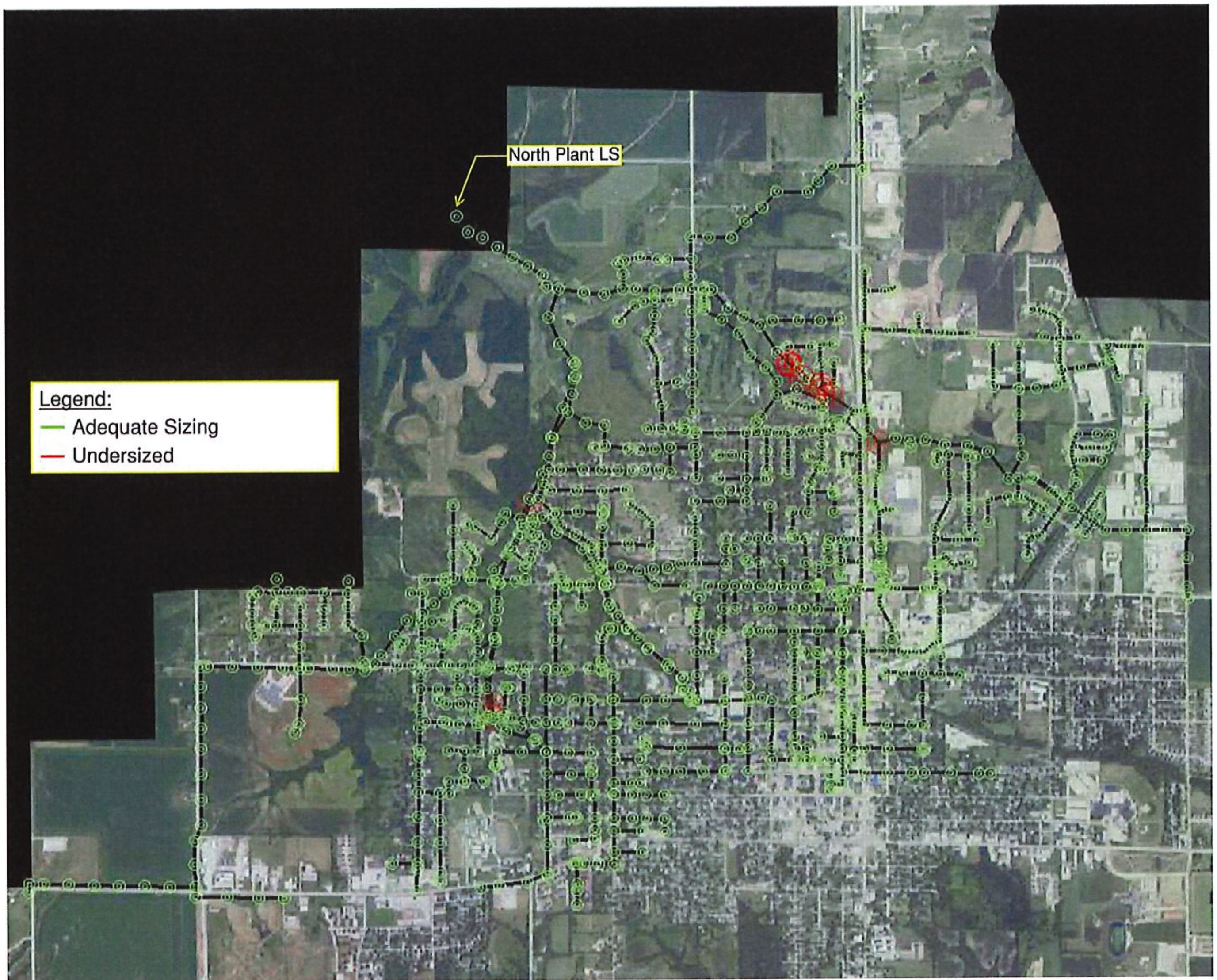


Figure 23: North Plant Lift Station Catchment Area Overflows, 100-yr, 24-hr Storm

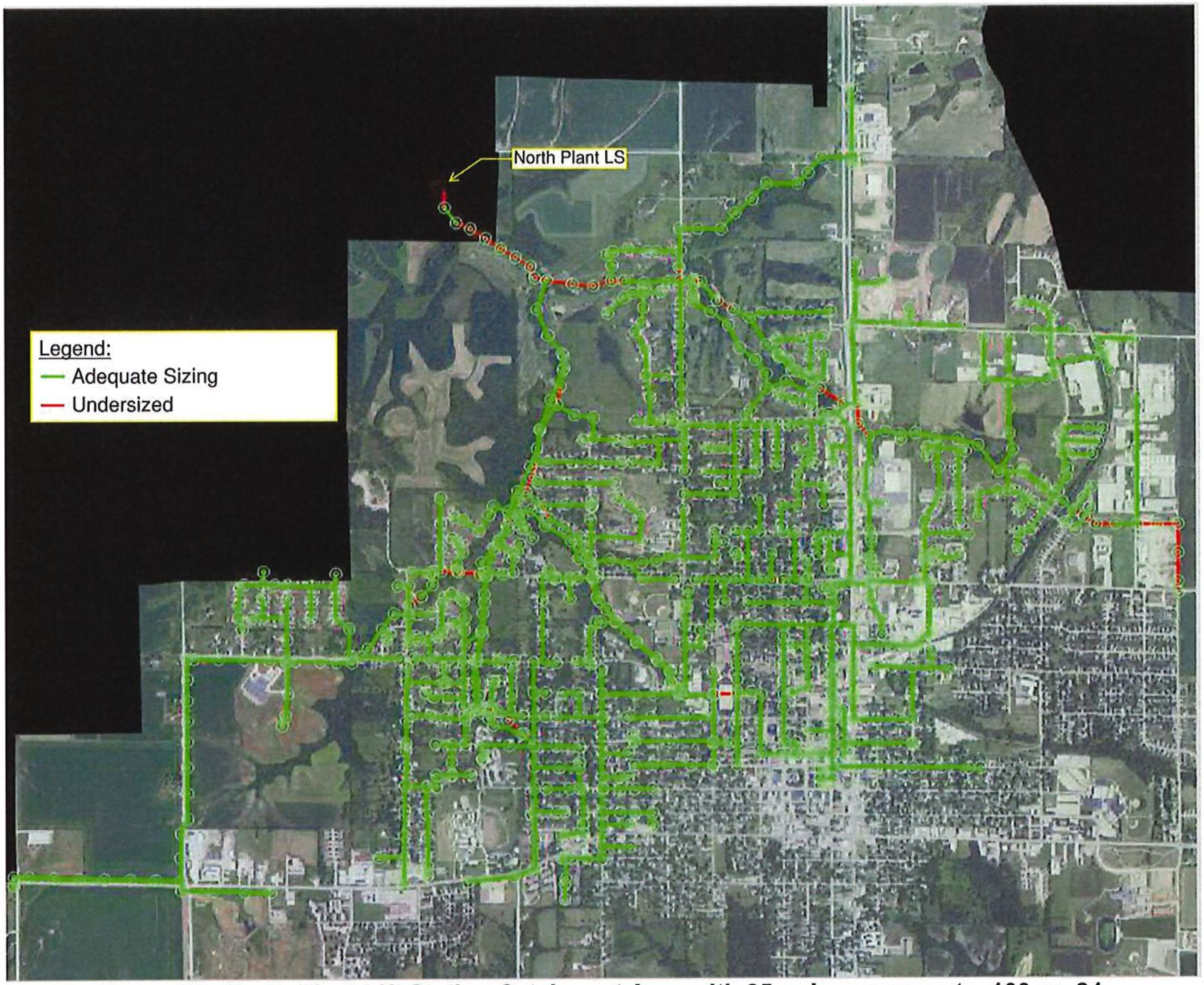


Figure 24: North Plant Lift Station Catchment Area with 25-yr Improvements, 100-yr, 24-hr Storm

