FACILITY PLAN

for

WASTEWATER TREATMENT FACILITIES

INDIANOLA, IOWA

April 2016

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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.

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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 peak flow 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 each of the potential receiving streams 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 Modified Ludzack-Ettinger (MLE) followed by chemical phosphorus removal. The MLE process will remove BOD, ammonia and total nitrogen ahead of the phosphorus removal. The three secondary treatment alternatives evaluated were: conventional 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 peak flow 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.

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 D.

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 Peak Flow 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. Peak flow 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

3.1. EXISTING SERVICE AREA

The Indianola North Wastewater Treatment Facility (WWTF) treats wastewater from the incorporated areas of town. The North plant was originally constructed in the 1970's. Prior to that time the City's wastewater was treated at the South Wastewater Treatment Plant. The two WWTFs operated in parallel for a number of years until just the North plant remained in operation. The wastewater flow comes primarily from residential and commercial flows with no permitted industrial contribution. The North plant is located on the northwest part of town and discharges plant effluent to Cavitt Creek. Cavitt Creek discharges flow into the Middle River within a couple miles from the North plant discharge point.

The City's collection system includes approximately 83 miles of sanitary sewer in the city and ten lift stations. The North WWTF includes a 27 million gallon earthen equalization basin and the South plant lift station includes approximately 13 million gallons of equalization. The collection system has historically received significant Inflow and Infiltration (I/I) to the sanitary system. The City recently implemented a four phased program to reduce I/I in the collection system. This program has recently been completed and the City has noticed a reduction in sanitary sewer flows reaching the North WWTF.

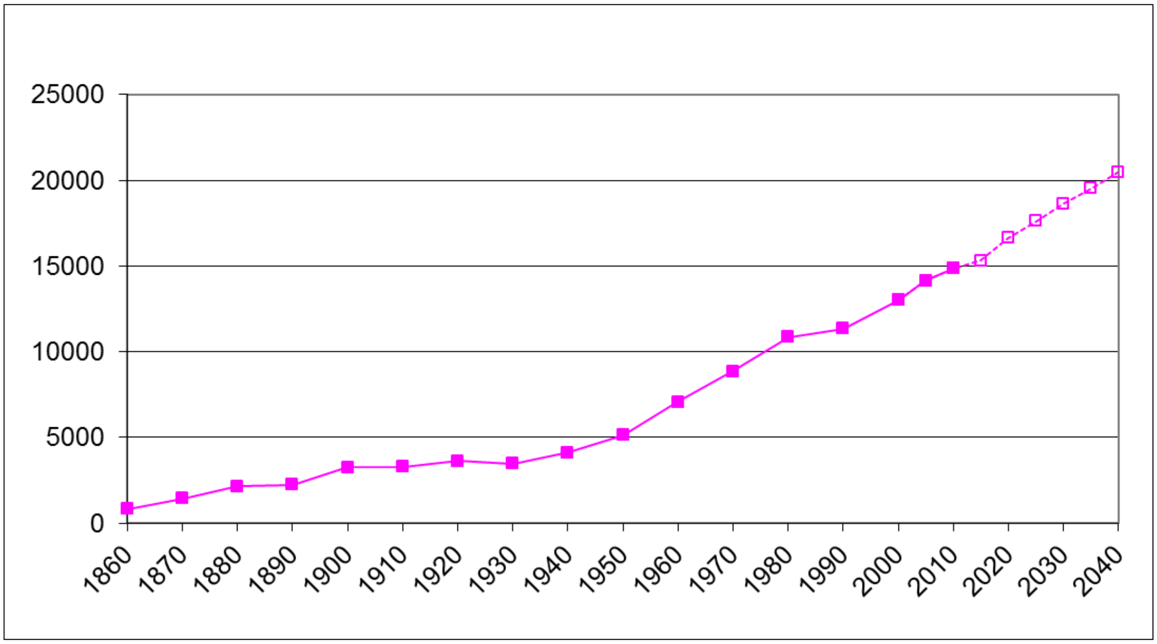
3.2. POPULATION

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.

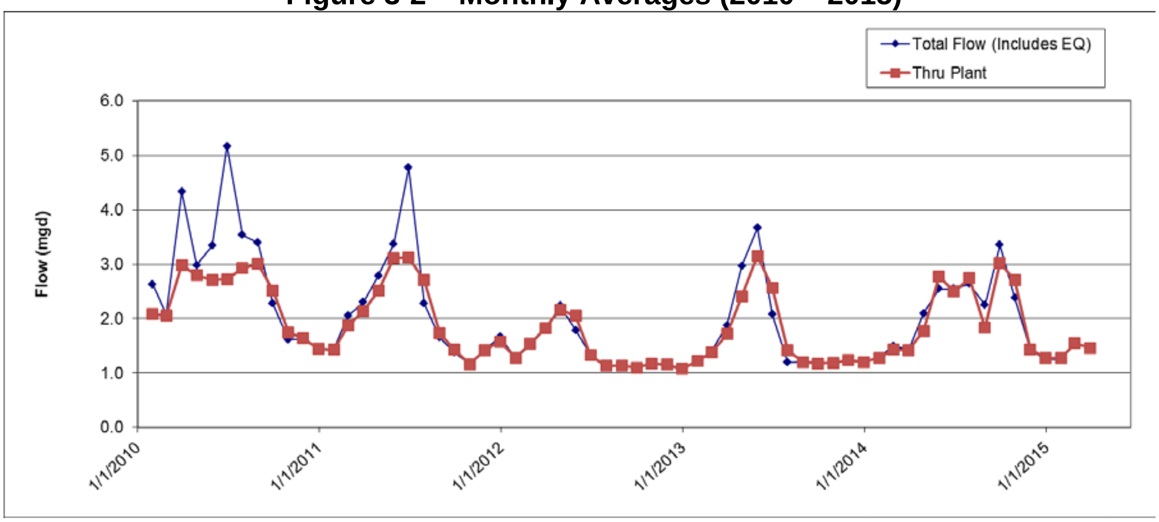
Table 3-2 - Influent Wastewater Flow Data for 2010 thru 2014

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

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.

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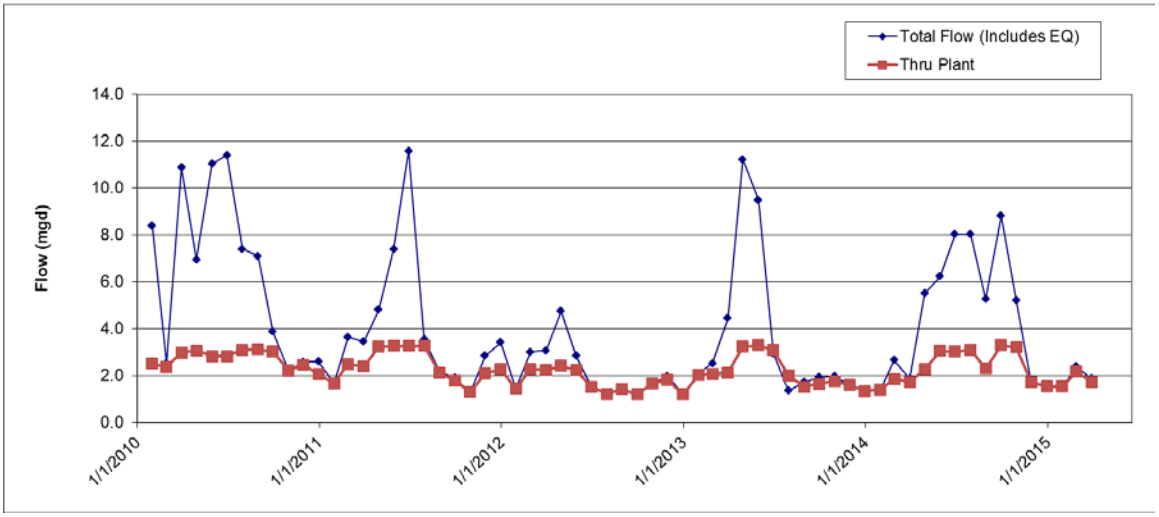
Figure 3-3 – Maximum Daily Flows (2010 – 2015)



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. Figure 3-3 summarizes the ADW, AWW, MWW, and PHWW flows (through March 2015).

Table 3-3 – Current Flows (2010 – 2015)

Parameter	Value
ADW	1.56 MGD
Daily Average	2.02 MGD
AWW	5.17 MGD
MWW	11.58 MGD
PHWW (est.*)	16.37 MGD

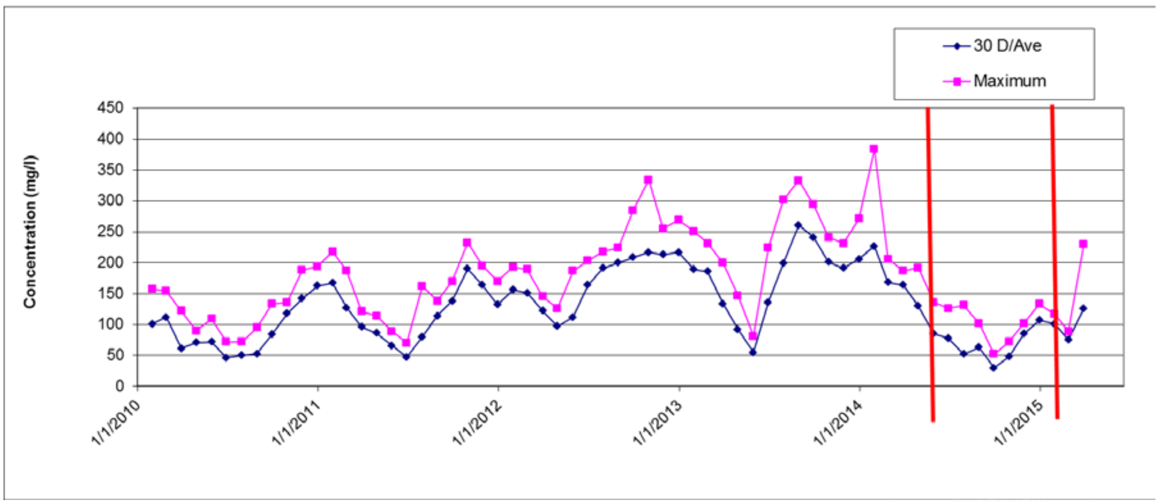
* PHWW flow estimated from sanitary sewer model. This flow was based on a 25 year, 24 hour storm with all collection system surcharges eliminated.