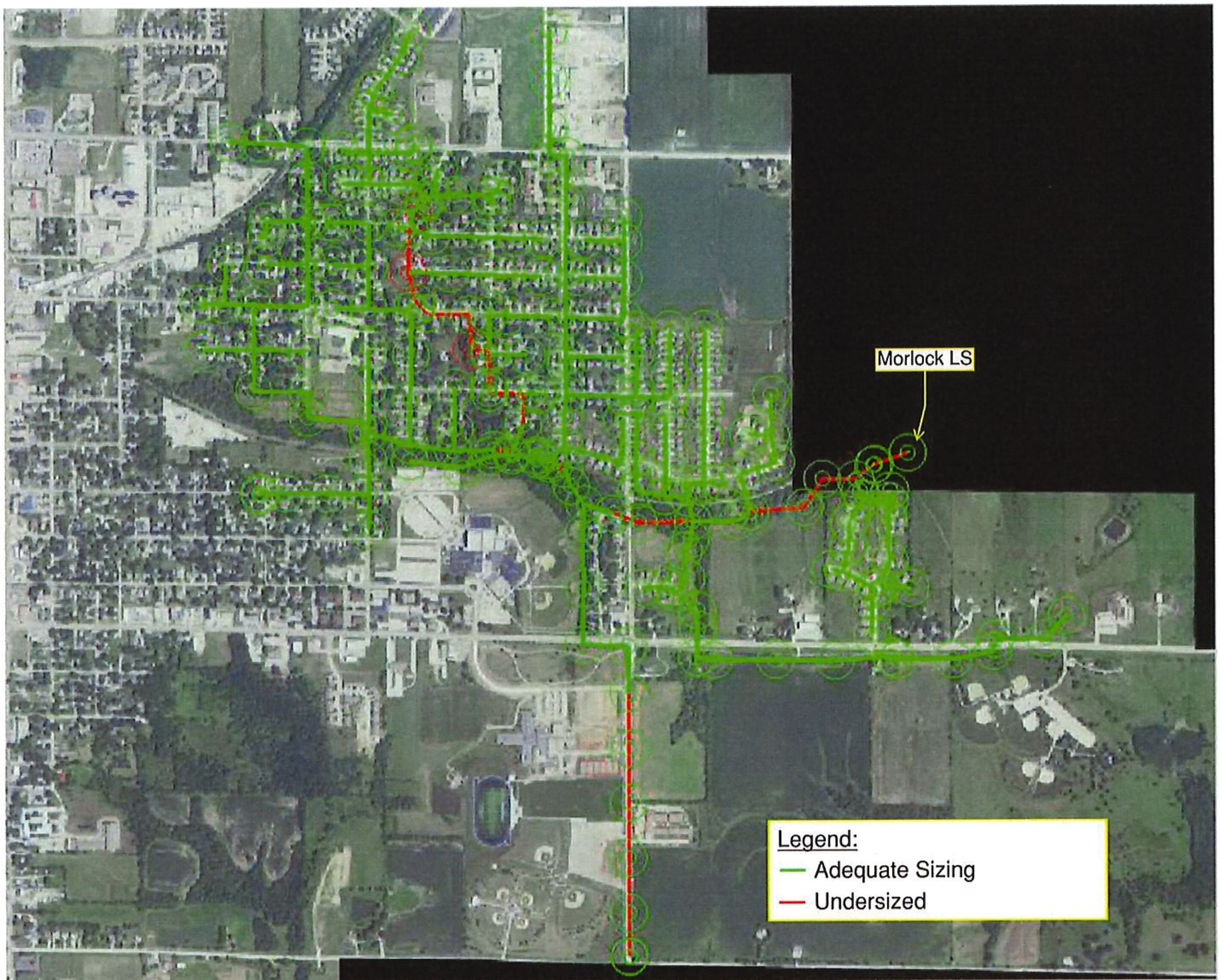


Figure 25: Morlock Lift Station Catchment Area, 100-yr, 24-hr Storm

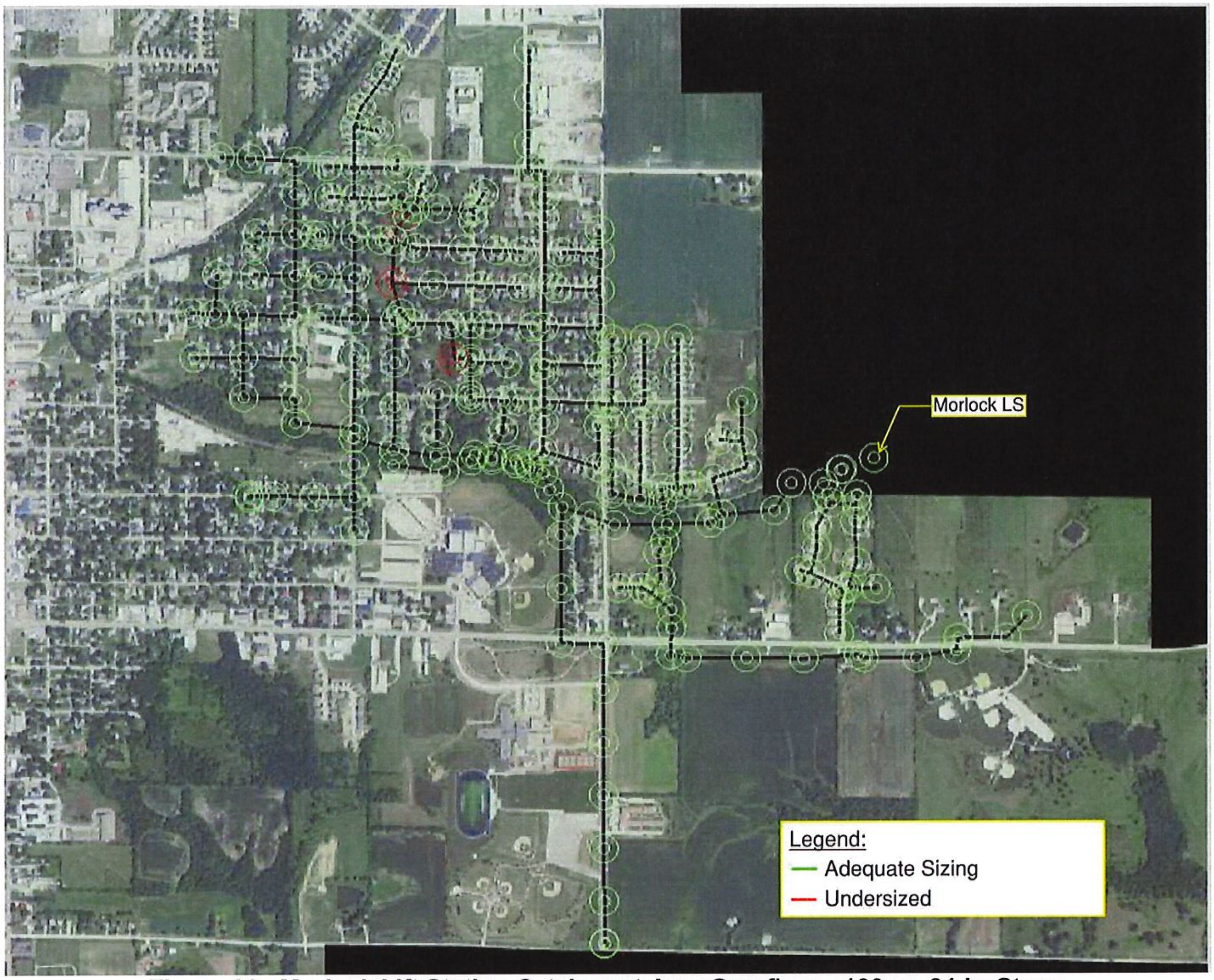


Figure 26: Morlock Lift Station Catchment Area Overflows, 100-yr, 24-hr Storm

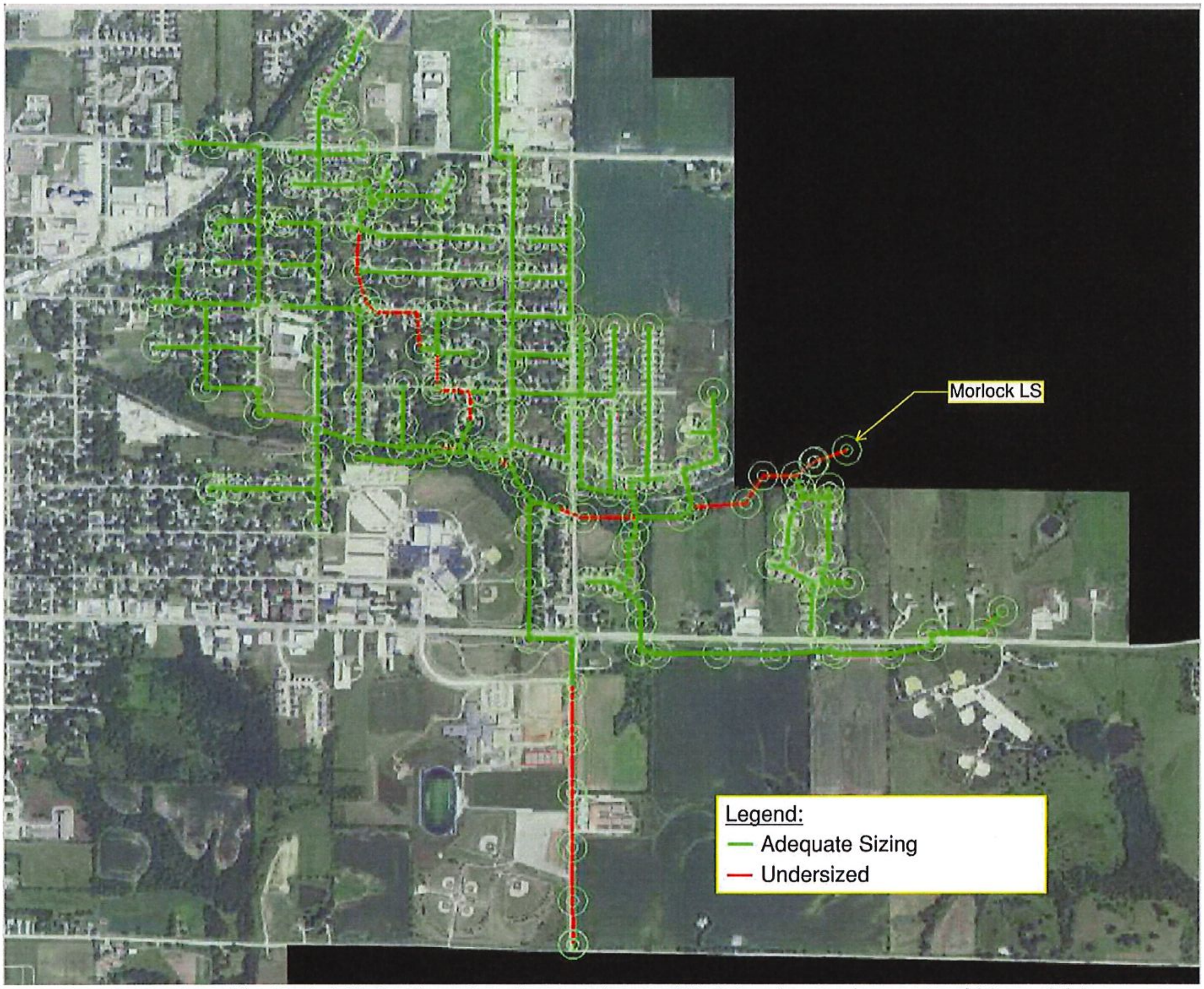


Figure 27: Morlock Lift Station Catchment Area with 25-yr Improvements, 100-yr, 24-hr Storm

Figure 28: South Plant Lift Station Catchment Area, 100-yr, 24-hr Storm

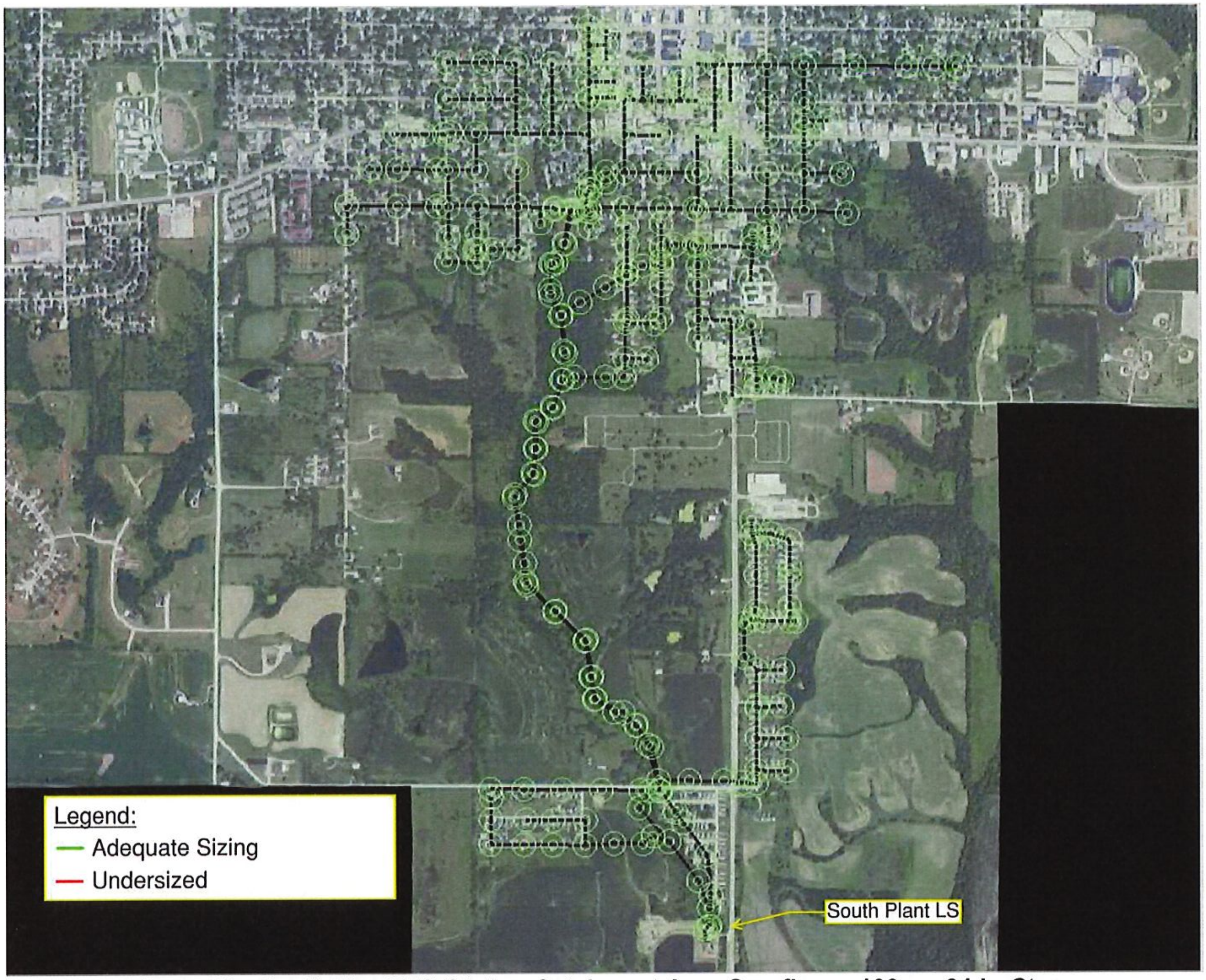


Figure 29: South Plant Lift Station Catchment Area Overflows, 100-yr, 24-hr Storm

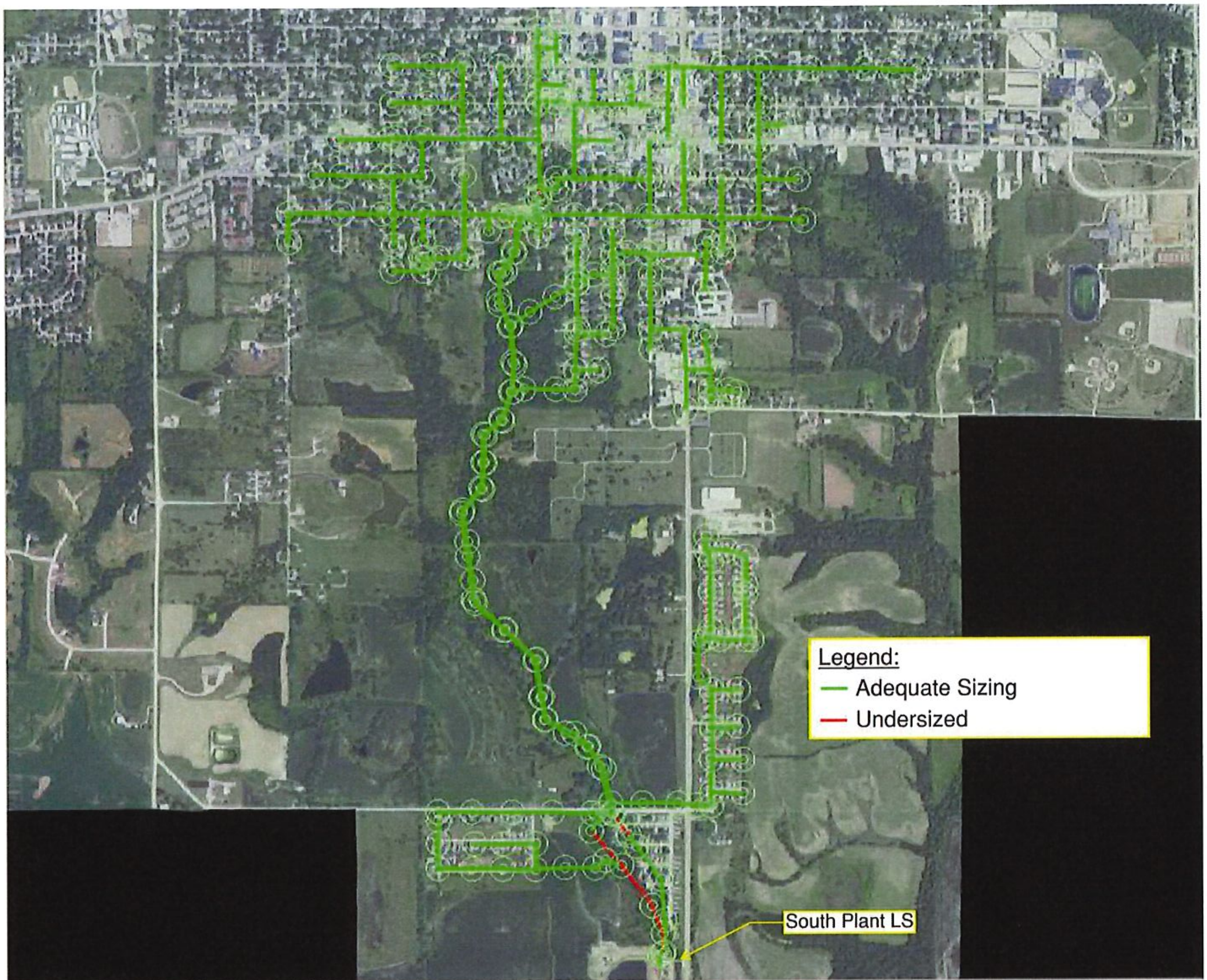


Figure 30: South Plant Lift Station Catchment Area with 25-yr Improvements, 100-yr, 24-hr Storm