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. The cBOD data was reviewed for period from 2010-2015 and is shown Figure 3-4.

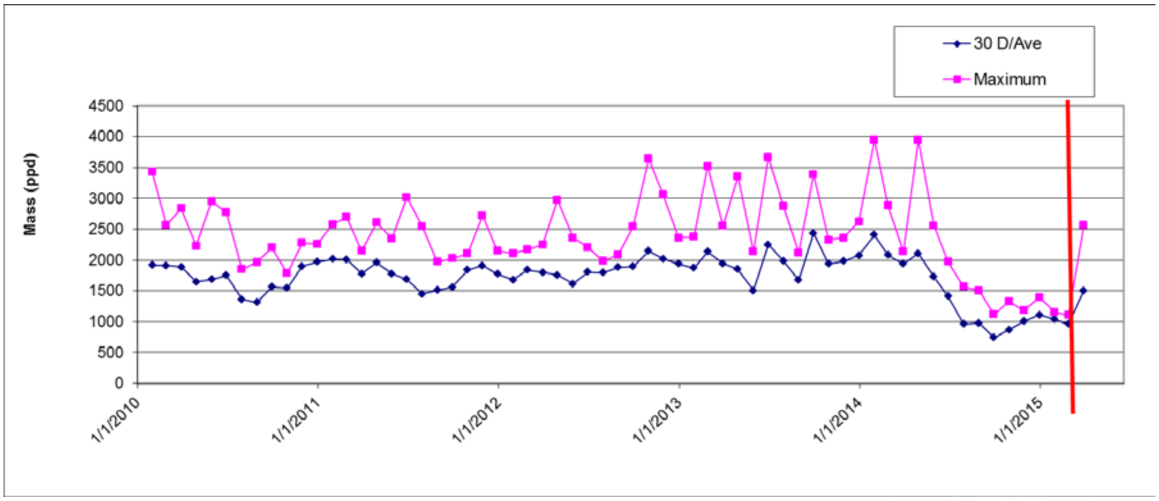
The cBOD concentration is typical of low to medium strength wastewater. It should be noted that data from June 2014 through February 2015 was thrown out since it is believed the deionized water used in the cBOD test was contaminated with copper from the distilled water still used. This chart compares 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.

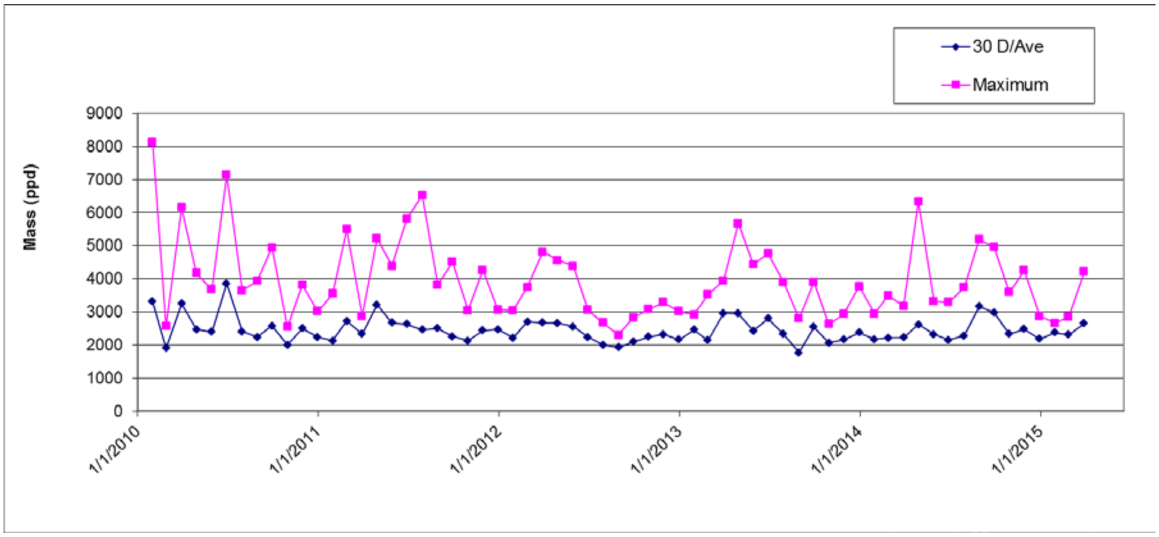
Table 3-4 – Current cBOD Loading (through 3/15)

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

<u>Total Suspended Solids</u>

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

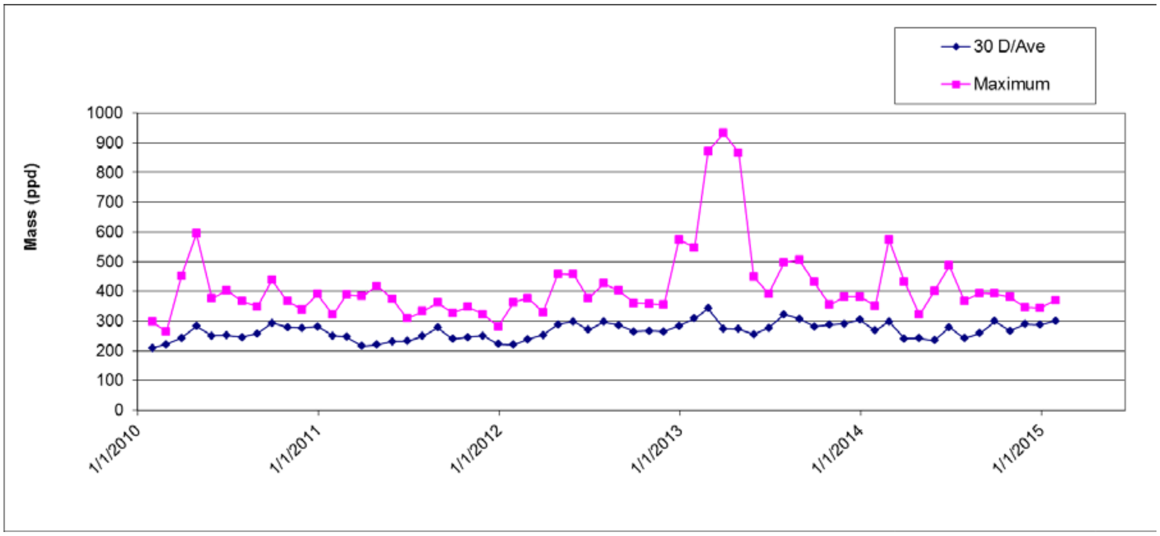
Parameter	Value (ppd)
Average Month	2,453
Max Month	3,859
Max Day	6,529*

* Outliers: 8118 and 7130

<u>Ammonia-Nitrogen and Total Kjeldahl Nitrogen</u>

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 may correspond to several wet weather events that took place in the spring of that year. Occasionally biofilm and sediment that build up in collection systems are scoured and flushed to the plant during wet weather events. 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.*

Figure 3-7 – Influent Ammonia



<u>Population Equivalent Analysis</u>

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 ratio is 1.32 max month/average. The average loading per capita is 0.12 lb/capita/day, which is lower than the typical value (0.17 lb/capita/day of BOD). However, since the cBOD test has been shown to underestimate the BOD strength of wastewater, the true BOD loading per capita may be close to the typical value. 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 266 lbs/day and 343 lbs/day for average and max month conditions respectively. This ratio is 1.29 max month/average. The average loading per capita is 0.017 lb/capita/day, which is within the typical range (0.011-0.026 lb/capita/day).

The monthly flows and loadings were reviewed from January 2010 to March 2015 to determine the AWW flow. The wettest month flow during this period was 5.17 MGD and identified as the AWW flow. To determine the MWW flow from 2010 to 2015 the maximum day was selected over the seven year period. The MWW flow for this period is 11.58 MGD. See Table 3-6 for a summary of the historic flow, cBOD and TSS loadings during the indicated time period.

Table 3-6 – 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	11.58 MGD	
PHWW	16.37 MGD	
cBOD		
Average	1840 lbs/day	0.12 lbs/cap/day
Max Month	2437 lbs/day	
Max Day	3952 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	266 lbs/day	0.017 lbs/cap/day
Max Month	343 lbs/day	
Max Day	932 lbs/day	

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 4-2. 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 4-1.

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.