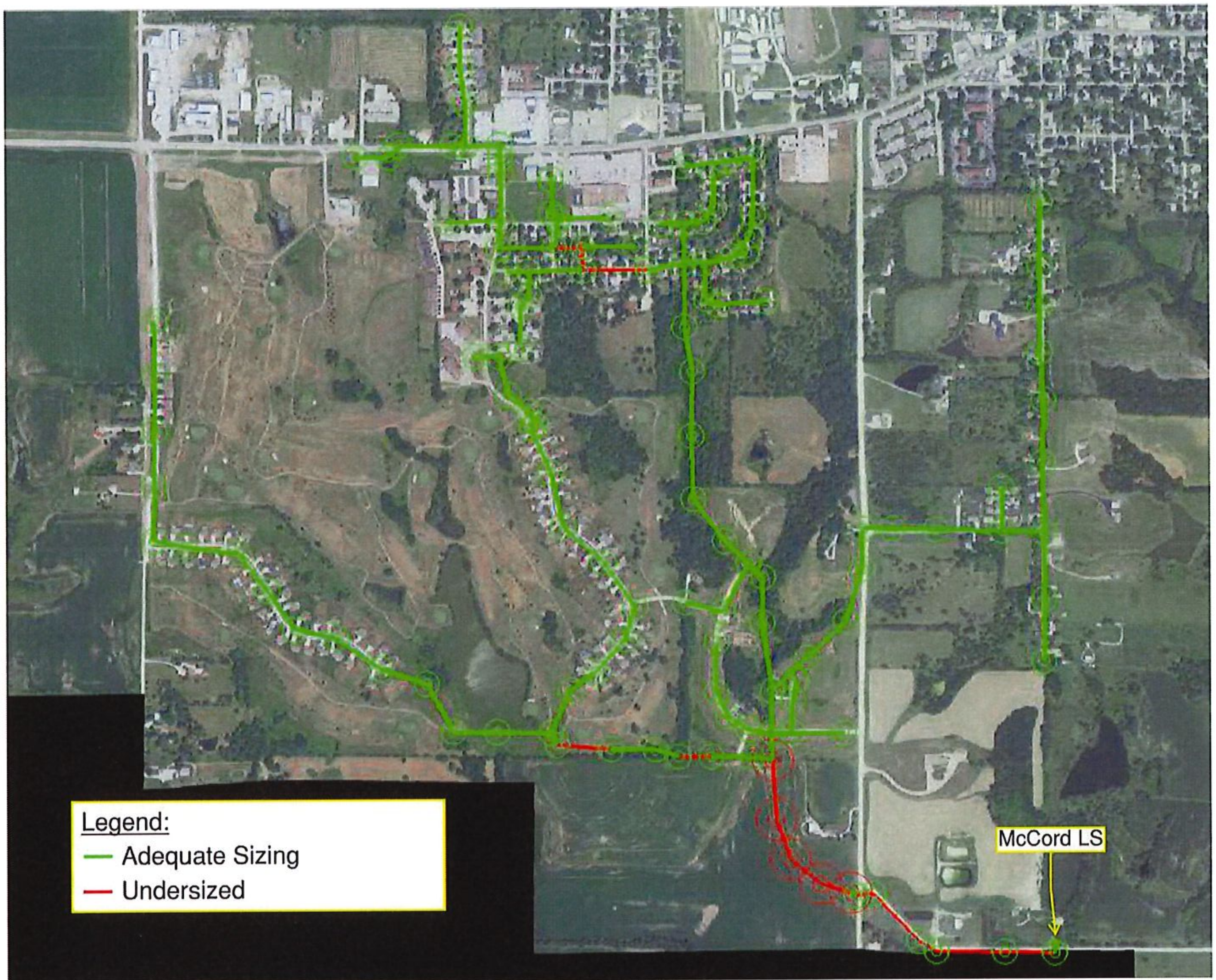


Figure 31: McCord Lift Station Catchment Area, 100-yr, 24-hr Storm

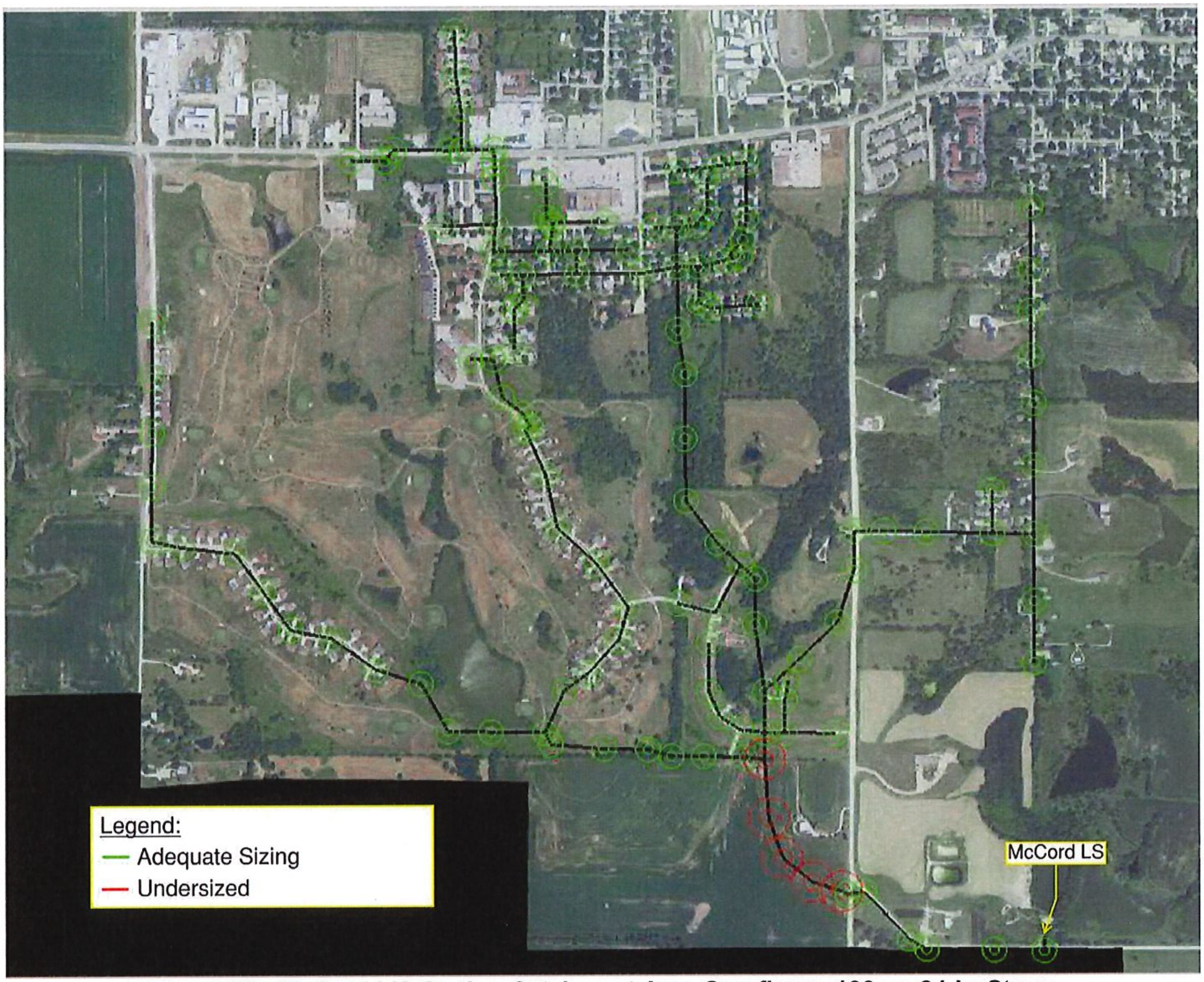


Figure 32: McCord Lift Station Catchment Area Overflows, 100-yr, 24-hr Storm

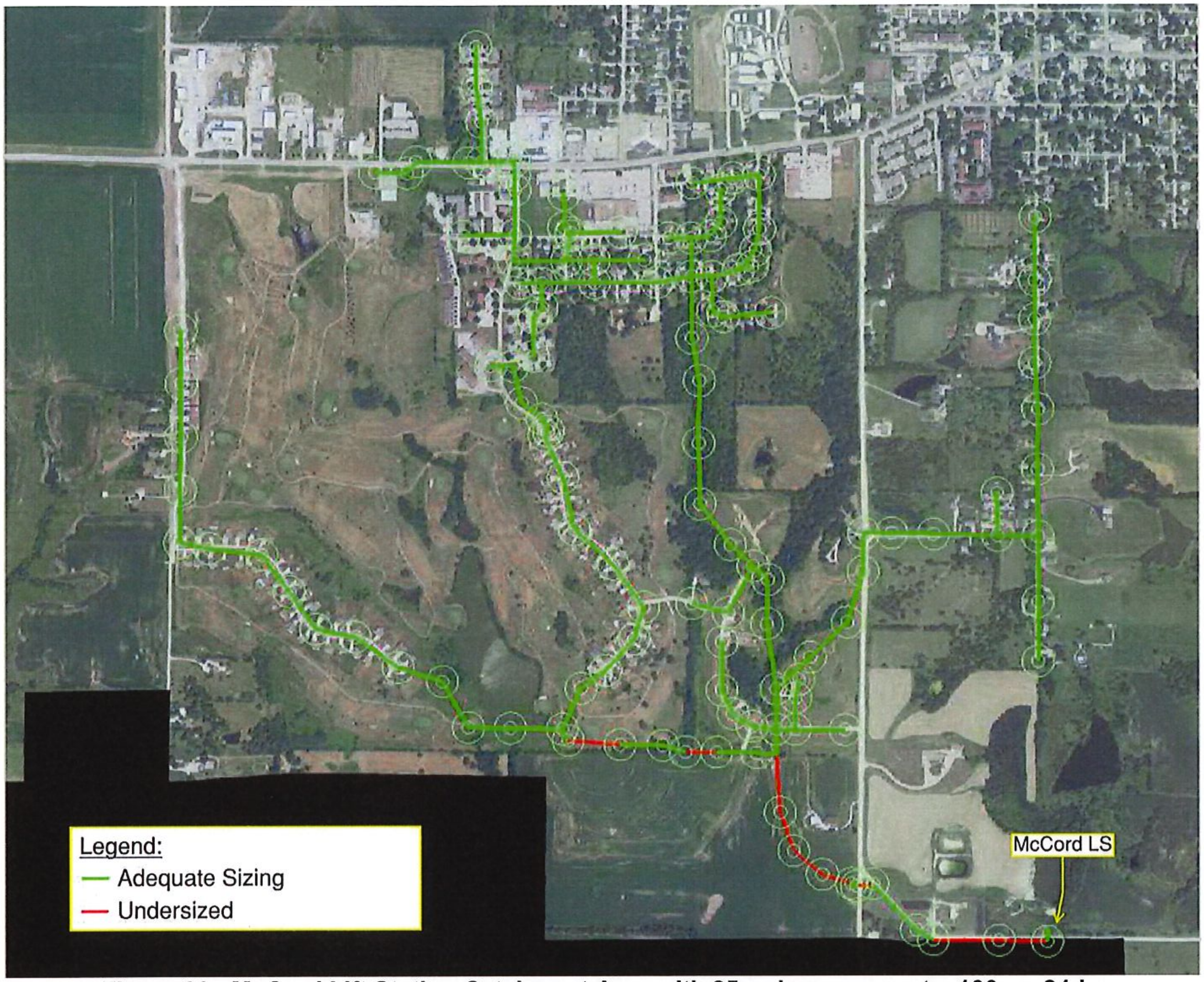


Figure 33: McCord Lift Station Catchment Area with 25-yr Improvements, 100-yr, 24-hr Storm

Figure 34: Plainview Lift Station Catchment Area, 100-yr, 24-hr Storm

Figure 35: Plainview Lift Station Catchment Area Overflows, 100-yr, 24-hr Storm

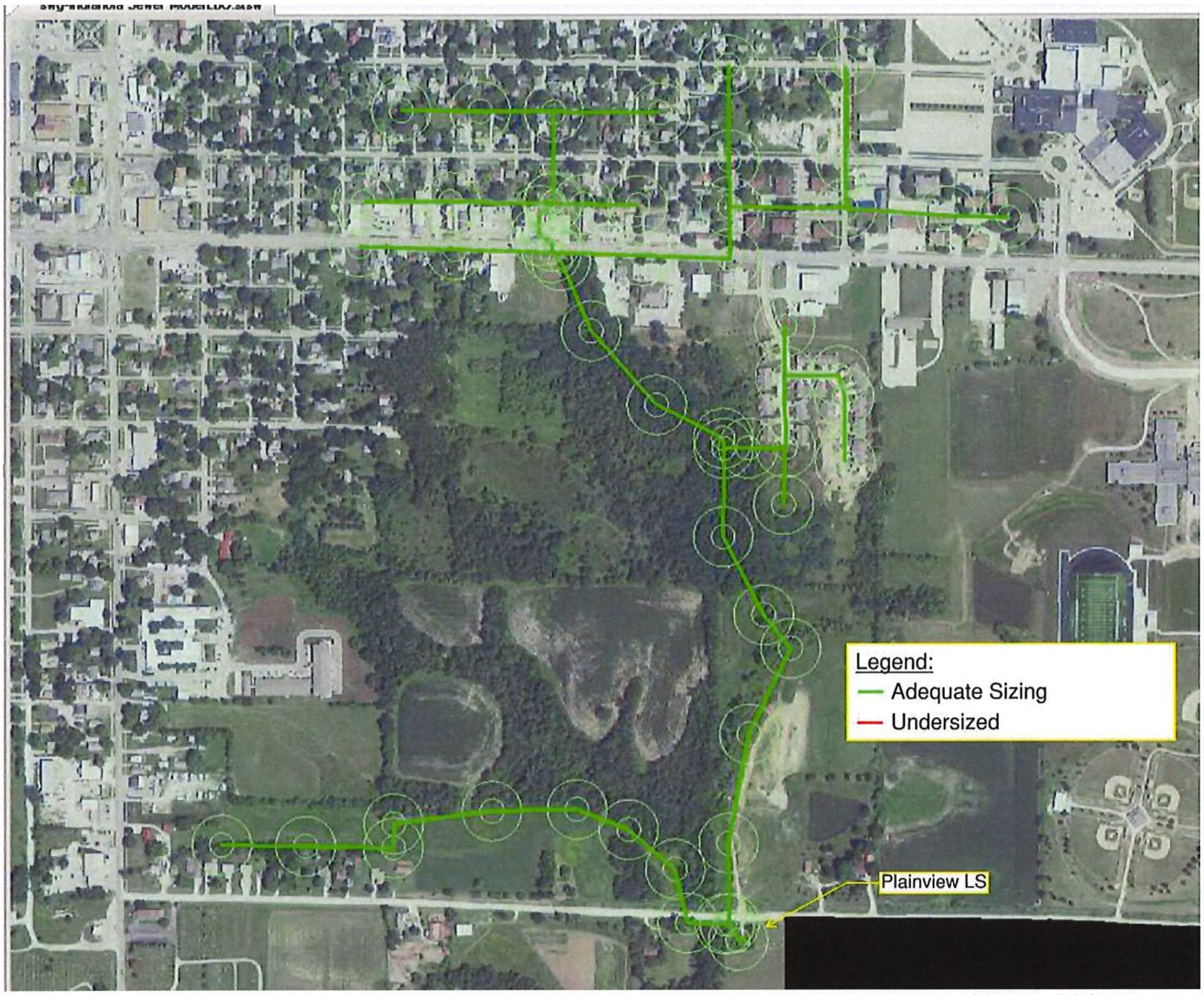


Figure 36: Plainview Lift Station Catchment Area with 25-yr Improvements, 100-yr, 24-hr Storm

As is shown in the first and second figures associated with each catchment area above, multiple pipes and manholes were found to be undersized to handle the 100-yr, 24-hr design storm. As a result, sewer surcharges and basement back-ups are expected throughout the existing system. As is shown in the third figure associated with each catchment area above, major surcharges within the system can be handled if the 25-yr, 24-hr improvement recommendations are completed. The 25-yr, 24-hr improvement recommendations will also help reduce, but not eliminate, basement back-up effects on property owners during a 100-yr, 24-hr storm event.

VIII. CONCLUSION AND RECOMMENDATION

Based on the information available, the model appears to be calibrated correctly to the existing system. Further calibration is recommended in the future to ensure accurate model results. This can easily be completed with additional flow data, including multiple substantial rainfall events. Also, the current model uses rainfall data from monitoring stations in nearby towns. To increase accuracy of the model even further, rainfall monitors should be installed in multiple locations around the City. This ensures the accuracy of rainfall data which is crucial to correct model calibration. To fully calibrate the model, flow monitoring should be done throughout the system to pinpoint areas contributing excessive amounts of I&I. The current model distributes I&I relatively evenly over each catchment area due to lack location information. In reality, certain sections of piping likely contribute a substantial amount of I&I in comparison to others. These sections will likely result in surcharging manholes and backups in locations not identified in this report.

In general, the large amount of inflow into the system is creating the most influential problems. The peaking factor of the wastewater is causing the collection system to be hydraulically overloaded. The most cost effective way to reduce inflow is smoke testing and home inspections. This will allow the City to identify and reduce the number of clear water connections which directly connect to the sanitary system. Another location for high inflow potential is leaking manholes. There are a number of brick manholes in the system that could be contributing to the inflow. These manholes could be lined or replaced to assist in the reduction of inflow as well as infiltration.

Typically, the next step after inflow has been addressed will be to determine the locations of greatest infiltration. This can either be completed using flow monitoring or televising. Flow monitoring is often better because televising is only a snapshot in time and planning televising to coincide with a rainfall event is problematic. Flow monitoring can be set up to measure flows at various points in the sewer system to help identify and isolate areas with high inflow and infiltration. Flows are measured continually over a period of time and can be correlated directly with rainfall events. Once problem lines are determined, the pipes could be lined or replaced. Typically longer or deeper runs are more cost effective to line than to replace.

The system model indicates that during high rain events sewers in many of the catchment areas will start to surcharge and cause backups. These issues can generally be solved by either increase the size of the collection system or decreasing the demand on the sewer system. Typically, eliminating inflow from the system is a more cost effective alternative then increasing the size of piping and utility structures and is the generally the first choice of action. Based on the model results, a relatively small reduction in inflow would allow the system to better accommodate a 100-year storm event with relatively few backups and/or overflowing manholes.

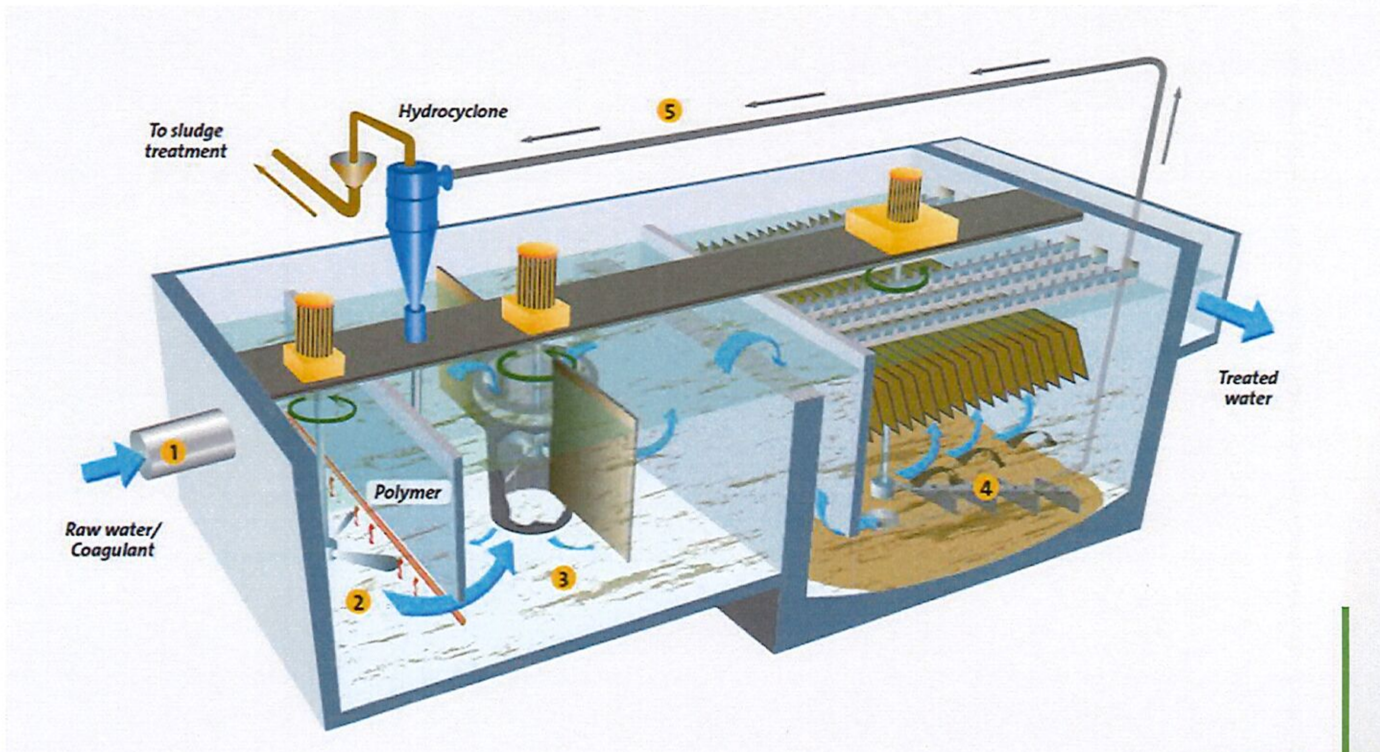
Appendix G - Wet Weather Side Stream Treatment Technologies

Overview of Wet Weather Treatment

HRGreen

- Chemical/physical treatment of flows above secondary treatment capacity
- High level removals - 60% BOD, 90% TSS, 30% TKN, 92% TP
- Blended effluent meets permit under all loading conditions (secondary treatment standards, ammonia, Total N, Total P, disinfection)
- Wet weather side-stream treatment is fraction of cost of expanding secondary treatment
- Protects secondary treatment from upsets during wet weather events