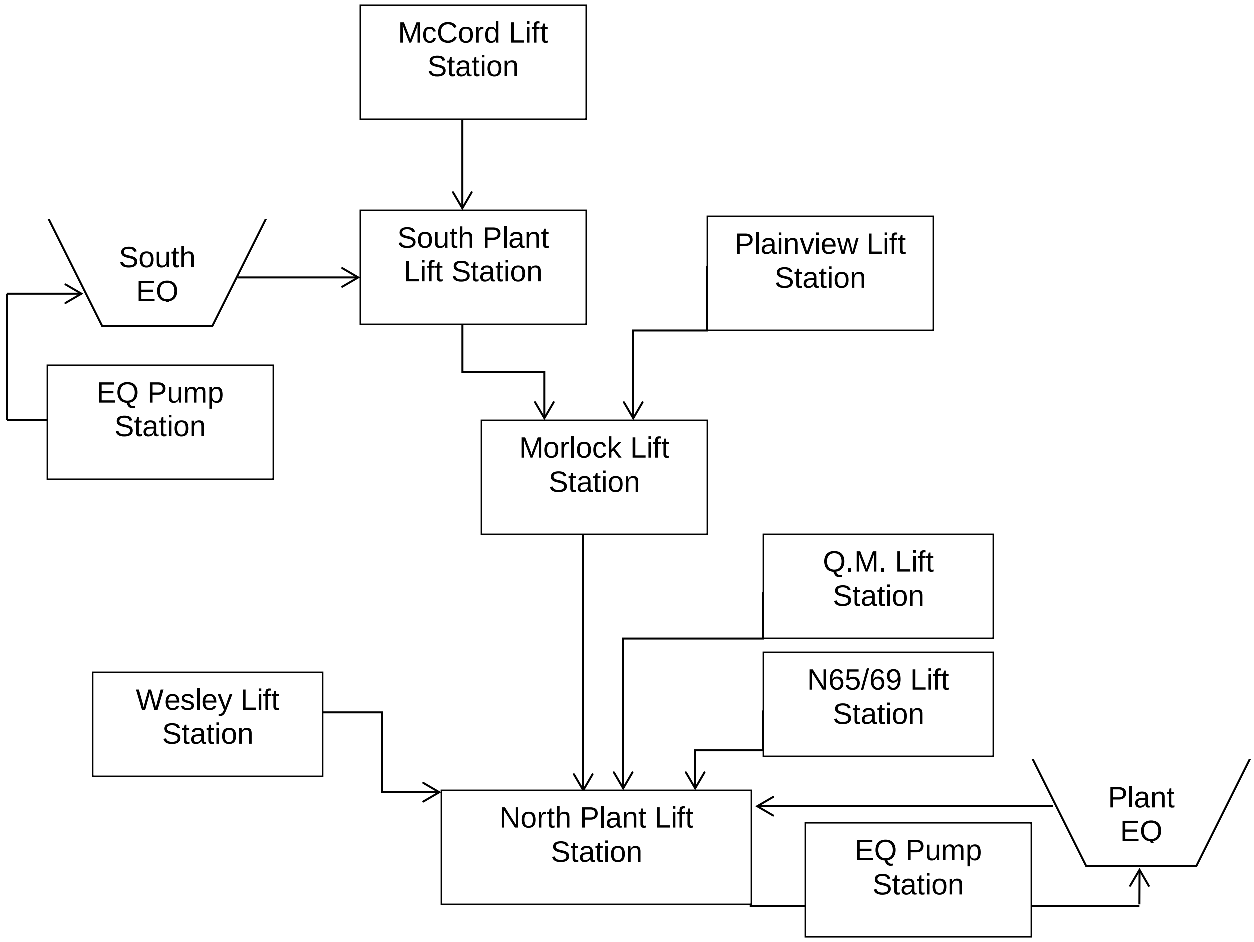


Figure 4-1 – Lift Station Flow Diagram

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.

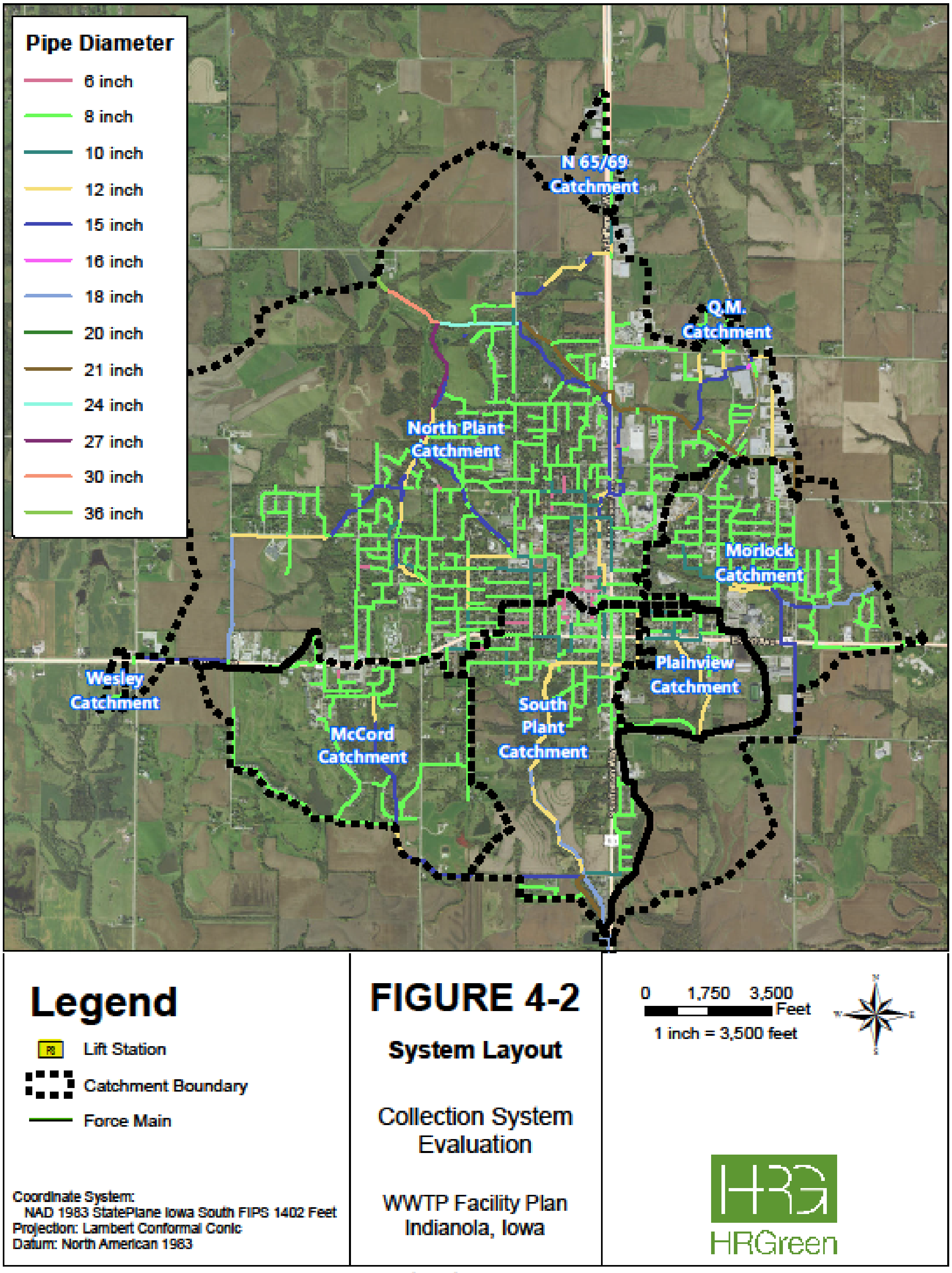


Figure 4-2 – System Layout

4.2. 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

4.3. 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. A Waste Load Allocation for each receiving stream has been developed by IDNR and is attached in Appendix B 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 included in Appendix A.

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 either of the receiving streams are indicated in the respective Waste Load Allocation presented in Appendix B.

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 phorphorus 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 phorphorus 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

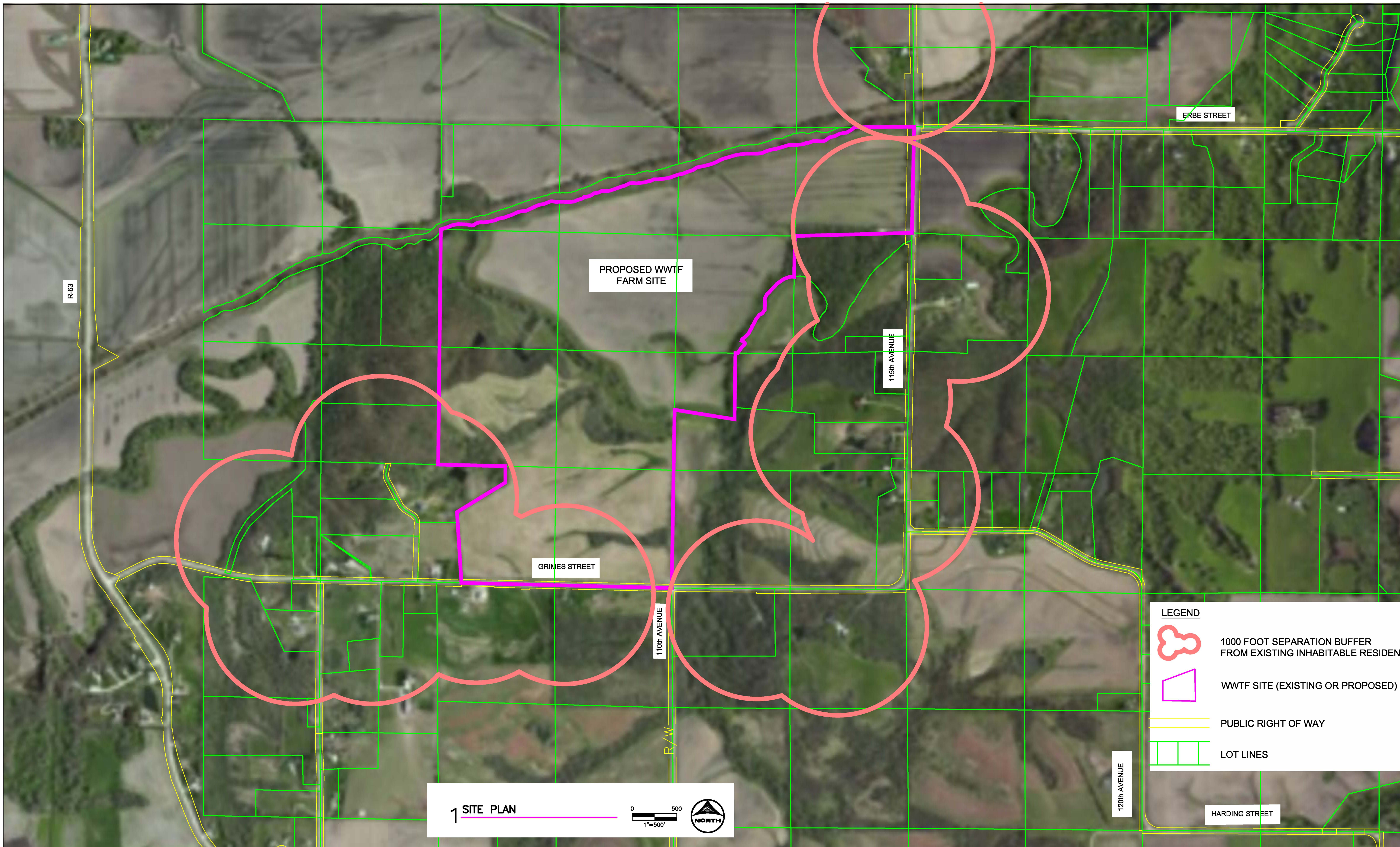
Parameter	Residential Flow	Industrial Flow	Total
Flow (MGD)			
ADW	2.09	0.21	2.30
Daily Average	2.70	0.21	2.91
AWW	5.70	0.21	5.91
MWW	12.11	0.21	12.32
PHWW	16.90	0.21	17.11
cBOD (lbs/day)			
Average	2463	525	2988
Max Month	3262	525	3787
Max Day	5289	525	5815
TSS (lbs/day)			
Average	3283	613	3896
Max Month	5165	613	5778
Max Day	8738	613	9351
Ammonia-N (lbs/day)			
Ave Month	356	61	417
Max Month	472	61	533
Max Day	765	61	826
TKN (lbs/day)			
Average	548	94	642
Max Month	725	94	820
Max Day	1919	94	2013
Total Phosphorus (lb/day) [(1)]			
Average Month	124	21	145
Max Month	162	21	183
Max Day	266	21	287

[(1)] Indianola WWTP does not have long history of monitoring influent Phosphorous. Design loads have been developed on small sample data.

5.4. TREATMENT PLANT SITE

The proposed new wastewater treatment plant facilities will be located at the Farm Site approximately 1.5 miles to the northwest of the existing NWWTF site. The Farm Site property includes approximately 360 acres of current farmland and river bottom land adjacent to Cavitt Creek and the Middle River owned by the City of Indianola. Ample space is available at the Farm Site for new treatment facilities to be sited to comply with the 1,000-foot site separation as required by the IDNR rules. Figure 5-1 shows the Farm Site and proposed site separation.

The Farm Site is currently leased to a farmer that harvests crops on much of the acreage. The north and east parts of the Farm Site are within the floodway of Cavitt Creek and the Middle River. No wastewater treatment facilities will be constructed in the floodway.



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BAR IS ONE INCH ON OFFICIAL DRAWINGS. 0 1" IF NOT ONE INCH, ADJUST SCALE ACCORDINGLY.

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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 C.

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.

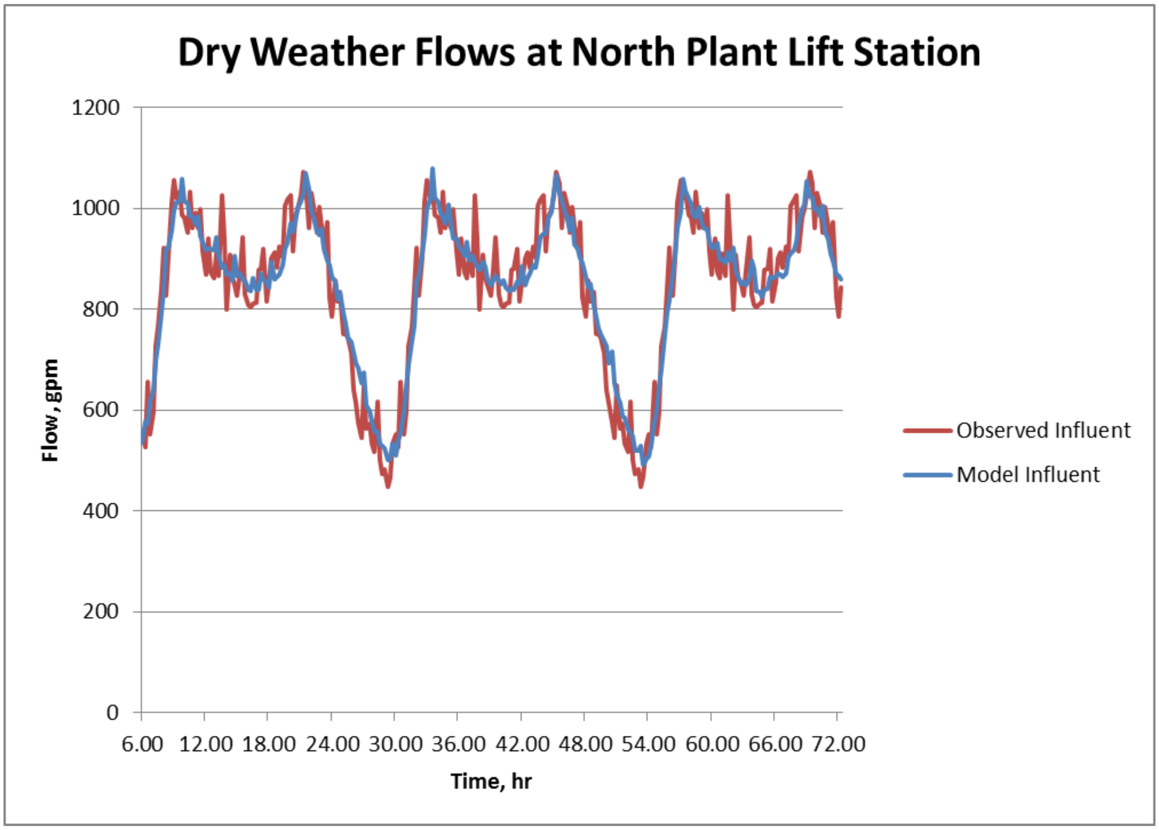


Figure 6-1 – Collection System Model – City of Indianola Lift Station Influent Model Flows vs. North Lift Station Influent Observed Flows

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.

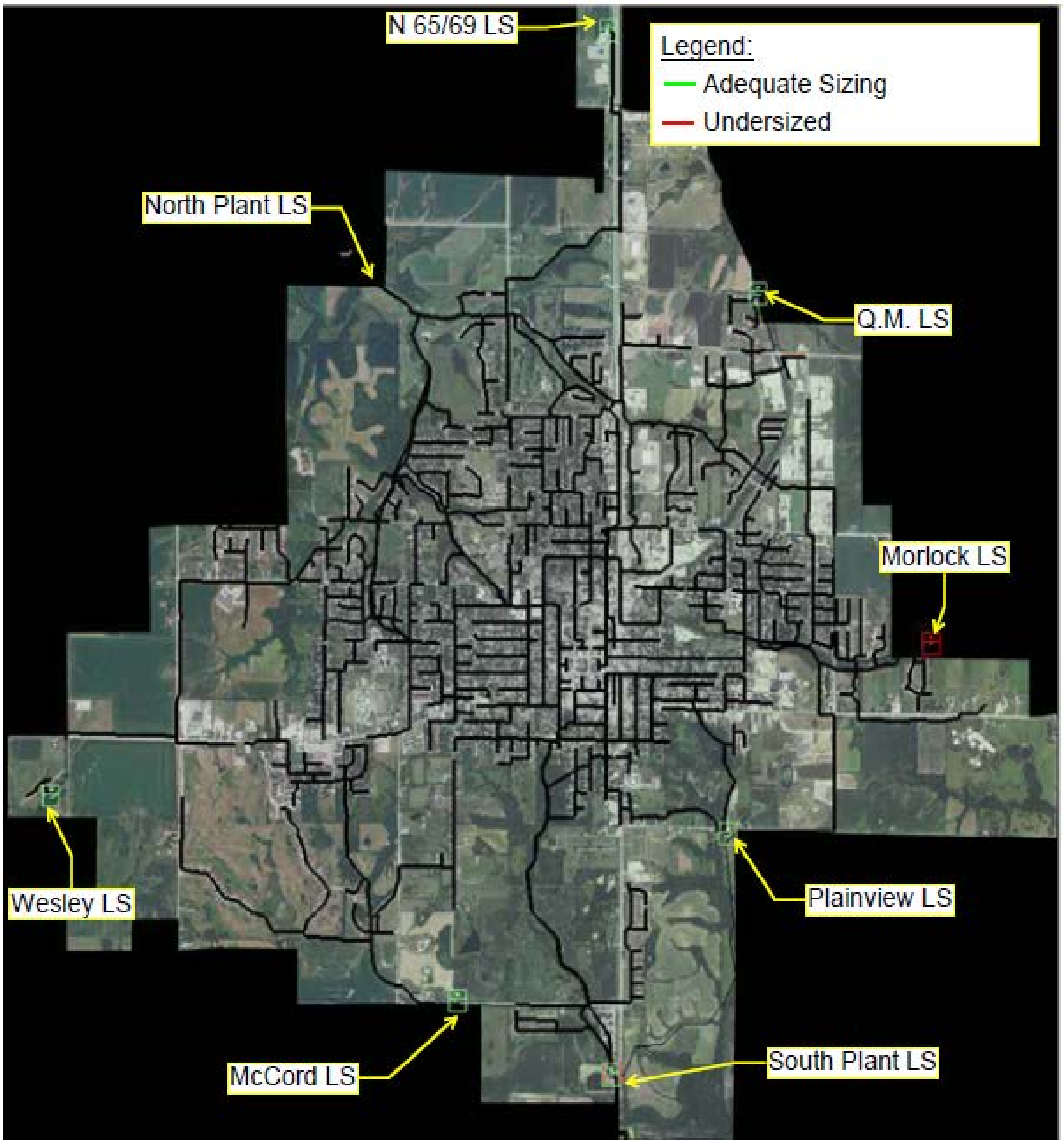


Figure 6-2 – Model Output – Lift Station Analysis During 25-Year, 24-Hour Storm

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.

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

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. Because of the high ratio of peak to average flows, influent wastewater equalization will continue to be important at 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; 1) Reuse of screening, raw wastewater pumping and equalization at the existing treatment plant site with new fine screening and grit removal at the Farm Site; and 2) Convey the influent flows to the Farm Site by gravity and construct new preliminary treatment and equalization there. The remaining portion of this section provides a detailed evaluation of these alternatives.

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

<u>Benefits of Preliminary Treatment alternative P1</u>

- 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 peak flow treatment at the Farm Site. During peak flows the new wastewater treatment plant would treat the first 6.0 mgd of flow with higher peak flows being bypassed around secondary treatment and treated by peak flow treatment and combined with fully treated flows.

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 17.1 mgd. 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 desireable. 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 Peak Flow 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 Peak Flow 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 Peak Flow Treatment process. This excess flow pump station will have automatic controls with preset pumping ranges for each selected mode of operation.

7.3.5. Peak Flow Treatment

Peak Flow Treatment is a new approach available to Iowa wastewater facilities to handling 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 D 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.

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

<u>Benefits of Preliminary Treatment alternative P2</u>

- All wastewater treatment facilities are on the Farm Site.
  - ✓ Easier to operate/maintain.

  - ✓ 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.

<u>Disadvantages of Preliminary Treatment alternative P2</u>

- 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.

Table 7-2 – Alternative P2 – Conceptual Opinion of Probable Construction 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
Peak flow 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.