Overview of Wet Weather Treatment



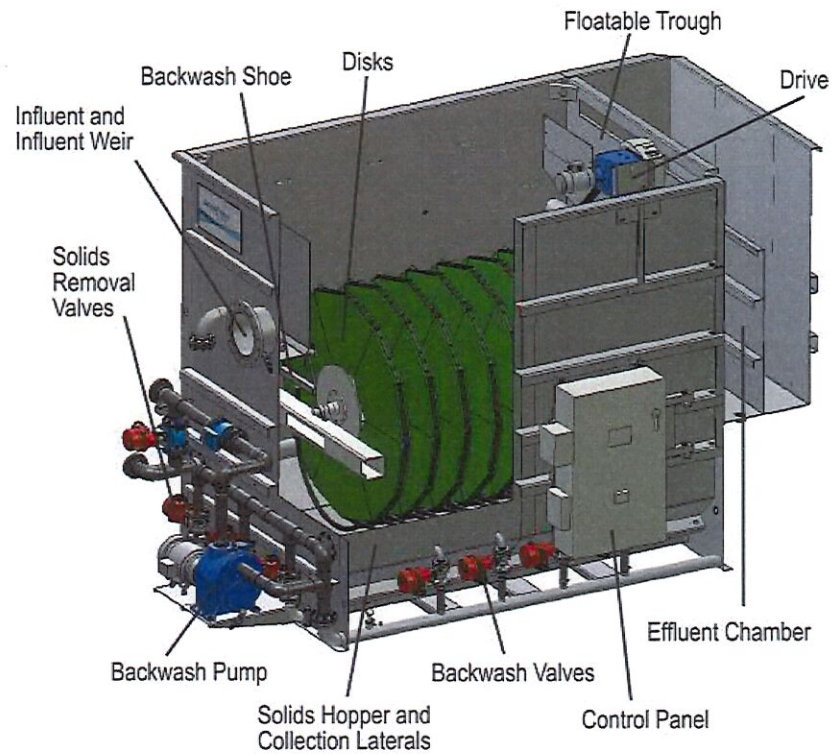
Overview of Wet Weather Treatment

HRGreen

	BOD	TSS	TKN	Phosphorus
Unit/Manufacturer	Removal	Removal	Removal	Removal
Actiflo[1]	36-62%	74-92%	25-30%	92-96%
DensaDeg[1]	37-63%	81-90%	28-40%	88-95%
AquaAerobics - Aqua Prime[2]	60%	90%	40%	92%

1 From USEPA Wastewater Technology Fact Sheet - Ballasted Flocculation - Pilot Results 2003

2 From vendor data



VEOLIA

ACTIFLO® Process

For Wet Weather and Wastewater Treatment

WATER TECHNOLOGIES

ACTIFLO®
Microsand Ballasted Clarification Process

Actiflo is a high rate, compact process developed by Veolia Water Technologies. The process operates with microsand which enhances floc formation and acts as a ballast to aid in rapid settlement of coagulated material.

The Actiflo process can be used at various stages of wastewater treatment including: enhanced primary treatment, wet weather clarification, high rate secondary clarification and final polishing for the removal of solids, phosphorus and metals.

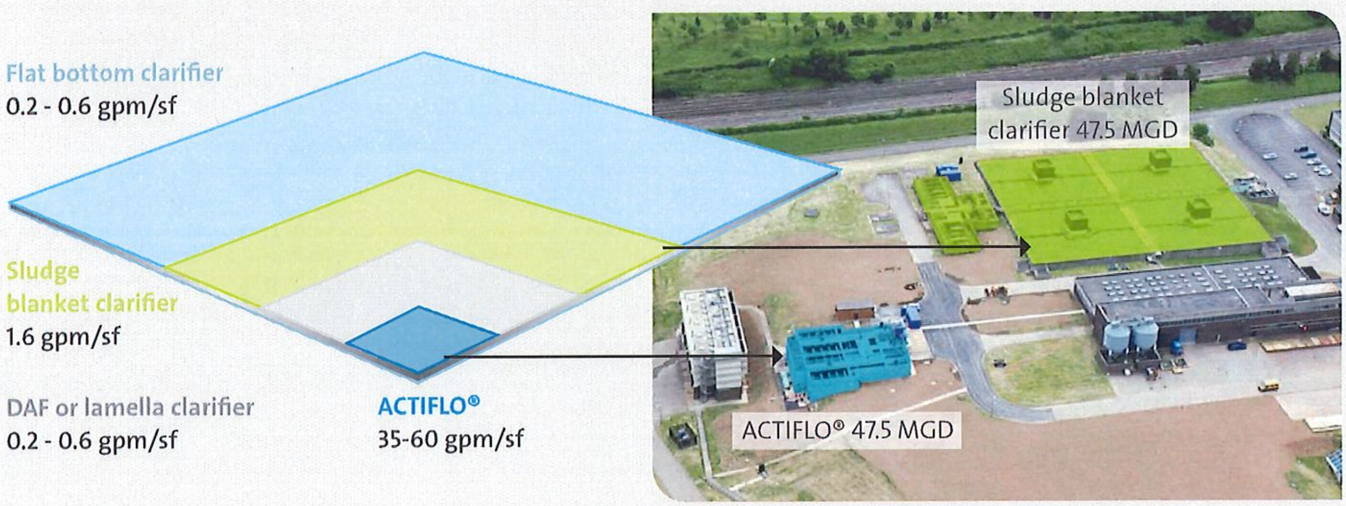
Typical ACTIFLO Performance

Application	loading Rates gpm/sf	Phosphorus (mg/l)	sBOD (mg/l)	BOD_5 (mg/l)	TSS (mg/l)	UV Transmittiance (%)
Wet Weather	60	0.5 - 1.5	10 - 20	< 30	< 20	50 - 70
*Bio ACTIFLO™	45	0.1 - 1.0	1 - 10	< 20	< 15	60 - 70
Secondary	20	0.5 - 1.5	1 - 10	< 10	< 10	65 - 75
Tertiary	45	as low as 0.05	< 10	< 10	≤ 5	75 - 90

*Pathogen removal, post disinfection, is equivalent to or exceeds that of a conventional activated sludge plant

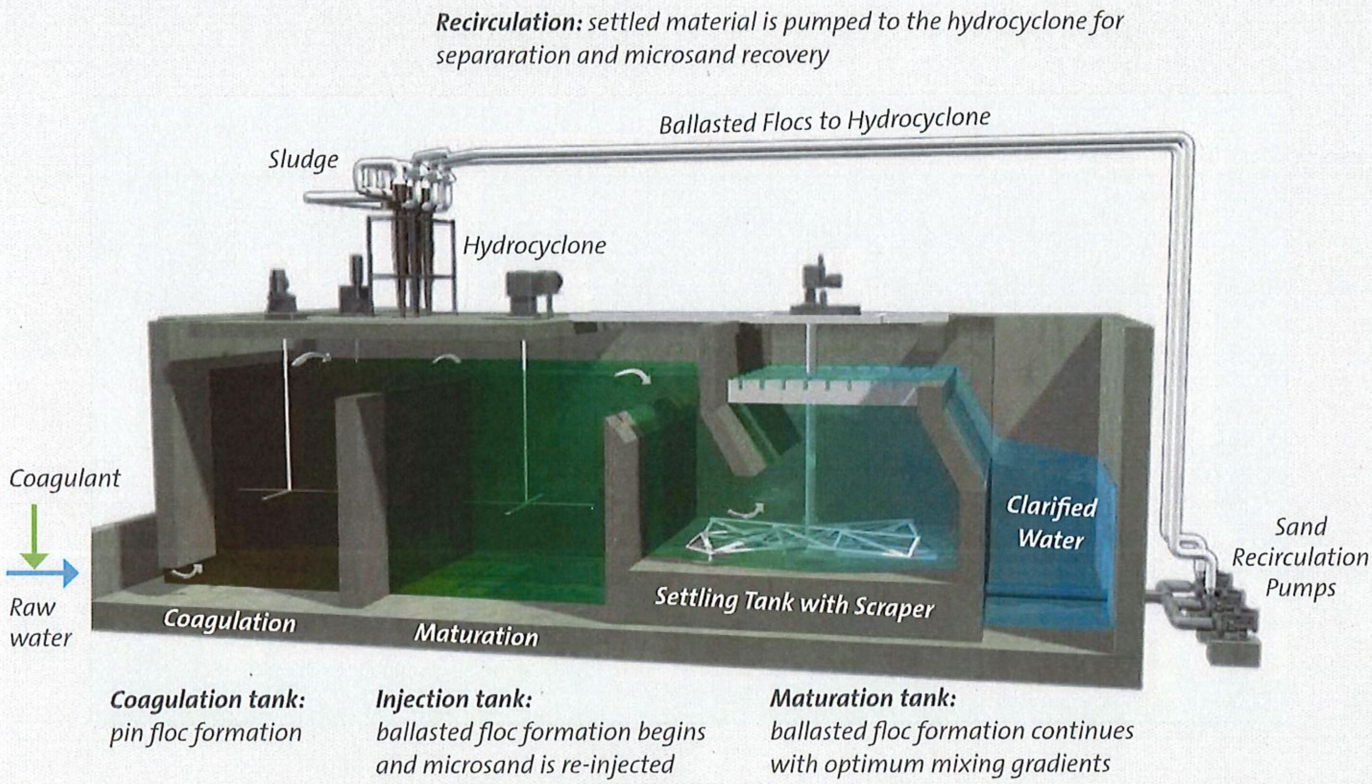
ACTIFLO Compactness Displaying Its True Potential

The microsand ballasted flocs display unique settling characteristics, which allow for clarifier designs with very high overflow rates and short retention times. These designs result in footprints that are 5 times smaller lamella clarifiers or dissolved air flotation (DAF) and up to 20 times smaller than conventional clarification systems.



*Surface water treatment reference

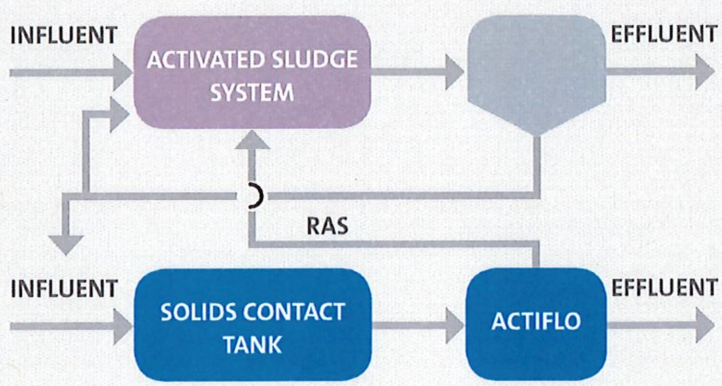
CSO/SSO Parallel Treatment with ACTIFLO



During peak wet weather flow conditions, many plants need to divert a portion of the total plant flow around their biological treatment process. To achieve high levels of TSS and particulate BOD removal of these diverted excess flows, the Actiflo process can be installed at the treatment plant or at a satellite facility within the collection system. The Actiflo process can be fully automated and the process train(s) can sit idle for extended periods of time and still be fully operational within 15 minutes of start-up.

CSO/SSO Treatment BIOACTIFLO™

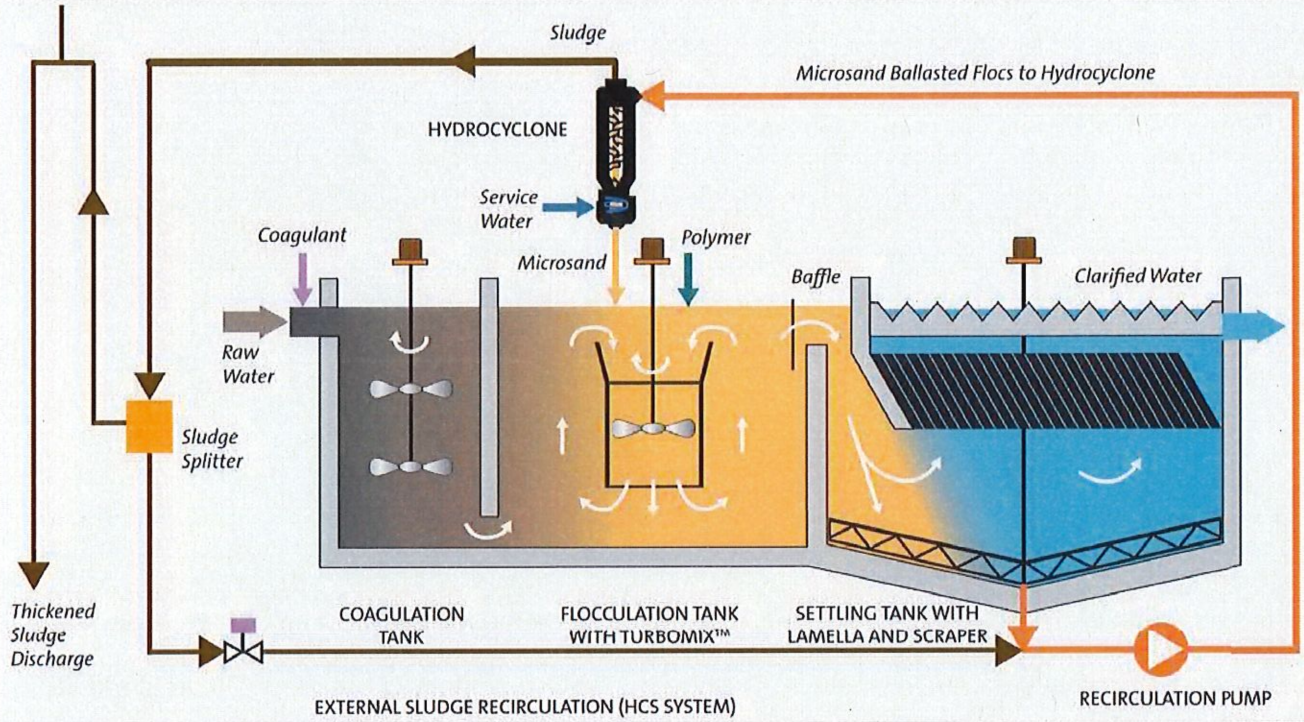
If flow diversion is not an option or the TSS and particulate BOD removal with Actiflo alone are not enough, a biological solids contact tank can be incorporated into the treatment flow path to improve soluble BOD removal through the system.



Return activated sludge (RAS) from the existing clarifiers is combined with the excess flows into a solids contact tank. A targeted mixed liquor suspended solids (MLSS) concentration is maintained in the contact tank to facilitate rapid uptake of soluble biological oxygen demand (BOD) via contact stabilization. Clarification with ACTIFLO follows, producing exceptional TSS and total BOD removal rates that allow for efficient disinfection.

Tertiary Treatment with ACTIFLO

With tighter discharge limits being imposed on wastewater treatment plants the need for a cost effective, flexible process has evolved. Over the years, the Actiflo process has proven its effectiveness in meeting extremely low phosphorus, metals and TSS limits.



Process Benefits

- Small process footprint; suited for restricted spaces and existing basin retrofits
- Low system headloss, incorporates into most existing hydraulic profiles
- Reduced civil engineering costs
- High degree of operational flexibility
- Minimum equipment to maintain, all easily accessible

For tertiary treatment applications, the Actiflo process offers:

- Ability to treat a wide range of influent phosphorus levels to extremely low limits
- Flexibility to meet future limits (phosphorus, metals) without modifying the process train
- The same tertiary treatment trains can also be used to treat wet weather flows
- Treatment of flows with high solids concentration without impacting effluent quality (solids washout from secondary clarifiers during peak flow)
- Reduction in sludge volume by incorporating a HCS system

AQUA-AEROBIC SYSTEMS, INC.
A Metawater Company

AquaPrime™

Cloth Media Filter

A Solution for Primary Treatment and Wet Weather Applications

AquaPrime™ Cloth Media Filter
Featuring OptiFiber® Pile Cloth Media

The AquaPrime cloth media filtration system is designed as an economical and efficient solution for the treatment of primary wastewater and wet weather applications. This system utilizes a disk configuration and the exclusive OptiFiber PF-14™ pile cloth filtration media to effectively filter high solids waste streams without the use of chemicals. This system is ideal for primary wastewater treatment and wet weather applications due to its proven removal efficiencies and high quality effluent, even under varying influent conditions.

The AquaPrime system is designed to handle a wide range of flows in a fraction of space compared to conventional primary clarifiers. The system's high solids removal in comparison to conventional treatment provides energy and operational savings within the wastewater treatment plant due to reduced loads to the secondary process or more solids for anaerobic digestion (energy harvesting).