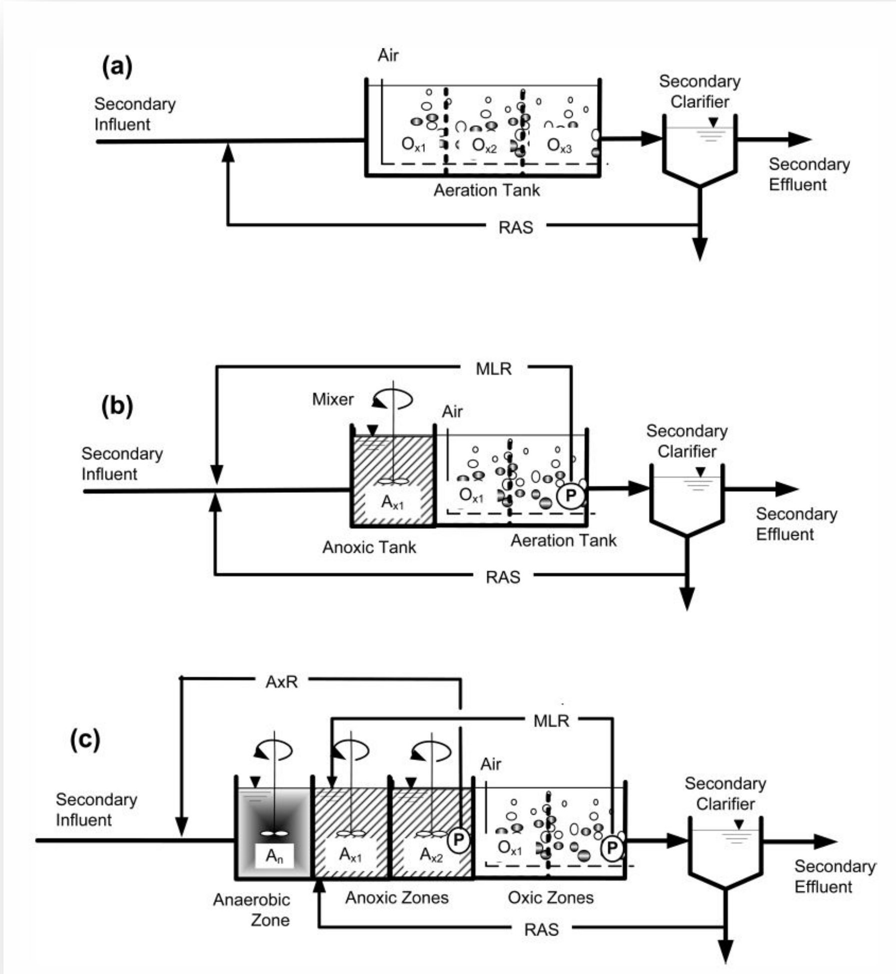


Figure 8-1 – Schematic of BNR Processes

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 4.2:1, 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.

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.

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

MLE Activated Sludge Process:

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. 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 during peak events will either be held in equalization for later treatment, or pass through peak flow treatment and blend with secondary treated flows prior to 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 three cast-in-place concrete oxidation ditches (reactors) followed by three cast-in-place concrete circular final clarifiers. Effluent from the 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.

8.2.1. Oxidation Ditch Reactors

Three 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.

Aerobic/Nitrification. 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 3,000,000 gallons. Side water depth will be verified during design but is expected to be in the range of 12 to 15 feet, depending on the type of mixer selected and the size of the impeller.

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, the Surface Overflow Rate controlled the sizing and the reliability criteria suggested the number of units that would be most cost-effective.

Figure 8-3 – Oxidation Ditch with Clarifiers

- Clarifier Options:

Number	Diameter	HRT at Avg. Flow
2 Clarifiers	88' Dia ea.	9.0 hours
3 Clarifiers	62' Dia ea.	6.7 hours
4 Clarifiers	51' Dia ea.	6.0 hours

Three circular cast-in-place concrete final clarifiers were selected based on expected performance and costs.

Ferric chloride or aluminum sulfate (alum) can be fed at the flow split structure for the final clarifiers or further upstream 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.

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

Benefits of Secondary Treatment alternative ST1

- 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 3.1 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.

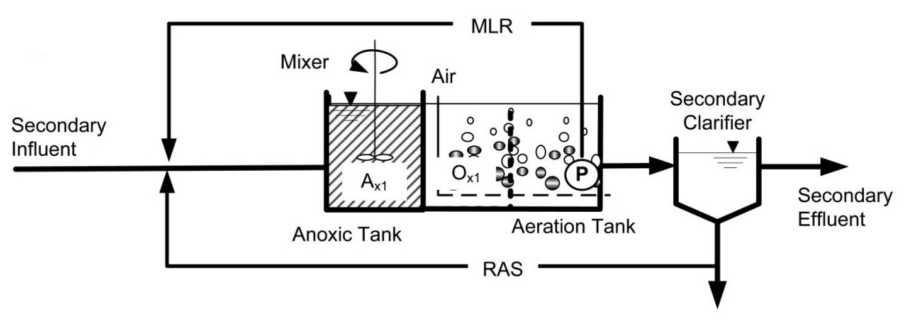


Figure 8-4 – Modified Ludzack-Ettinger (MLE) Process

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.

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 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	30.0	47.6
Volume, cu ft.	118,734	118,734
, gal	888,192	888,192
Detention time, hrs.	7.3	3.55

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 150% of the average flow or the required RAS flow for 6.0 MGD. The design pumping rate will be 3,330 gpm, 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.

8.3.3. 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.

8.3.4. Benefits and Disadvantages of Secondary Treatment Alternative ST2

Benefits of Secondary Treatment alternative ST2

- 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.

Disadvantages of Secondary Treatment alternative ST2

- 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 compliment 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 are required with an SBR system.

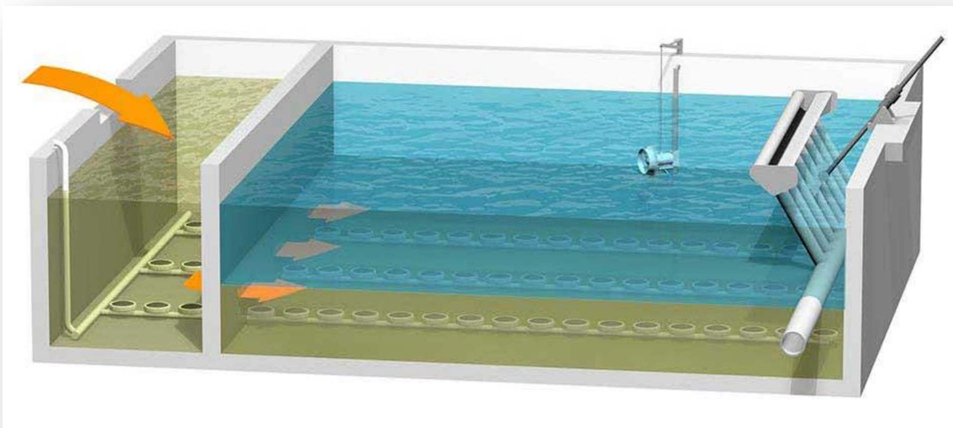


Figure 8-7 – SBR Process

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 wil be pumped to the solids treatment process.

8.4.1. 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.

8.4.2. Benefits and Disadvantages of Secondary Treatment Alternative ST3

<u>Benefits of Secondary Treatment alternative ST3</u>

- 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.

<u>Disadvantages of Secondary Treatment alternative ST3</u>

- 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 8.0 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

Benefits of Solids Processing alternative SP1

- 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

Disadvantages of Solids Processing alternative SP1

- 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

Benefits of Solids Processing alternative SP2

- 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

Disadvantages of Solids Processing alternative SP2

- 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 not 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. PT2 + ST1 + SP1

(Gravity sewer to Farm Site, Headworks Building, Grit Removal, Daily Equalization, Peak Flow 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 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 peak flow 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, Peak Flow 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 peak flow 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, Peak Flow 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 peak flow 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. Note that additional UV disinfection would be required for this alternative when the peak flow treatment system is operational during disinfection season. 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 PT2 + ST1 + SP2.

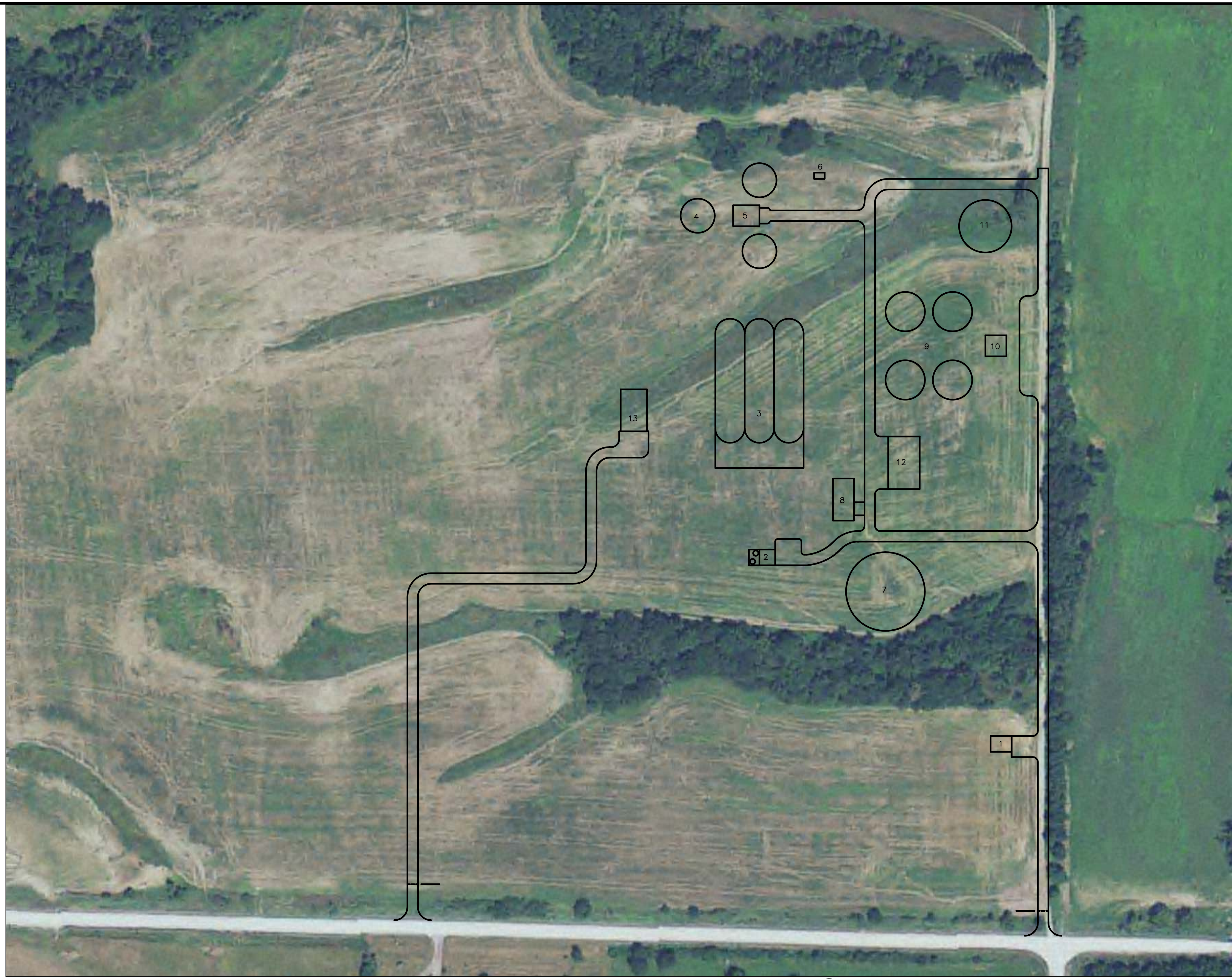
12.2. CONVEYANCE

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 Peak Flow 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



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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APPROVED:	MAD	JOB NUMBER:	40120059
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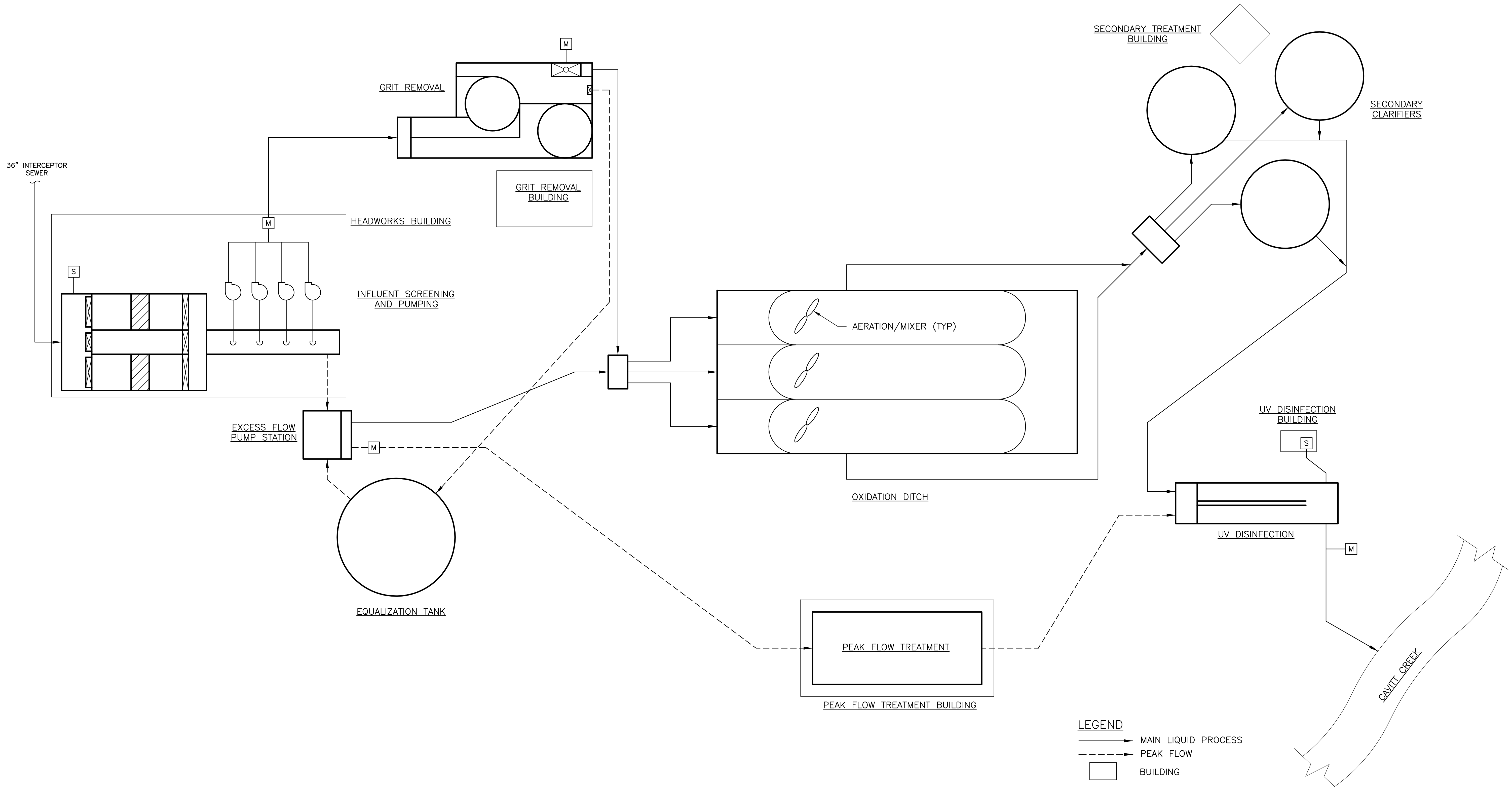
INDIANOLA – SITING STUDY
CITY OF INDIANOLA
INDIANOLA, IOWA 2013

WASTEWATER TREATMENT
SITE PLAN

SHEET NO.
FIG 12–1



LIQUID TREATMENT PROCESS SCHEMATIC

PRELIMINARY
NOT FOR CONSTRUCTION

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INDIANOLA WASTEWATER SYSTEM IMPROVEMENTS
CITY OF INDIANOLA
INDIANOLA, IOWA

GENERAL
LIQUID TREATMENT PROCESS SCHEMATIC

SHEET NO.
FIG 12-2

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

above 6.0 mgd to the Peak Flow Treatment process. The excess flow pump station will be a submersible pump station with a connected valve vault.

The Peak Flow Treatment system will be 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.

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. 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.

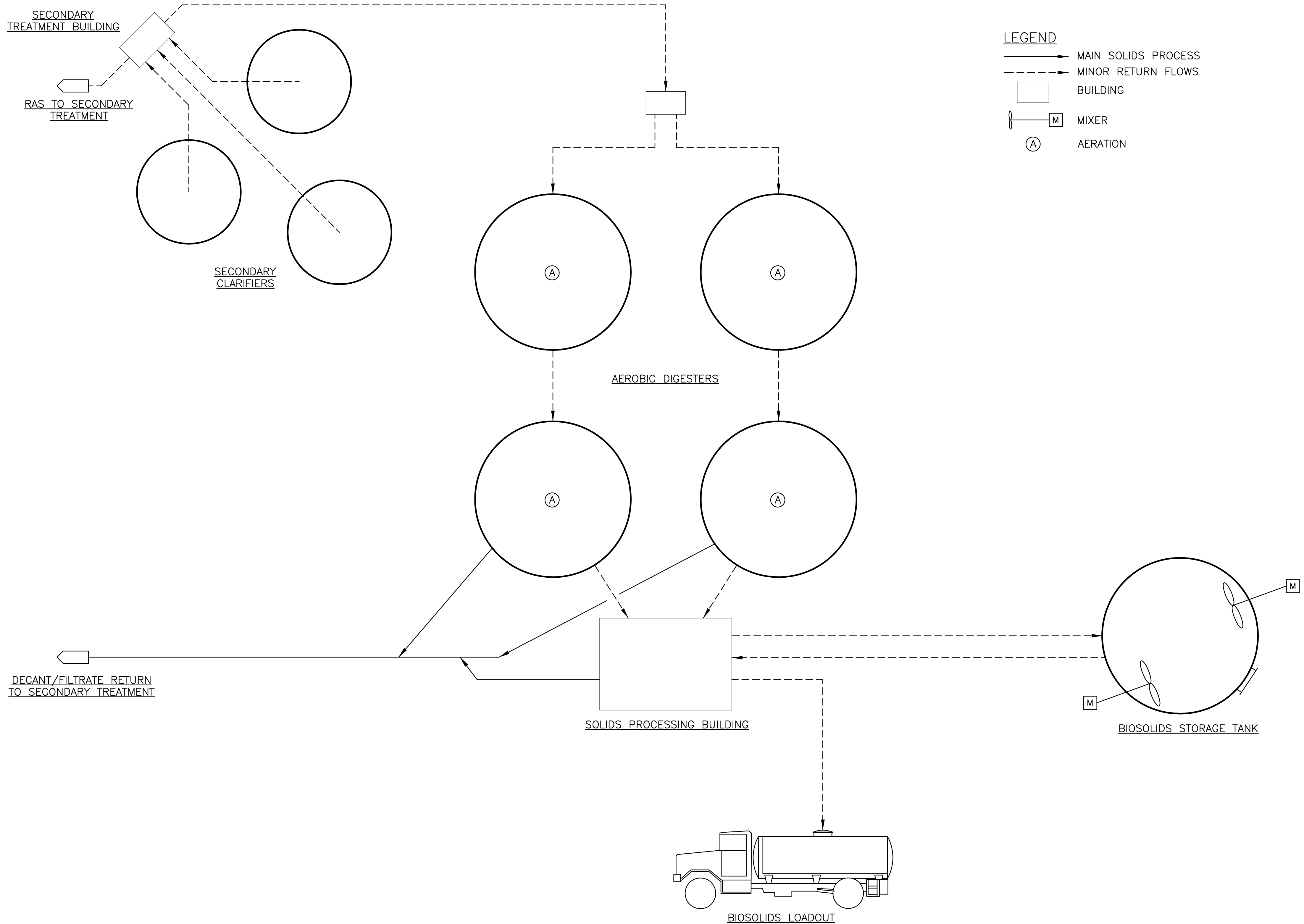
Three secondary clarifiers will be provided to settle the activated sludge following the oxidation ditches. The clarified effluent will flow over weirs to the disinfection process. The activated sludge settling in the clarifiers will be pumped back to the treatment process as return activated sludge 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 most of 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 Cavitt Creek. The UV disinfection will also disinfect flows from the Peak Flow 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.