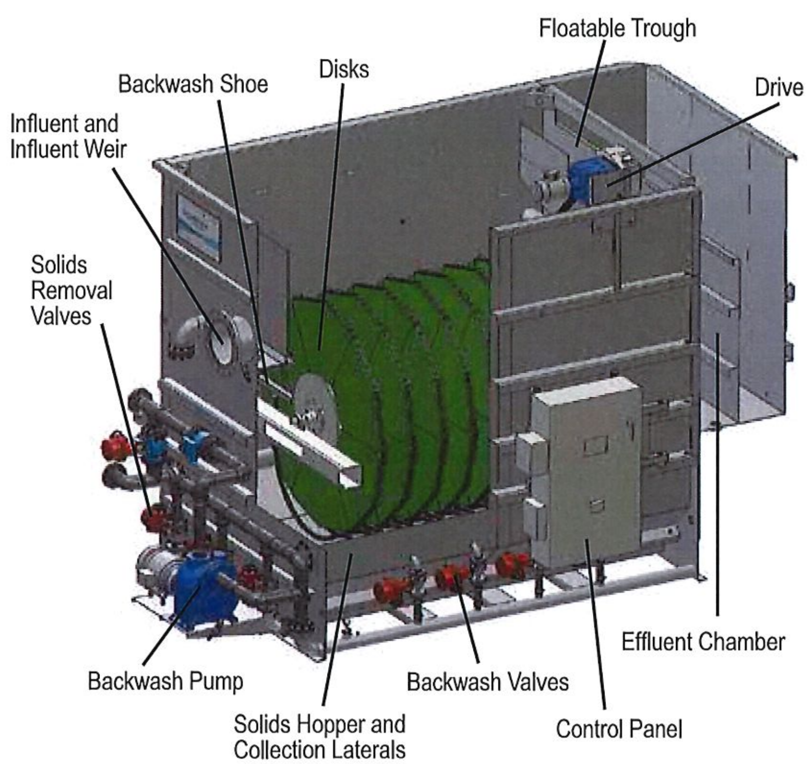
Features and Advantages

- Vertically oriented cloth media disks reduce required footprint
- Each disk is lightweight, with removable segments for ease of maintenance
- Effective backwash system that fluidizes cloth fibers to release stored solids
- Specifically designed floatable and solids removal zones
- Available in several configurations
- Fully automatic PLC control with color touchscreen HMI
- Reduced energy costs in the secondary process due to a reduction in organic loading
- Can be configured for dual use application for tertiary and wet weather operation
- Simple start-up with unattended operation for remote locations
- More solids for increased gas production in anaerobic digesters for primary applications

Applications

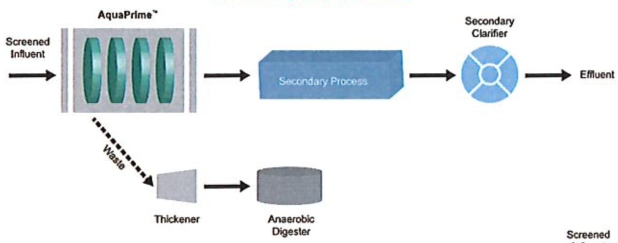
- Primary Filtration
- Primary Effluent Filtration
- Stormwater
- Sanitary Sewer Overflow (SSO)
- Combined Sewer Overflow (CSO)
- High Solids Applications (Municipal and Industrial)

An AquaPrime™ system operating at a municipality for primary treatment.

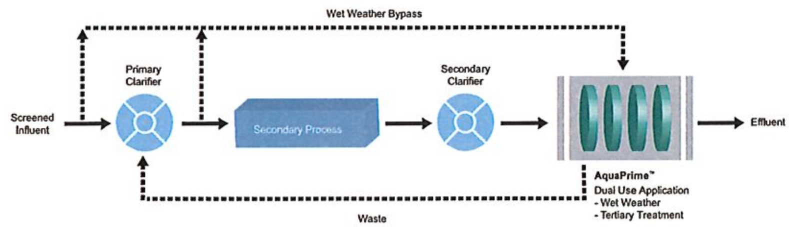


Typical Locations For AquaPrime™ Treatment

Primary Filtration



Wet Weather



Modes of Operation

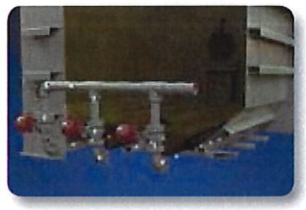
The AquaPrime cloth media filtration system operates on four (4) modes of operation: FILTRATION, BACKWASH, SOLIDS WASTING and FLOATABLE WASTING. For graphical representation, the AquaPrime modes of operation are described below:

Filtration Mode:

- Influent wastewater/wet weather flow enters the filter by gravity
- Stationary cloth media disks are completely submerged
- Solids deposit on the outside of the cloth media forming a mat as filtrate flows through the media
- Tank liquid level rises as headloss builds due to the collection of solids
- Filtrate is collected in the hollow center tube and discharged over an effluent weir
- Heavier solids settle to the specifically designed hoppered tank bottom

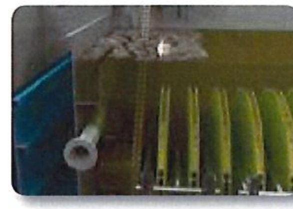
Backwash Mode:

- Solids are backwashed at a predetermined liquid level or time
- Backwash shoes directly contact the cloth media and solids are removed by vacuum pressure using a backwash pump
- Disks rotate slowly and two disks are backwashed at a time (unless a single disk is utilized)
- Filtration is not interrupted
- Backwash water is directed to waste handling facilities (thickening, digester, etc.)



Solids Wasting Mode

- Heavier solids in the collection hopper are removed on an intermittent basis
- Backwash/Solids Pump provides suction to the solids collection manifold for wasting of settled solids
- Solids are pumped back to the waste handling facilities (thickening, digesters, etc.)



Floatable Wasting Mode

- Floatable scum is allowed to collect on the water surface
- After a preset number of backwashes, the water level is allowed to rise above the preset high level
- As the water level increases, floating scum is removed by flowing over the scum removal weir
- Scum wasting water is directed to the plant's waste handling facility

A "Green" Advantage Product
Lower Energy • Small Carbon Footprint

Appendix H - Store and Treat vs. Wet Weather Side Stream Treatment

MEMO

To:	Iowa DNR
From:	Joe Frankl, P.E. - HR Green
Subject:	Indianola Wastewater Facility Planning – Store & Treat Vs. Wet Weather Side Stream Treatment
Project No.	40150016
Date:	April 2018

Background

Accommodating wide variations in flow rates and organic mass loadings is one of the most difficult challenges in operation of a wastewater treatment plant (WWTP). HR Green completed an analysis comparing two different strategies for handling wet weather peak flows for the City of Indianola as discussed in this memo. "Store and Treat" is the practice of shaving off the peak flows above the WWTP capacity and diverting the excess flow to equalization then bringing that flow back for treatment through the WWTP as the peak flows subside. This practice for treatment of peak flows has been used for ages in Iowa.

An alternative practice now gaining some attention is Wet Weather Side Stream Treatment of flows above the WWTP's secondary treatment capacity and then blending the flow from the side stream with the secondary treatment effluent. Depending on the nature of the peak flows to the WWTP, this alternative may be best suited for the community.

Peak Flows

See Table 1 for Indianola's current and future design flows. As illustrated in the table, Indianola currently experiences a peaking factor (ratio of Peak Hourly Wet Weather [PHWW] flow to Average Dry Weather flow [ADW]) of 8.76 and 6.27 for their current and future design flows, respectively.

Table 1 – Current and Future Flows

Parameter	**Current**	**Future**
Flow (MGD)		
ADW	1.56	2.30
Daily Ave	2.02	2.91
AWW	5.17	5.91
MWW	8.36	9.10
PHWW	13.67	14.41

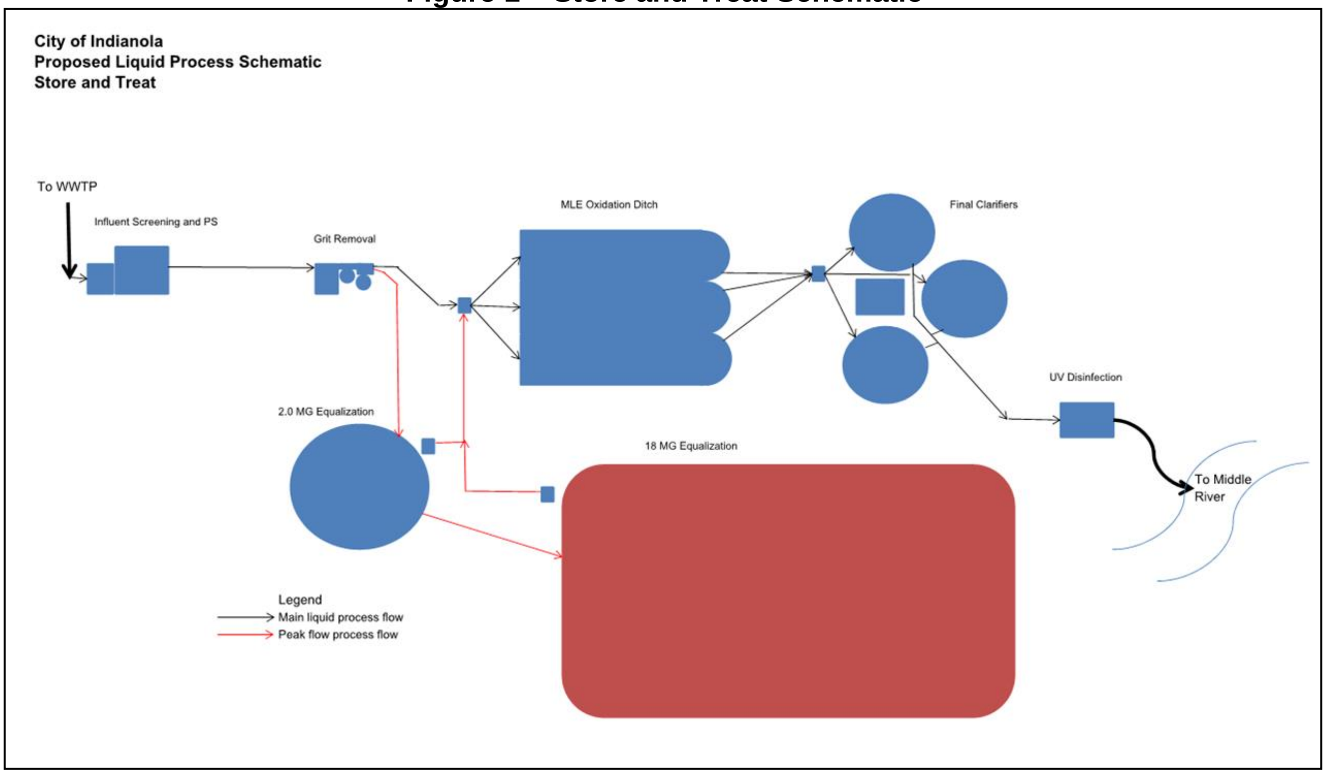
In sizing the WWTP capacity and therefore equalization volume or side stream treatment capacity, the degree of treatment required and resulting feasible treatment process schemes establish the cost economy available by using store and treat or side stream treatment methods. Generally, WWTP's in Iowa are sized such that the maximum hydraulic capacity of the plant should be equal to or greater than Average Wet Weather (AWW) flows. The AWW design flow for Indianola is 5.91 mgd; therefore, the WWTP should be sized greater than or equal to 5.91 mgd. All flows above 5.91 mgd will be stored and treated or shaved off to the wet weather side stream treatment process.

Peak wastewater flows are generally very dilute in strength in nature which can cause problems with plants designed for biological nutrient removal. Indianola has experienced very dilute wastewater strength during wet periods. Due to the historical dilute wastewater in Indianola and the ratio of carbon to nitrogen in the raw influent, supplemental carbon will likely need to be fed for biological nutrient removal to work effectively.

<u>Store and Treat</u>

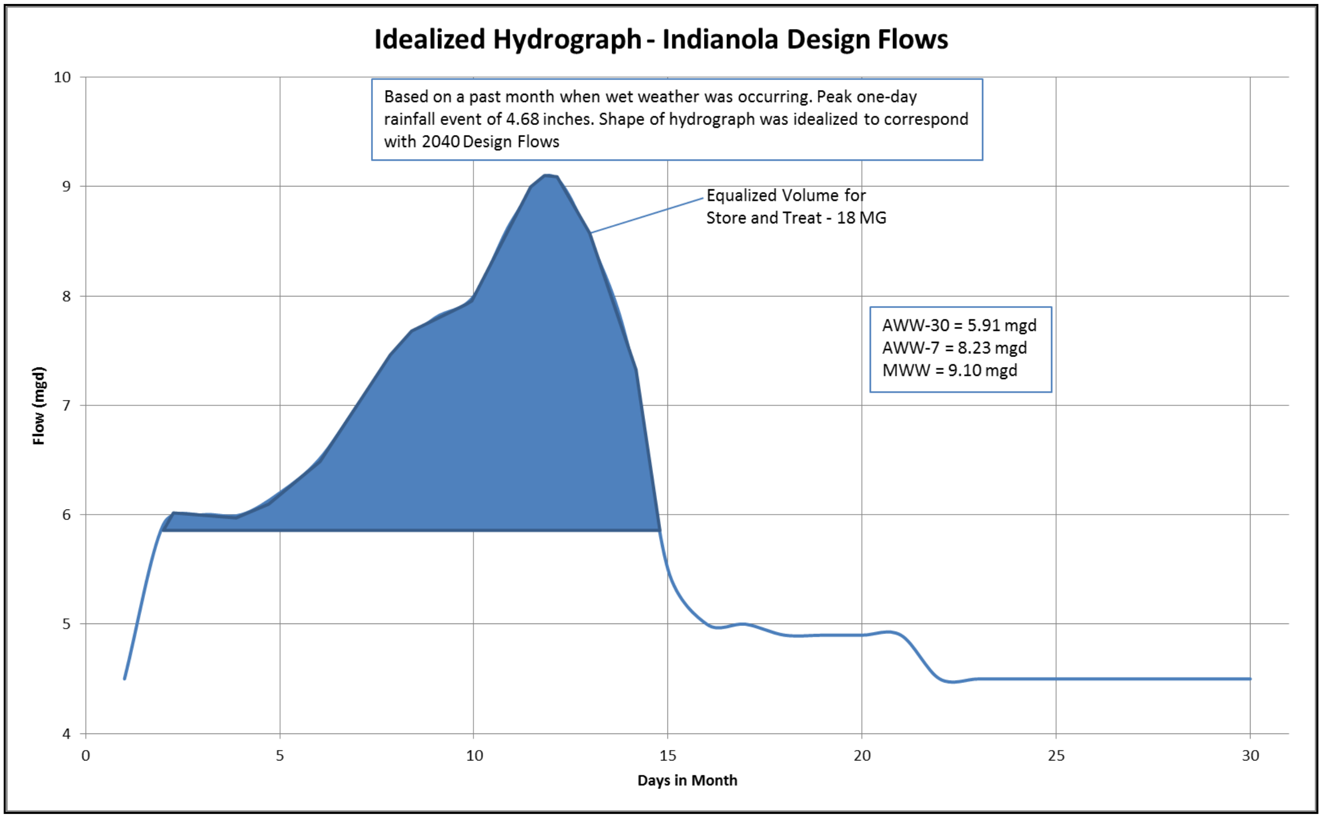
Store and Treat or Flow Equalization is a method used to overcome the operational problems caused by flowrate variations, to improve the performance of the downstream processes, and to reduce the size and cost of downstream treatment facilities. Store and Treat is a means to reduce the magnitude of peak flow events and to spread the loading to the WWTP over a period of time. However, Store and Treat does not lessen the volume of water that will need to be treated. Below is a schematic of Store and Treat and how it would be configured at Indianola's WWTP. Locating the Equalization Basin downstream of preliminary treatment will lessen the operational difficulties associated.

Figure 1 – Store and Treat Schematic



The Equalization Basin is sized based on attenuating flows above the AWW. See Figure 2 below for an illustration of the Equalization Basin sizing. Figure 2 is an Idealized Hydrograph of the Indianola Design Flows over a 30-day period. The minimum volume for an Equalization Basin to attenuate the Design Flows would be 18 million gallons. During final design it would be likely that the actual volume of the Equalization Basin would be increased over 18 million gallons to account for contingency if there are any unforeseen changes in wet weather flow patterns.

Figure 2 – Store and Treat Hydrograph



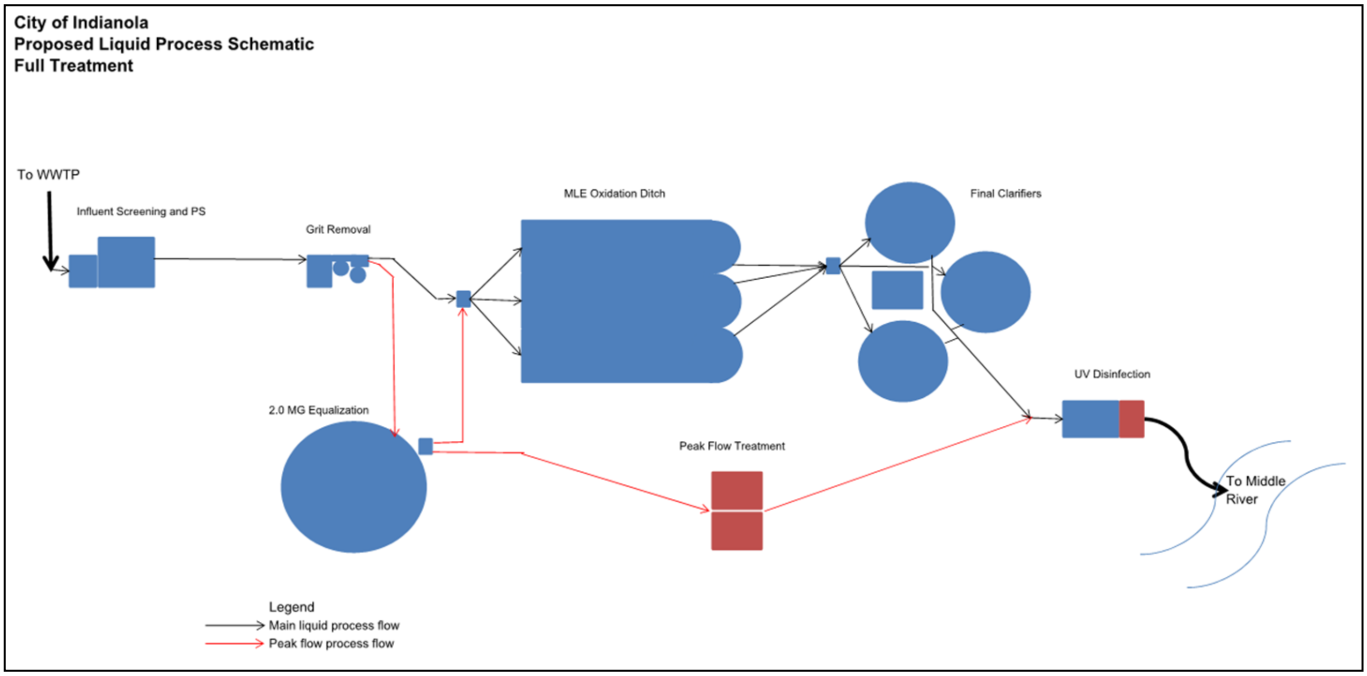
The Equalization Basin will be an earthen basin. This cell will be designed based on a maximum water depth of ten (10) feet with two (2) feet of freeboard. Flow to the Equalization Basin will be overflow from the Daily Equalization Basin.

<u>Wet Weather Side Stream Treatment</u>

Wet Weather Side Stream Treatment (sometimes referred to as Peak Flow Treatment) is a new approach available to EPA Region 7 wastewater facilities to treat peak flows under extreme weather conditions. A guidance document entitled “Key Principles and Consideration Factors for Incorporation on Non-Biological Peak Flow Processing Approaches in Iowa Wastewater Facilities” has been developed for IDNR review. A copy of this guidance document is included in Appendix A of the Facility Plan.

The Wet Weather Side Stream Treatment (WWSST) is sized differently than the Store and Treat alternative since it is a flow through treatment and not storage attenuation. The flow through treatment of the WWSST plus the WWTP’s capacity need to be rated to handle the PHWW flow of 14.41 mgd. A nominal treatment capacity of 10 mgd with two trains at 5 mgd each will be provided by the WWSST such that the combined capacity with the WWTP is approximately 15.91 mgd. Having two trains of 5 mgd each will provide additional flexibility and redundancy at MWW flow. See Figure 3 for a schematic of WWSST. The UV Disinfection system would have to be upsized for this option as effluent from the WWSST units goes directly into the UV disinfection process.

Figure 3 – Wet Weather Side Stream Treatment Schematic



WWSST technologies are developing at a fast rate as the pressure to eliminate SSOs from peak flow events occurs. Particularly in EPA Region 7 states where WWSST may be considered as an acceptable alternative for peak flows. Generally the technologies are physical treatment focusing on removing suspended solids to produce a low cBOD and TSS effluent. A coagulant is frequently added where removing phosphorus is required. A final selection of WWSST technology will be completed during final design. Details of these two wet weather side stream treatment technologies are included in Appendix G. Many of the WWSST solutions can be installed in dual modes: wet weather side stream treatment mode and tertiary treatment mode. Although it is not envisioned that Indianola would need to use WWSST in tertiary mode for permit now and in the near future, it could be beneficial if there was ever a plant upset or unanticipated future effluent limit.

<u>Comparison</u>

Biowin modeling was used to analyze the treatment associated with both of these alternatives and the expected effluent removals performance data. The effluent goals of the Biowin model were Secondary Treatment Standards (cBOD, TSS, pH), Water Quality Based (Middle River Receiving Stream: NH3-N, DO, E. Coli, etc.) and Iowa Nutrient Strategy (TN and TP). See Appendix J for additional information. It should be noted that both the Store and Treat alternative and the WWSST alternative were able to meet the established effluent goals. However, the WWSST was able to provide slightly better effluent quality than Store and Treat. Additionally, the Store and Treat alternative needed significantly more supplemental carbon (approximately 30% more) than the WWSST alternative. This is generally due to the mode of operation difference between the two alternatives. The Store and Treat attenuates and brings flow back to the WWTP at a single fixed flow capacity whereas the WWSST alternative treats the peak flows as flows reach the WWTP. See Figures 4 and 5 which illustrate the percent of capacity utilized for the Store and Treat option and the WWSST option, respectively.

Figure 4 – Store and Treat Capacity Utilized

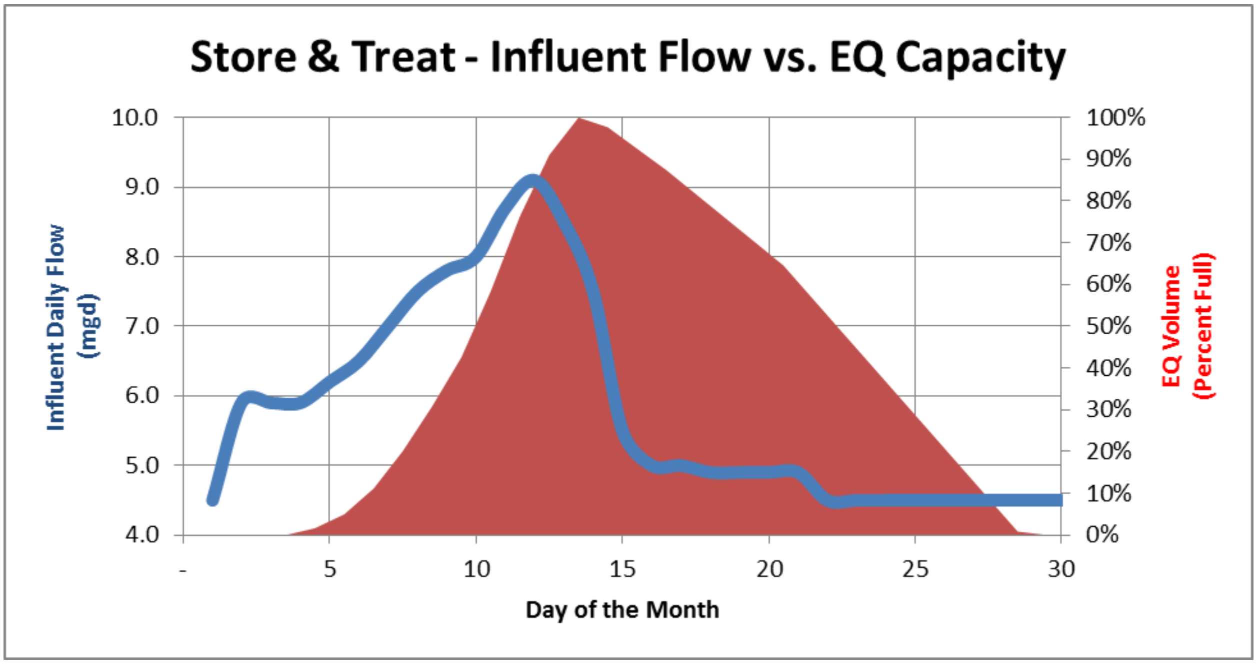
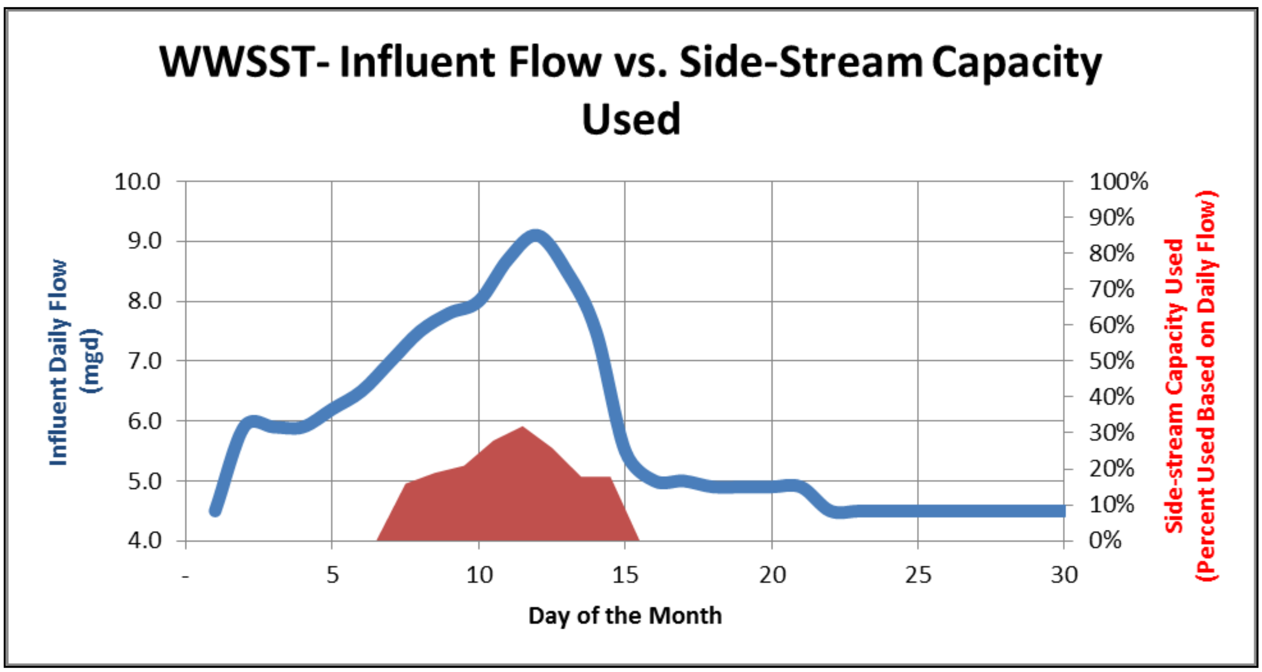


Figure 5 – Wet Weather Side Stream Capacity Utilized



Several advantages and disadvantages associated with the Store and Treat alternative are listed below:

Advantages:
- Equalization Basins reduce the size of downstream unit processes
- All flow goes through WWTP and biological treatment

Disadvantages:
- Relatively large land area is needed
- Odor issues often associated with EQ Basins
- Higher capital costs
- More supplemental carbon is needed
- Higher risk of SSO in extended wet weather period
- Wastewater can lose temperature and grow algae, both which could inhibit downstream treatment

Similarly, several advantages and disadvantages with the Wet Weather Side Stream Treatment alternative are listed below:

Advantages:
- WWSST reduce the size of downstream unit processes
- More stable downstream biological process (by treating and removing peak, dilute flows instead of storing and bringing back through biological WWTP)
- Less supplemental carbon needed)
- Lower capital cost
- Can provide tertiary treatment in addition to wet weather treatment

Disadvantages:
- All flow does not go through biological WWTP
- Higher risk of SSO in short but severe peak event
- Larger UV Disinfection

Conclusion

A cost comparison is shown below in Table 2 for the Store and Treatment option and the WWSST option.

Based on the cost comparison and all of the other reasons as discussed in the Comparison above, Wet Weather Side Stream Treatment was chosen over Store and Treat. This conclusion is supported in the City of Indianola's WWTP Facility Plan.

Table 2 – Cost Comparison

Item	Description	Cost
Reduced Cost from Side Stream Treatment Option		
Peak Flow Treatment Deduct	Package Equipment	-$800,000
	Enclosure Structure	-$400,000
	Chemical Feed Syatems	-$100,000
	Mechanical/ Plumbing	-$800,000
	Electrical/Controls	-$120,000
	subtotal	-$2,220,000
UV Disinfection Deduct		-$300,000
	Total Deducts from Full Treatment	-$2,520,000
Added Cost for Equalization Basin and Return Pump Station		
Earthen Equalization Basin	18.0 MG (w/ clay liner)	$9,000,000
Return PS from Eq Basin	Submersible PS - Structure, Pumps, piping	
	and valves, electrical, controls, access	$180,000
	Total Additional Cost for Equalization	$9,180,000
Net Additional Cost for Store and Treat Option		**$6,660,000**

Appendix I - Oxidation Ditch by WesTech

Indianola WWTF

Iowa

Engineer
HR Green

Furnished by
Tom Dumbaugh
tdumbaugh@westech-inc.com

Represented by
Cor Sonner
Vessco Inc.
Ames, Iowa
(515) 233-8599
csonner@vessco.com

WESTECH

WesTech Opportunity Number: 1560700
Friday, February 16, 2018

Item A – Two (2) OxyStream™ Biological Nutrient Removal Systems
WesTech Equipment Model Number AES2C3

Process Design

Description	Unit	Dimension/Capacity
Flow (Design)	MGD	5.91
BOD (Influent)	lbs/day	7307
(Effluent)	mg/L	
TSS (Influent)	lbs/day	9351
TKN (Influent)	lbs/day	1103
Ammonia (Effluent)	mg/L	10
TN (Effluent)	mg/L	10
TP (Influent)	lbs/day	217
(Effluent)	mg/L	1
Waste Temp (Min/Max)	°C	10/20
Site Elevation	ft. above sea level	970

Equipment

Description	Type	Quantity
Anaerobic Mixers	Submersible	4
Anoxic Mixers	Submersible	4
Aerators	Landy7	4
Bypass Channel Gate	Manual	2
Adjustable Effluent Weir	Manual	2
DO Control System	LDO	2 Probes, 1 Controller
VFD	Stand Alone Panel	4
PLC-Based Control	HMI Interface	1

Equipment Description (Submersible Mixers)		
Description	**Unit**	**Dimension/Capacity**
Anaerobic Submersible Mixers		
Power Feed	V/Ph/Hz	480/3/60
Motor Power	HP	2.4
Impeller Material	-	304SS
Power/Hoist Cable Length	ft	25
Rail/Crane Material	-	304SS
Hoist Cable Material	-	316SS
Anoxic Submersible Mixers		
Power Feed	V/Ph/Hz	480/3/60
Motor Power	HP	2.4
Impeller Material	-	304SS
Power/Hoist Cable Length	ft	25
Rail/Crane Material	-	304SS
Hoist Cable Material	-	316SS

Equipment Description (Aerators)		
Description	**Unit**	**Dimension/Capacity**
Aerator		
Motor Power	HP	50
Motor Voltage	V/Ph/Hz	480/3/60
Motor Speed	rpm	1800
Motor Frame	-	TEFC, C-Face
Motor B-10 Bearing Life	hours	100,000
Motor Heater	V	120
Reducer Service Factor	-	2.5
Reducer B-10 Bearing Life	hours	100,000
Reducer B-10 Life (Output)	hours	250,000
Reducer Oil Heater	V	120
Impeller Diameter	mm	1900
Impeller Thickness	inches	3/8
Impeller Material	-	A36 Steel
Jackstuds Material	-	A307 ZP
Mounting Bars Material	-	A36 Steel

Equipment Description (Bypass Channel Gate)

Description	Unit	Dimension/Capacity
Bypass Channel Gate		
Manual/Automated	-	Manual
Gate Material	-	A36 Steel
Gate Width/Height/Thickness	ft/ft/in	2.5/12.5/0.25
Handwheel Material	-	Aluminum
Handwheel Diameter	in	20

Equipment Description (Effluent Weir Gate)

Description	Unit	Dimension/Capacity
Effluent Weir Gate		
Manual/Automated	-	Manual
Weir Gate Material	-	Aluminum
Weir Length	ft	10
Vertical Weir Travel	in	30

Equipment Description (DO Control System)

Description	Unit	Dimension/Capacity
DO Probes		
Probe Type	-	LDO
Mounting Configuration	-	Pole Mount
Cable Length	ft	33
Range	mg/L	0 – 20.0
Accuracy	-	± 0.05 ppm below 1 ppm ± 0.1 ppm below 5 ppm ± 0.2 ppm above 5 ppm
DO Controller		
Communication Protocol	-	MODBUS 232/485 Profibus DP
4-20 mA Outputs	-	2
Display	in	1.89 x 2.67

Equipment Description (Variable Frequency Drives)		
Description	**Unit**	**Dimension/Capacity**
Variable Frequency Drives		
Power	HP	60*
Power Feed	V/Ph/Hz	480/3/60
Enclosure Type	-	NEMA 12
Enclosure Cooling	-	6
VFD Rectifier	6/12/18 Pulse	6
dv/dt Filter	Y/N	N

*For heavy-duty application VFD's are recommended one size greater than motor HP

Equipment Description (PLC Control System)		
Description	**Unit**	**Dimension/Capacity**
PLC Control System		
Power Feed	V/Ph/Hz	120/1/60
Enclosure Type	-	NEMA 12
UPS	Y/N	N
HMI Size	Inches	10
HMI Manufacturer	-	Allen Bradley
PLC Manufacturer	-	Allen Bradley
PLC Model	-	1769 CompactLogix

Coatings

All steel items, with the exception of the drive mechanism, will be prepared per SSPC-SP10 and coated with one (1) coats Tnemec N140 epoxy, 3-5 mils each. The drive mechanism will be finished painted in the shop with the manufacturer's recommended paint system.