SOILDS TREATMENT PROCESS SCHEMATIC

PRELIMINARY
NOT FOR CONSTRUCTION

DRAWN BY: CMB	JOB DATE: 2015	BAR IS ONE INCH ON OFFICIAL DRAWINGS. 0 — 1" IF NOT ONE INCH, ADJUST SCALE ACCORDINGLY.	
APPROVED: JRR	JOB NUMBER: 40150016		
CAD DATE: 4/25/2016 3:46:08 PM			
CAD FILE: O:\40150016\CAD\Dwgs\FIG 12–3.dwg			

NO.	DATE	BY	REVISION DESCRIPTION

HRGreen
HRGreen.com

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

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.

12.5. SUMMARY OF DESIGN PARAMETERS

Item		Size/Capacity
WWTP Flows		
ADW		2.30 mgd
AWW		5.91 mgd
MWW		12.32 mgd
PHWW		17.11 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	417	826
TKN, lbs/day	642	2,013
Total Phosphorus, lbs/day		
Mechanical Screens		
No. of units		2
Clear opening size, in		¼
Max flow per screen, mgd		18.0
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	3
Tank volume, each, gallons	1,000,000
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	9.0

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. 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
Complete Antidegradation Analysis - Submit to IDNR	May 2016
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
30% Complete	March 2019
60% Complete	June 2019
90% Complete	August 2019
Submit Final Design for IDNR Construction Permit	September 2019
Construction Permit Issued	December 2019
Bidding/Award	January 2020
Construction Begins	March 2020
Construction Substantially Complete	November 2021
Construction Complete	June 2022

APPENDIX A
Existing NPDES Discharge Permit

IOWA DEPARTMENT OF NATURAL RESOURCES

National Pollutant Discharge Elimination System (NPDES) Permit

JAN 2002

RECORD COPY
File 91-33-0-01
Section Initials

PERMITTEE

CITY OF INDIANOLA
CITY CLERK-CITY HALL
PO BOX 299
INDIANOLA, IA 50125

IDENTITY AND LOCATION OF FACILITY

INDIANOLA CITY OF STP (NORTH)
Section 26, T 76N, R24W
WARREN County, Iowa

CERTIFIED
0144 7793
Return Receipt Requested

IOWA NPDES PERMIT NUMBER: 9133001

DATE OF ISSUANCE: 01-02-2002

DATE OF EXPIRATION: 01-01-2007

RECEIVING STREAM

CAVITT CREEK

ROUTE OF FLOW

YOU ARE REQUIRED TO FILE FOR RENEWAL OF THIS PERMIT BY: 07-05-2006

EPA NUMBER: IA0027669

This permit is issued pursuant to the authority of section 402(b) of the Clean Water Act (33 U.S.C 1342(b)), Iowa Code section 455B.174, and rule 567--64.3, Iowa Administrative Code. You are authorized to operate the disposal system and to discharge the pollutants specified in this permit in accordance with the effluent limitations, monitoring requirements and other terms set forth in this permit.

You may appeal any conditions of this permit by filing a written notice of appeal and request for administrative hearing with the director of this department within 30 days of your receipt of this permit.

Any existing, unexpired Iowa operation permit or Iowa NPDES permit previously issued by the department for the facility identified above is revoked by the issuance of this Iowa NPDES operation permit.

FOR THE DEPARTMENT OF NATURAL RESOURCES

By

Wayne Farrand, Supervisor
Wastewater Section
ENVIRONMENTAL PROTECTION DIVISION

Facility Name: INDIANOLA CITY OF STP (NORTH)

Permit Number: 9133001

Outfall Number	**Outfall Description**
001	DISCHARGE FROM AN ACTIVATED SLUDGE WASTWATER TREATMENT FACILITY.

Receiving Stream: CAVITT CREEK

Route of Flow:

Class B(LR) waters are limited resource warm waters in which flow or other physical characteristics limit the ability of the water body to maintain a balanced warm water community. Such waters support only populations composed of species able to survive and reproduce in a wide range of physical and chemical conditions, and are not generally harvested for human consumption.

002 NORTH PLANT STORM WATER RETENTION PONDS EMERGENCY OVERFLOW.

Receiving Stream: CAVITT CREEK

Route of Flow:

Class B(LR) waters are limited resource warm waters in which flow or other physical characteristics limit the ability of the water body to maintain a balanced warm water community. Such waters support only populations composed of species able to survive and reproduce in a wide range of physical and chemical conditions, and are not generally harvested for human consumption.

003 SOUTH PLANT LIFT STATION EMERGENCY OVERFLOW.

Receiving Stream: SOUTH RIVER

Route of Flow:

Class B(LR) waters are limited resource warm waters in which flow or other physical characteristics limit the ability of the water body to maintain a balanced warm water community. Such waters support only populations composed of species able to survive and reproduce in a wide range of physical and chemical conditions, and are not generally harvested for human consumption.

Facility Name: INDIANOLA CITY OF STP (NORTH)

Permit Number: 9133001

Effluent Limitations

Outfall No.: 001 DISCHARGE FROM AN ACTIVATED SLUDGE WASTWATER TREATMENT FACILITY.

You are prohibited from discharging pollutants except in compliance with the following effluent limitations:

Wastewater Parameter	Season	Type of Limit	% Removal	EFFLUENT LIMITATIONS: Concentration: 7 Day Average/Min	Concentration: 30 Day Average	Concentration: Daily Maximum	Concentration: Units	Mass: 7 Day Average	Mass: 30 Day Average	Mass: Daily Maximum	Mass: Units
CBOD5	YEARLY	FINAL	85	40.0	25.0		MG/L	834.0	521.0		LBS/DAY
TOTAL SUSPENDED SOLIDS	YEARLY	FINAL	85	45.0	30.0		MG/L	938.0	626.0		LBS/DAY
AMMONIA NITROGEN (N)	JAN	FINAL			7.2	15.4	MG/L		133.0	320.0	LBS/DAY
AMMONIA NITROGEN (N)	FEB	FINAL			8.1	14.5	MG/L		150.0	300.0	LBS/DAY
AMMONIA NITROGEN (N)	MAR	FINAL			6.3	14.9	MG/L		116.0	309.0	LBS/DAY
AMMONIA NITROGEN (N)	APR	FINAL			2.8	15.9	MG/L		52.0	329.0	LBS/DAY
AMMONIA NITROGEN (N)	MAY	FINAL			2.4	15.3	MG/L		45.0	319.0	LBS/DAY
AMMONIA NITROGEN (N)	JUN	FINAL			1.7	14.6	MG/L		32.0	303.0	LBS/DAY
AMMONIA NITROGEN (N)	JUL	FINAL			1.5	17.8	MG/L		28.0	369.0	LBS/DAY
AMMONIA NITROGEN (N)	AUG	FINAL			1.4	16.4	MG/L		26.0	340.0	LBS/DAY
AMMONIA NITROGEN (N)	SEP	FINAL			1.9	16.7	MG/L		36.0	346.0	LBS/DAY
AMMONIA NITROGEN (N)	OCT	FINAL			3.8	15.9	MG/L		71.0	330.0	LBS/DAY
AMMONIA NITROGEN (N)	NOV	FINAL			4.6	14.8	MG/L		86.0	308.0	LBS/DAY
AMMONIA NITROGEN (N)	DEC	FINAL			5.4	16.1	MG/L		101.0	335.0	LBS/DAY
PH (MINIMUM - MAXIMUM)	YEARLY	FINAL		6.0		9.0	STD UNITS				
DISSOLVED OXYGEN (MINIMUM)	YEARLY	FINAL		4.2			MG/L				
ACUTE TOXICITY, CERIODAPHNIA	YEARLY	FINAL							1.0		NO TOXICITY
ACUTE TOXICITY, PIMEPHALES	YEARLY	FINAL							1.0		NO TOXICITY

Note: If seasonal limits apply, summer is from April 1 through October 31, and winter is from November 1 through March 31.

Facility Name: INDIANOLA CITY OF STP (NORTH)

Permit Number: 9133001

Monitoring and Reporting Requirements

(a) Samples and measurements taken shall be representative of the volume and nature of the monitored wastewater.

(b) Analytical and sampling methods specified in 40 CFR Part 136 or other methods approved in writing by the department shall be utilized.

(c) Chapter 63 of the Iowa Administrative Code provides you with further explanation of your monitoring requirements.

(d) You are required to report all data including calculated results needed to determine compliance with the limitations contained in this permit. This includes daily maximums and minimums, 30-day averages and 7-day averages for all parameters that have concentration (mg/l) and mass (lbs/day) limits. Also, flow data shall be reported in million gallons per day (MGD).

(e) Results of all monitoring shall be recorded on forms provided by, or approved by, the department, and shall be submitted to the department by the fifteenth day following the close of the reporting period. Your reporting period is on a monthly basis, ending on the last day of each reporting period.

Outfall Number	Wastewater Parameter	Sample Frequency	Sample Type	Monitoring Location
001	CBOD5	2 TIMES PER WEEK	24 HOUR COMPOSITE	RAW WASTE
001	TOTAL SUSPENDED SOLIDS	1 TIME PER WEEK	24 HOUR COMPOSITE	RAW WASTE
001	PH (MINIMUM - MAXIMUM)	2 TIMES PER WEEK	GRAB	RAW WASTE
001	TEMPERATURE	2 TIMES PER WEEK	GRAB	RAW WASTE
001	FLOW	7/WEEK OR DAILY	24 HOUR TOTAL	FINAL EFFLUENT
001	CBOD5	2 TIMES PER WEEK	24 HOUR COMPOSITE	FINAL EFFLUENT
001	TOTAL SUSPENDED SOLIDS	1 TIME PER WEEK	24 HOUR COMPOSITE	FINAL EFFLUENT
001	AMMONIA NITROGEN (N)	7/WEEK OR DAILY	24 HOUR COMPOSITE	FINAL EFFLUENT
001	PH (MINIMUM - MAXIMUM)	2 TIMES PER WEEK	GRAB	FINAL EFFLUENT
001	DISSOLVED OXYGEN (MINIMUM)	2 TIMES PER WEEK	GRAB	FINAL EFFLUENT
001	TEMPERATURE	2 TIMES PER WEEK	GRAB	FINAL EFFLUENT
001	ACUTE TOXICITY, CERIODAPHNIA	1 EVERY 12 MONTHS	24 HOUR COMPOSITE	FINAL EFFLUENT
001	ACUTE TOXICITY, PIMEPHALES	1 EVERY 12 MONTHS	24 HOUR COMPOSITE	FINAL EFFLUENT
001	PH (MINIMUM - MAXIMUM)	3 TIMES PER WEEK	GRAB	ANAEROBIC DIGESTER 1 CONTENTS
001	ALKALINITY,TOTAL (AS CACO3)	1 TIME PER WEEK	GRAB	ANAEROBIC DIGESTER 1 CONTENTS

Facility Name: INDIANOLA CITY OF STP (NORTH)

Permit Number: 9133001

Monitoring and Reporting Requirements

(a) Samples and measurements taken shall be representative of the volume and nature of the monitored wastewater.

(b) Analytical and sampling methods specified in 40 CFR Part 136 or other methods approved in writing by the department shall be utilized.

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(e) Results of all monitoring shall be recorded on forms provided by, or approved by, the department, and shall be submitted to the department by the fifteenth day following the close of the reporting period. Your reporting period is on a monthly basis, ending on the last day of each reporting period.

Outfall Number	Wastewater Parameter	Sample Frequency	Sample Type	Monitoring Location
001	TEMPERATURE	3 TIMES PER WEEK	GRAB	ANAEROBIC DIGESTER 1 CONTENTS
001	VOLATILE ACIDS	1 TIME PER WEEK	GRAB	ANAEROBIC DIGESTER 1 CONTENTS
001	DISSOLVED OXYGEN (MINIMUM)	3 TIMES PER WEEK	GRAB	AERATION BASIN 2 CONTENTS
001	SOLIDS,MIXED LIQUOR SUSPENDED	3 TIMES PER WEEK	GRAB	AERATION BASIN 2 CONTENTS
001	TEMPERATURE	3 TIMES PER WEEK	GRAB	AERATION BASIN 2 CONTENTS
001	30-MINUTE SETTLEABILITY	3 TIMES PER WEEK	GRAB	AERATION BASIN 2 CONTENTS
001	DISSOLVED OXYGEN (MINIMUM)	3 TIMES PER WEEK	GRAB	AERATION BASIN 1 CONTENTS
001	SOLIDS,MIXED LIQUOR SUSPENDED	3 TIMES PER WEEK	GRAB	AERATION BASIN 1 CONTENTS
001	TEMPERATURE	3 TIMES PER WEEK	GRAB	AERATION BASIN 1 CONTENTS
001	30-MINUTE SETTLEABILITY	3 TIMES PER WEEK	GRAB	AERATION BASIN 1 CONTENTS

Facility Name: INDIANOLA CITY OF STP (NORTH)

Permit Number: 9133001

Industrial Contributor Discharges

Industrial Contributor: SOUTH CENTRAL IOWA LANDFILL AGENCY

Outfall Number	Outfall Description
001	SANITARY LANDFILL LEACHATE TRUCKED TO THE CITY WASTEWATER TREATMENT PLANT.

Facility Name: INDIANOLA CITY OF STP (NORTH)

Permit Number: 9133001

Industrial Contributor Effluent Limitations

Industrial Contributor: SOUTH CENTRAL IOWA LANDFILL AGENCY

Outfall No.: 001 SANITARY LANDFILL LEACHATE TRUCKED TO THE CITY WASTEWATER TREATMENT PLANT.

You are prohibited from discharging pollutants except in compliance with the following effluent limitations:

Wastewater Parameter	Season	Type of Limit	% Removal	EFFLUENT LIMITATIONS							
				Concentration				Mass			
				7 Day Average/Min	30 Day Average	Daily Maximum	Units	7 Day Average	30 Day Average	Daily Maximum	Units
FLOW	YEARLY	FINAL			0.002	0.002	MGD				
BIOCHEMICAL OXYGEN DEMAND (BOD5)	YEARLY	FINAL							15.0	15.0	LBS/DAY
TOTAL SUSPENDED SOLIDS	YEARLY	FINAL							25.0	25.0	LBS/DAY
AMMONIA NITROGEN (N)	YEARLY	FINAL							6.4	6.4	LBS/DAY
PH (MINIMUM - MAXIMUM)	YEARLY	FINAL		6.0		9.5	STD UNITS				
CADMIUM, TOTAL (AS CD)	YEARLY	FINAL							0.0002	0.0002	LBS/DAY
CHROMIUM, TOTAL (AS CR)	YEARLY	FINAL							0.002	0.002	LBS/DAY
COPPER, TOTAL (AS CU)	YEARLY	FINAL							0.022	0.022	LBS/DAY
LEAD, TOTAL (AS PB)	YEARLY	FINAL							0.0013	0.0013	LBS/DAY
NICKEL, TOTAL (AS NI)	YEARLY	FINAL							0.0036	0.0036	LBS/DAY
NITROGEN, TOTAL KJELDAHL (AS N)	YEARLY	FINAL							7.5	7.5	LBS/DAY
OIL AND GREASE	YEARLY	FINAL			100.0	100.0	MG/L				
ZINC, TOTAL (AS ZN)	YEARLY	FINAL							0.0334	0.0334	LBS/DAY
BETX	YEARLY	FINAL			0.75	0.75	MG/L				
TOTAL PETROLEUM HYDROCARBONS	YEARLY	FINAL			10.0	10.0	MG/L				

Note: If seasonal limits apply, summer is from March 15 through November 15, and winter is from November 16 through March 14.

Facility Name: INDIANOLA CITY OF STP (NORTH)

Permit Number: 9133001

Industrial Contributor Monitoring and Reporting Requirements

Industrial Contributor: SOUTH CENTRAL IOWA LANDFILL AGENCY

(a) Samples and measurements taken shall be representative of the volume and nature of the monitored wastewater.

(b) Analytical and sampling methods specified in 40 CFR Part 136 or other methods approved in writing by the department shall be utilized.

(c) Chapter 63 of the Iowa Administrative Code provides you with further explanation of your monitoring requirements.

(d) You are required to report all data including calculated results needed to determine compliance with the limitations contained in this permit. This includes daily maximums and minimums, 30-day averages and 7-day averages for all parameters that have concentration (mg/l) and mass (lbs/day) limits. Also, flow data shall be reported in million gallons per day (MGD).

(e) Results of all monitoring shall be recorded on forms provided by, or approved by, the department, and shall be submitted to the department by the fifteenth day following the close of the reporting period. Your reporting period is on a monthly basis, ending on the last day of each reporting period.

Outfall Number	Wastewater Parameter	Sample Frequency	Sample Type	Monitoring Location
001	FLOW	1 EVERY BATCH	24 HOUR TOTAL	PRIOR TO DISCHARGE TO CITY SEWER
001	BIOCHEMICAL OXYGEN DEMAND (BOD5)	1 EVERY 3 MONTHS	GRAB	PRIOR TO DISCHARGE TO CITY SEWER
001	TOTAL SUSPENDED SOLIDS	1 EVERY 3 MONTHS	GRAB	PRIOR TO DISCHARGE TO CITY SEWER
001	AMMONIA NITROGEN (N)	1 EVERY 3 MONTHS	GRAB	PRIOR TO DISCHARGE TO CITY SEWER
001	PH (MINIMUM - MAXIMUM)	1 EVERY 3 MONTHS	GRAB	PRIOR TO DISCHARGE TO CITY SEWER
001	CADMIUM,TOTAL (AS CD)	1 EVERY MONTH	GRAB	PRIOR TO DISCHARGE TO CITY SEWER
001	CHROMIUM,TOTAL (AS CR)	1 EVERY MONTH	GRAB	PRIOR TO DISCHARGE TO CITY SEWER
001	COPPER,TOTAL (AS CU)	1 EVERY MONTH	GRAB	PRIOR TO DISCHARGE TO CITY SEWER
001	LEAD,TOTAL (AS PB)	1 EVERY MONTH	GRAB	PRIOR TO DISCHARGE TO CITY SEWER
001	NICKEL,TOTAL (AS NI)	1 EVERY MONTH	GRAB	PRIOR TO DISCHARGE TO CITY SEWER
001	NITROGEN,TOTAL KJELDAHL (AS N)	1 EVERY 3 MONTHS	GRAB	PRIOR TO DISCHARGE TO CITY SEWER
001	OIL AND GREASE	1 EVERY 3 MONTHS	GRAB	PRIOR TO DISCHARGE TO CITY SEWER
001	ZINC,TOTAL (AS ZN)	1 EVERY MONTH	GRAB	PRIOR TO DISCHARGE TO CITY SEWER
001	BETX	1 EVERY MONTH	GRAB	PRIOR TO DISCHARGE TO CITY SEWER

Facility Name: INDIANOLA CITY OF STP (NORTH)

Permit Number: 9133001

Industrial Contributor Monitoring and Reporting Requirements

Industrial Contributor: SOUTH CENTRAL IOWA LANDFILL AGENCY

(a) Samples and measurements taken shall be representative of the volume and nature of the monitored wastewater.

(b) Analytical and sampling methods specified in 40 CFR Part 136 or other methods approved in writing by the department shall be utilized.

(c) Chapter 63 of the Iowa Administrative Code provides you with further explanation of your monitoring requirements.

(d) You are required to report all data including calculated results needed to determine compliance with the limitations contained in this permit. This includes daily maximums and minimums, 30-day averages and 7-day averages for all parameters that have concentration (mg/l) and mass (lbs/day) limits. Also, flow data shall be reported in million gallons per day (MGD).

(e) Results of all monitoring shall be recorded on forms provided by, or approved by, the department, and shall be submitted to the department by the fifteenth day following the close of the reporting period. Your reporting period is on a monthly basis, ending on the last day of each reporting period.

Outfall Number	Wastewater Parameter	Sample Frequency	Sample Type	Monitoring Location
001	TOTAL PETROLEUM HYDROCARBONS	1 EVERY MONTH	GRAB	PRIOR TO DISCHARGE TO CITY SEWER
001	SANITARY LANDFILL LEACHATE	1 EVERY 12 MONTHS	GRAB	PRIOR TO DISCHARGE TO CITY SEWER

Facility Name: City of Indianola (North)
NPDES Permit Number: 91-33-0-01

ADDITIONAL MONITORING REQUIREMENTS – SOUTH CENTRAL IOWA LANDFILL AGENCY

The permittee shall analyze a representative sample of the leachate discharge from the South Central Iowa Landfill Agency at least annually for each of the pollutants listed below. Also, the permittee shall monitor the volume of waste discharged for BOD5, TSS, TKN, NH3-N, Oil & Grease, and metals at the frequencies specified on pages 8 and 9 of this permit.

Conventional Pollutants and Metals

Biochemical Oxygen Demand (BOD_5)
Total Organic