On-Site Services

WesTech Trips to the Site	
Number of Trips	2
Number of Days	4

Field Service

Included field service is for installation inspection, startup, and operator training. Any additional trips that the customer may request can be purchased at the standard WesTech daily rates plus travel and living expenses.

Spare Parts

Spare Parts	
Low Oil Cutout Switch	1
High Speed Coupling	1

Comments and Clarifications

The proposed system was designed based on the information provided and WesTech's standard equipment. The proposed equipment is backed by a 1 Year warranty.

Items Not Included in WesTech's Base Scope of Supply

- Electrical Wiring
- Conduit
- Piping
- Valves/Fittings
- Lubricating Oil/Grease
- Field Welding
- Field Erection

This proposal has been reviewed and is approved for issue by Cody Maxfield on February 16, 2018.

Item B – Three (3) OxyStream™ Biological Nutrient Removal Systems
WesTech Equipment Model Number AES2C3

Process Design

Description	Unit	Dimension/Capacity
Flow (Design)	MGD	5.91
BOD (Influent)	lbs/day	7307
(Effluent)	mg/L	
TSS (Influent)	lbs/day	9351
TKN (Influent)	lbs/day	1103
Ammonia (Effluent)	mg/L	10
TN (Effluent)	mg/L	10
TP (Influent)	lbs/day	217
(Effluent)	mg/L	1
Waste Temp (Min/Max)	°C	10/20
Site Elevation	ft. above sea level	970

Equipment

Description	Type	Quantity
Anaerobic Mixers	Submersible	6
Anoxic Mixers	Submersible	6
Aerators	Landy7	6
Bypass Channel Gate	Manual	3
Adjustable Effluent Weir	Manual	3
DO Control System	LDO	3 Probes, 2 Controllers
VFD	Stand Alone Panel	6
PLC-Based Control	HMI Interface	1

Equipment Description (Submersible Mixers)

Description	Unit	Dimension/Capacity
Anaerobic Submersible Mixers		
Power Feed	V/Ph/Hz	480/3/60
Motor Power	HP	2.4
Impeller Material	-	304SS
Power/Hoist Cable Length	ft	25
Rail/Crane Material	-	304SS
Hoist Cable Material	-	316SS
Anoxic Submersible Mixers		
Power Feed	V/Ph/Hz	480/3/60
Motor Power	HP	2.4
Impeller Material	-	304SS
Power/Hoist Cable Length	ft	25
Rail/Crane Material	-	304SS
Hoist Cable Material	-	316SS

Equipment Description (Aerators)

Description	Unit	Dimension/Capacity
Aerator		
Motor Power	HP	30
Motor Voltage	V/Ph/Hz	480/3/60
Motor Speed	rpm	1800
Motor Frame	-	TEFC, C-Face
Motor B-10 Bearing Life	hours	100,000
Motor Heater	V	120
Reducer Service Factor	-	2.5
Reducer B-10 Bearing Life	hours	100,000
Reducer B-10 Life (Output)	hours	250,000
Reducer Oil Heater	V	120
Impeller Diameter	mm	1600
Impeller Thickness	inches	3/8
Impeller Material	-	A36 Steel
Jackstuds Material	-	A307 ZP
Mounting Bars Material	-	A36 Steel

Equipment Description (Bypass Channel Gate)

Description	Unit	Dimension/Capacity
Bypass Channel Gate		
Manual/Automated	-	Manual
Gate Material	-	A36 Steel
Gate Width/Height/Thickness	ft/ft/in	2.5/10.5/0.25
Handwheel Material	-	Aluminum
Handwheel Diameter	in	20

Equipment Description (Effluent Weir Gate)

Description	Unit	Dimension/Capacity
Effluent Weir Gate		
Manual/Automated	-	Manual
Weir Gate Material	-	Aluminum
Weir Length	ft	10
Vertical Weir Travel	in	30

Equipment Description (DO Control System)

Description	Unit	Dimension/Capacity
DO Probes		
Probe Type	-	LDO
Mounting Configuration	-	Pole Mount
Cable Length	ft	33
Range	mg/L	0 – 20.0
Accuracy	-	± 0.05 ppm below 1 ppm ± 0.1 ppm below 5 ppm ± 0.2 ppm above 5 ppm
DO Controller		
Communication Protocol	-	MODBUS 232/485 Profibus DP
4-20 mA Outputs	-	2
Display	in	1.89 x 2.67

Equipment Description (Variable Frequency Drives)		
Description	**Unit**	**Dimension/Capacity**
Variable Frequency Drives		
Power	HP	40*
Power Feed	V/Ph/Hz	480/3/60
Enclosure Type	-	NEMA 12
Enclosure Cooling	-	6
VFD Rectifier	6/12/18 Pulse	6
dv/dt Filter	Y/N	N

*For heavy-duty application VFD's are recommended one size greater than motor HP

Equipment Description (PLC Control System)		
Description	**Unit**	**Dimension/Capacity**
PLC Control System		
Power Feed	V/Ph/Hz	120/1/60
Enclosure Type	-	NEMA 12
UPS	Y/N	N
HMI Size	Inches	10
HMI Manufacturer	-	Allen Bradley
PLC Manufacturer	-	Allen Bradley
PLC Model	-	1769 CompactLogix

Coatings

All steel items, with the exception of the drive mechanism, will be prepared per SSPC-SP10 and coated with one (1) coats Tnemec N140 epoxy, 3-5 mils each. The drive mechanism will be finished painted in the shop with the manufacturer's recommended paint system.

On-Site Services

WesTech Trips to the Site	
Number of Trips	2
Number of Days	4

Field Service

Included field service is for installation inspection, startup, and operator training. Any additional trips that the customer may request can be purchased at the standard WesTech daily rates plus travel and living expenses.

Spare Parts

Spare Parts	
Low Oil Cutout Switch	1
High Speed Coupling	1

Comments and Clarifications

The proposed system was designed based on the information provided and WesTech's standard equipment. The proposed equipment is backed by a 1 Year warranty.

Items Not Included in WesTech's Base Scope of Supply

- Electrical Wiring
- Conduit
- Piping
- Valves/Fittings
- Lubricating Oil/Grease
- Field Welding
- Field Erection

This proposal has been reviewed and is approved for issue by Cody Maxfield on February 16, 2018.

Budget Pricing

Proposal Name: Indianola WWTF
Proposal Number: 1560700
Friday, February 16, 2018

1. Bidder's Contact Information

Company Name	WesTech Engineering, Inc.
Contact Name	Tom Dumbaugh
Phone	801.265.1000
Email	tdumbaugh@westech-inc.com
Address: Number/Street	3665 S West Temple
Address: City, State, Zip	Salt Lake City, UT 84115

2. Pricing

Currency	US Dollars

Scope of Supply

A	(2) OxyStream™ Biological Nutrient Removal Systems Model Number AES2C3	$643,000
B	(3) OxyStream™ Biological Nutrient Removal Systems Model Number AES2C3	$734,500
	Taxes (sales, use, VAT, IVA, IGV, duties, import fees, etc.)	Not Included

Prices are for a period not to exceed 30 days from date of proposal.

Field Service

Daily Rate	$960

Prices do not include field service unless noted, but it is available at the daily rate plus expenses. The customer will be charged for a minimum of three days for time at the jobsite. Travel will be billed at the daily rate. Any canceled charges due to the customer's request will be added to the invoice. The greater of visa procurement time or a two week notice is required prior to trip departure date.

3. Payment Terms

Submittals Approved	15%
Release for Fabrication	35%
Net 30 days from Shipment	50%

All payments are net 30 days. Partial shipments are allowed. Other terms per WesTech proforma invoice.

4. Schedule

Submittals, after PO receipt	6 to 8 Weeks
Customer Review Period	2 weeks
Ready to Ship, after Submittal Approval	18 to 20 weeks
Total Weeks from PO to Shipment	**26 to 30 weeks**

Terms & Conditions: This proposal, including all terms and conditions contained herein, shall become part of any resulting contract or purchase order. Changes to any terms and conditions, including but not limited to submittal and shipment days, payment terms, and escalation clause shall be negotiated at order placement, otherwise the proposal terms and conditions contained herein shall apply.

Freight: Prices quoted are **F.O.B. shipping point** with freight allowed to a readily accessible location nearest to jobsite. All claims for damage or loss in shipment shall be initiated by purchaser.

Paint: If your equipment has paint included in the price, please take note to the following. Primer paints are designed to provide only a minimal protection from the time of application (usually for a period not to exceed 30 days). Therefore, it is imperative that the finish coat be applied within 30 days of shipment on all shop primed surfaces. Without the protection of the final coatings, primer degradation may occur after this period, which in turn may require renewed surface preparation and coating. If it is impractical or impossible to coat primed surfaces within the suggested time frame, WesTech strongly recommends the supply of bare metal, with surface preparation and coating performed in the field. All field surface preparation, field paint, touch-up, and repair to shop painted surfaces are not by WesTech.

OxyStream Layout and Concrete Estimate

WesTech

Project Information

Project Name:	Indianola	Project Number:	1560700
Engineer:	-	Completed by:	CTM
Date:	2/22/2018	Checked by:	Preliminary

Design Parameters

Ditch Parameters

# of Ditches	2	
Aerators/Ditch	2	
Depth	12	ft
Channel Width	24	ft
Straight Length	79.60	ft
Channel Freeboard	1.5	ft
Aeration Freeboard	6	ft

Assumptions

Exterior Walls	14	in	thick
Interior Walls	12	in	thick
Deck	12	in	thick
Floor	10	in	thick
Footings	18	in	thick
Footings	60	in	tall

Volume

Anaerobic	0.09225	Mgal (ea)
Anoxic	0.1395	Mgal
Aerobic	1.18	Mgal
Total	1.32	Mgal

Footprint

	Width	Length	
Anaerobic	32.06	32.06	ft
Anoxic	99.00	15.70	ft
Aerobic	99.00	152.60	ft
Total (2 ditches)	201.33	204.68	ft

Concrete Estimate

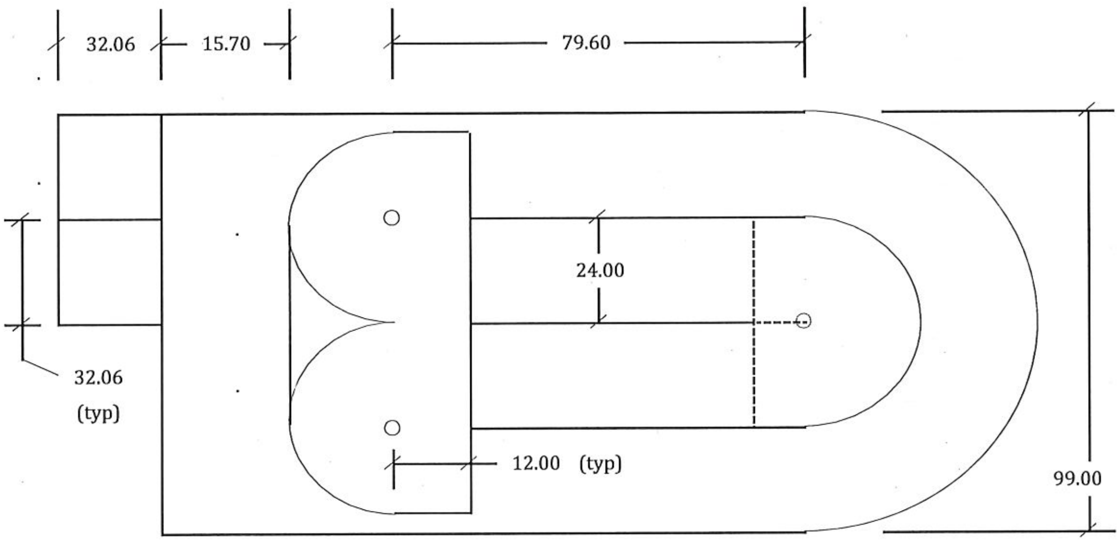
OxyStream BASIN OUTER WALLS	285	cu-yd
OxyStream BASIN INNER WALLS	383	cu-yd
OxyStream BASIN FLOOR	840	cu-yd
OxyStream BASIN FOOTINGS	301	cu-yd
Aerator Deck(s)	227	cu-yd
Total Estimated Concrete	**2036**	**cu-yd**

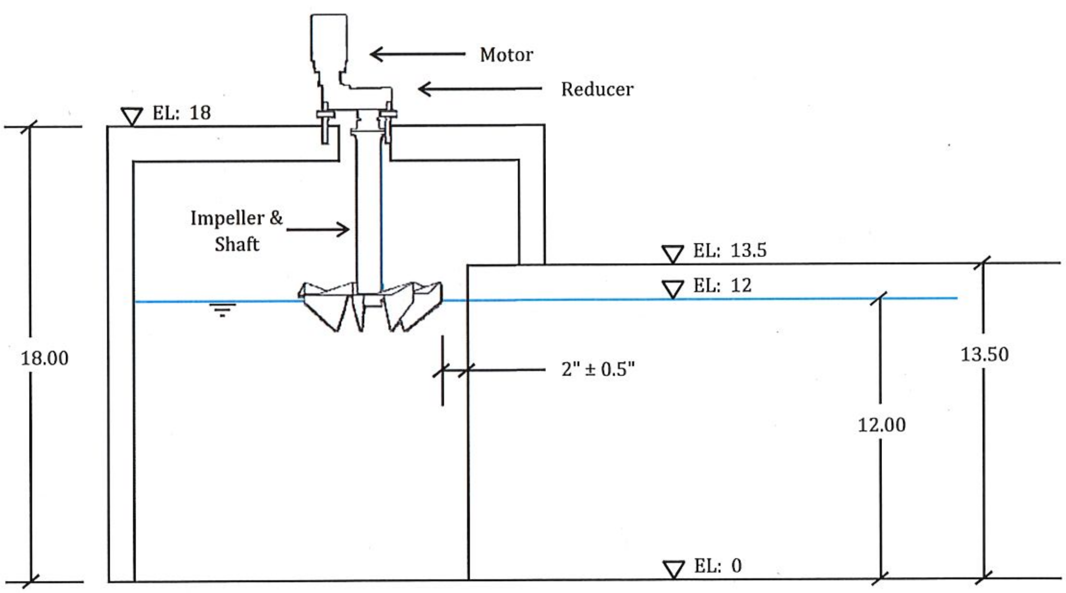
OxyStream Layout and Concrete Estimate

Ditch Layout

Drawing Not To Scale
Dimensions listed are interior dimensions
Dimensions Given in Feet

Shown: 1 of 2





OxyStream Layout and Concrete Estimate

WesTech

Project Information

Project Name:	Indianola	Project Number:	1560700
Engineer:	-	Completed by:	CTM
Date:	2/22/2018	Checked by:	Preliminary

Design Parameters

Ditch Parameters

# of Ditches	3	
Aerators/Ditch	2	
Depth	10	ft
Channel Width	20	ft
Straight Length	83.54	ft
Channel Freeboard	1.5	ft
Aeration Freeboard	6	ft

Assumptions

Exterior Walls	14	in	thick
Interior Walls	12	in	thick
Deck	12	in	thick
Floor	10	in	thick
Footings	18	in	thick
Footings	60	in	tall

Volume

Anaerobic	0.0615	Mgal (ea)
Anoxic	0.093	Mgal
Aerobic	0.79	Mgal
Total	0.88	Mgal

Footprint

	Width	Length	
Anaerobic	28.67	28.67	ft
Anoxic	83.00	14.98	ft
Aerobic	83.00	144.54	ft
Total (3 ditches)	253.33	192.53	ft

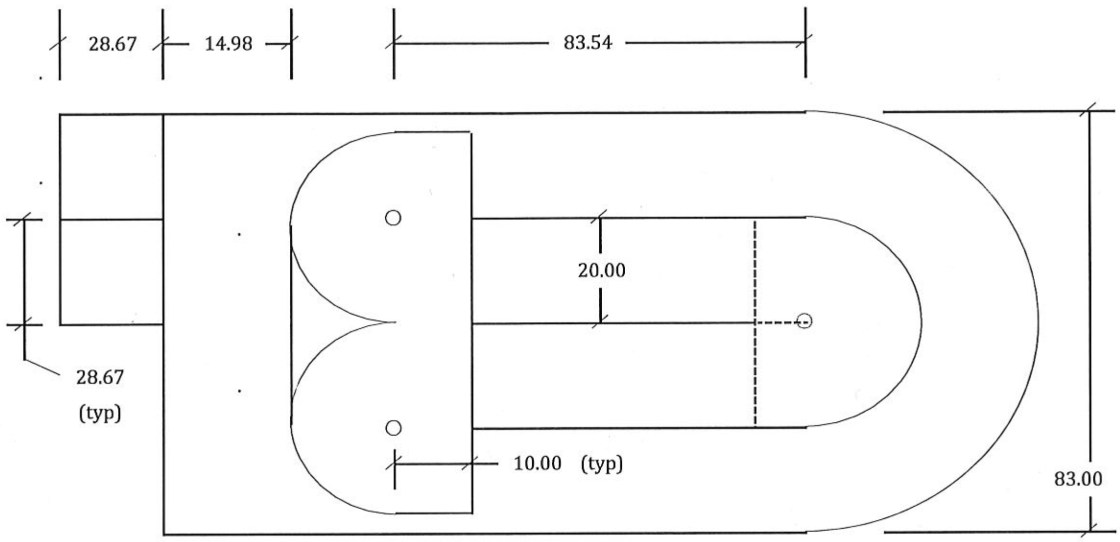
Concrete Estimate

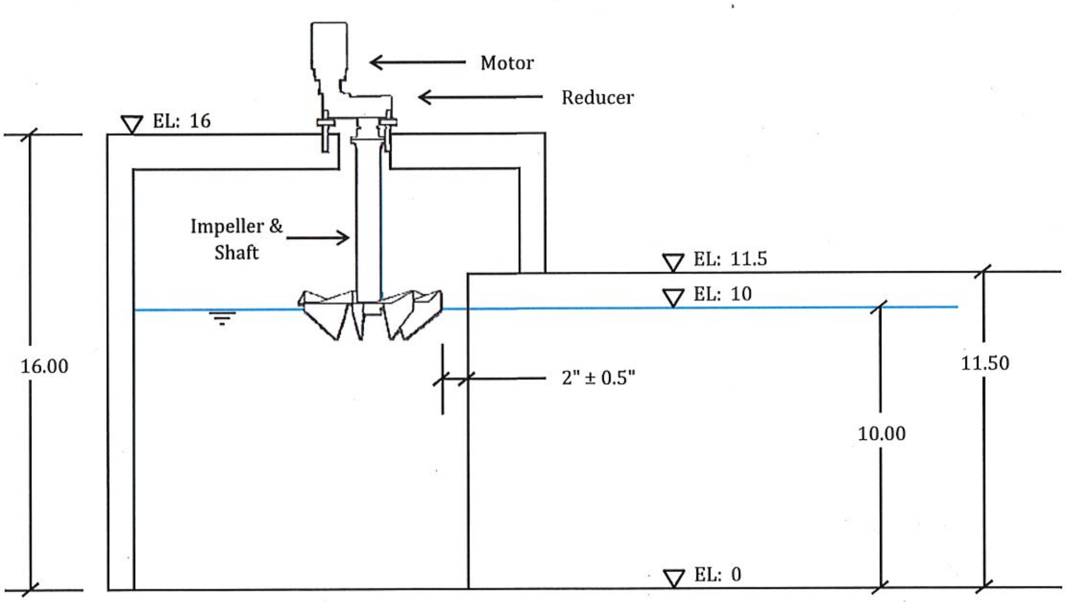
OxyStream BASIN OUTER WALLS	278	cu-yd
OxyStream BASIN INNER WALLS	396	cu-yd
OxyStream BASIN FLOOR	1015	cu-yd
OxyStream BASIN FOOTINGS	344	cu-yd
Aerator Deck(s)	238	cu-yd
Total Estimated Concrete	**2270**	**cu-yd**

Ditch Layout

Drawing Not To Scale
Dimensions listed are interior dimensions
Dimensions Given in Feet

Shown: 1 of 3





Appendix J - Biowin Process Modeling Summary

Biowin Modeling Background

HRGreen

- Purpose of Biowin model is to simulate the performance of a WWTP as a whole and the individual unit processes and evaluate WWTP performance and ability to meet effluent limits with design influent flows and loadings.
- Effluent Goals
  - Secondary Treatment Standards for cBOD, TSS, pH
  - Water quality based goals (per revised WLA to Middle River dated 08/11/17) for NH_3-N, DO, E. Coli and other parameters
  - Iowa Nutrient Strategy for Total Nitrogen and Total Phosphorous

Biowin Modeling Scenarios

HRGreen

- Completed 5 Biowin runs at various WWTP load/flow conditions.
- Completed 2 additional runs comparing store and treat vs. full treatment considering 30-day peak hydrograph and typical wet weather loadings.

Scenario	Description
1	AWW Flow, Max Day Loading
2	2 x AWW Flow, Max Day Loading (includes side-stream treatment)
3	AWW Flow, Ave. Month Loading
4	0.25 x AWW Flow, Ave. Month Loading
5	Daily Ave. Flow, Ave. Month Loading
6	Store & Treat 30-Day Wet Weather
7	Full Treatment 30-Day Wet Weather

Biowin Modeling Results

- Completed 5 Biowin runs at various WWTP load/flow conditions
- Completed 2 additional runs comparing store and treat vs. full treatment
- All runs meet BOD, TSS, NH3, TN,TP
- Model runs identify where supplement carbon is needed

Biowin Modeling Results Scenarios 1-5

HRGreen

Scenario	cBOD5	TSS	NH3-N	TN	TP
1) AWW Flow, Max Day Loading	4.2	14.5	0.3	7.9	0.8
2) 2 x AWW Flow, Max Day Loading (blended)	10.5	12.5	2.4	4.5	0.8
3) AWW Flow, Ave. Month Loading	3.1	13.2	0.3	5.1	0.6
4) 0.25 x AWW Flow, Ave. Month Loading	1.2	2.7	0.2	6.8	0.7
5) Daily Ave. Flow, Ave. Month Loading	1.6	5.8	0.2	6.9	0.7

Biowin Modeling Results Scenarios 6&7

HRGreen

Scenario	cBOD5	TSS	NH3-N	TN	TP
6) Store and Treat – 30 Day	5.1	22.4	0.8	7.7	0.9
7) Full Treatment – 30 Day (blended)	2.3	9.8	0.3	7.5	0.5

Appendix K - Wastewater Treatment Plant Staffing

E. WASTEWATER TREATMENT PLANT STAFFING

The Indianola NWWTF currently has a staff of six employees to manage, operate and maintain the wastewater treatment plant and maintain the City's sanitary sewer collection system including sanitary sewers, seven lift stations and force mains. The six employees include the Wastewater Superintendent. Each of the operations staff completes the laboratory analysis needed for operations and IDNR reporting. The operations staff also is responsible for doing routine and minor maintenance on equipment.

Historically, staffing recommendations for WWTPs has been most frequently estimated by the guidance document "Estimating Staffing for Municipal Wastewater Treatment Facilities" from the U.S. Environmental Protection Agency published in 1973. This document estimates staff hours required by looking at operations and maintenance hours required for each process based on the capacity of the WWTP. For the proposed WWTP improvements the EPA guidance document recommends 11 employees. This does not include the operation and maintenance requirements for the collection system.

Generally, this document is outdated because it doesn't account for reduced manpower for SCADA systems in modern treatment plant operations. Generally the basic automation of a wastewater treatment plant today requires less manual operation.

The recommended WWTP staff for the City of Indianola for the proposed new wastewater treatment plant and collection system maintenance is shown below:

Position	No of Employees
Superintendent	1
Operations staff (includes collections)	5
Maintenance Technician	1
Lab Technician	1
Admin/clerical	0.5
Total	8.5

The proposed increase in employees over the current level is 2.5 employees. A laboratory technician should be added to handle all the compliance testing and to help relieve the duty from the operations staff. A maintenance technician should be added to account for the additional instrument and controls maintenance that will be needed for the operations instruments. A half-time administrative assistant should be provided to help manage the office activities and for clerical duties.

As a comparison to these recommendations, two similar Iowa Grade IV treatment plants about the same size were reviewed to compare the number of employees. The Marshalltown WWTP is a 6.0 mgd AWW plant that has a cBOD capacity of 8,000 lbs/day. Marshalltown has a Superintendent, Assistant Superintendent, office manager, 2.5 laboratory personnel, 4 operators, 2 maintenance electricians, and 2 swing maintenance/operators for a total of 13.5 employees. In addition to the plant this staff maintains 9 sanitary and 2 stormwater lift stations but does not maintain collection systems. Burlington WWTP is another 6.0 mgd

AWW plant in eastern Iowa. Burlington has 8 employees that operate the wastewater treatment plant and maintain the sanitary lift stations. The rest of collections system maintenance is handled separately by Public Works.

Appendix L – Exhibit 9B – Preliminary Review of Facility Plan Checklist

Iowa Department of Natural Resources
Wastewater Engineering Section

Exhibit 9B - Preliminary Review of Facility Plan Checklist

“Facility Plan” means a report certified by a professional engineer licensed to practice in Iowa and prepared in conformance with Chapter 11 of the Iowa Wastewater Facilities Design Standards (IWWFDS). A Facility Plan will not be required for non-funded minor sewer extensions, minor trunk and interceptor sewers, and minor pump stations where comprehensive planning is not completed, necessary or required. Facility planning submittals may be returned if they are deemed incomplete by the Department.

The transmittal letter referenced in Section 11.2.2 of the IWWFDS and a completed Exhibit 9B checklist by the engineer shall be bound with the engineering report. The transmittal letter must:

- Describe fully the scope of the project identified in Design Schedule A.
- Provide a statement on the feasibility of the project.
- Include a statement that this report has been accepted by the client.
- Indicate that the proposed project is in conformance with the long range planning of the area.
- Reference all information and approved planning reports necessary for a review.
- Clearly indicate the purpose of the submittal.

Exhibit 9B is divided into four sections as follows:

- Section 1 – All Projects
- Section 2 – New or Expanded Wastewater Treatment Facility Projects
- Section 3 – Earthen Basin Projects
- Section 4 – SRF Funded Projects

Section 1 must be completed for all projects. Sections 1 and 2 must be completed for projects involving new or expanded wastewater treatment facilities. Sections 1, 2, and 3 must be completed for projects that consist of new or expanded wastewater treatment lagoon facilities. Sections 1 and 3 must be completed for projects involving new or expanded equalization with earthen basins. In addition, complete Section 4 if the project is SRF funded.

Responses of **“Yes”, “No”, “?”, or Not Applicable (“N/A”)** may be used by DNR in completing Exhibit 9B Preliminary Review with explanations given, as appropriate. A “?” mark may be used by DNR staff where additional follow-up, or the consideration of additional information may be warranted before a comment is offered. Every attempt should be made to complete the Exhibit 9B preliminary review checklist using good engineering judgment and as accurately as possible for the benefit of decision makers. If the response is “No” by the engineer for location maps and/or geotechnical report, the transmittal letter must acknowledge that the Facility Plan is incomplete and provide adequate need and justification for the Department to initiate a concept review.

Section 1 – All Projects

1. Yes A work initiation meeting determination has been made. If the meeting was determined to be necessary, the meeting has been held. The scope and milestones for the project have been clearly established.
2. Yes A project location and a recommended alternative have been proposed by the A/E and the conclusion accepted by the Owner in accordance with Step 17, Section 11.2 of the Iowa Wastewater Facilities Design Standards and Design Schedule A.
3. N/A A completed and signed Design Schedule A has been submitted in accordance with Section 11.1 of the Iowa Wastewater Facilities Design Standards.
4. Yes Any proposed variation from the design standards contained in Chapter 567 IAC 64 is identified by the Engineer in accordance with Design Schedule A with justification provided in accordance with DNR rules.
5. Yes A complete and achievable project implementation schedule has been provided identifying all project milestones in accordance with Section 11.2.5.3(k) of the Design Standards.
6. Yes The Appendix (Technical Information and Design Criteria) is provided per Design Standard 11.2.11.
7. Yes The facility plan is signed and certified by a professional engineer licensed in the State of Iowa.

Section 1 – Comment Box:

Section 2 – New or Expanded Wastewater Treatment Plant Projects

8. Yes The Owner has filed an application for a new or amended NPDES permit as needed for the improvements described in the Facility Plan and has notified the review engineer of this submission.
9. Yes Completed Design Schedules F and G have been submitted in accordance with Section 11.1 of the Iowa Wastewater Facilities Design Standards.
10. Yes The location maps are prepared by the Engineer in accordance with Design Schedule F to the recommended scale and provide all requested detail to conduct a site survey investigation for the proposed new or expanded wastewater treatment facilities.
11. Yes All hydraulic and organic design loadings in Design Schedule G and the Facility Plan are consistent with the preliminary design loadings concurred by the Department.
12. Yes The project has conformed to the Waste Load Allocation (WLA) determination and the effluent limits which have been established by the DNR through Steps 9, 11, 12, 13, and 14 of the wastewater construction permitting procedures.
13. Yes Where anti-degradation requirements apply, the recommended alternative is consistent with the anti-degradation alternatives analysis approved by the Department.
14. Yes New Process Evaluation - all required engineering data and design basis formulated from the data for New Process Evaluation has been approved by the Department under Section 14.4.3 and was prepared by a licensed professional engineer other than the one employed by the manufacturer or patent holder.

Section 2 – Comment Box:

Information needed from Design Schedule F was submitted with the WWTP Siting Study.

Section 3 – Projects with Earthen Basins (Lagoon and Equalization Basins)

15. --- A completed geotechnical investigation engineering report is provided as a supplement to the engineer's report.

Section 3 – Comment Box:

Section 4 – State Revolving Fund (SRF) Loan Projects

16. --- The proposed project is a fundable category (Refer to Subrule 567 IAC 90.2) for receipt of a CWSRF loan.
17. --- The Intended Use Plan application (Exhibit 8) is enclosed with the Facility Plan and the "Assurance with Respect to Real Property Acquisition" form.
18. --- The Property/Easement Acquisition Schedule is included.
19. --- The Owner has submitted all required Exhibit 5 information to the Environmental Review Services Coordinator in order to initiate the SRF environmental review.

Section 4 – Comment Box:

SRF funding will be applied for in the future once final design of the WWTP in underway.

This page for DNR Use Only

DNR Decisions:

--- 9B Complete

--- Concept Review Request

Conclusions by DNR: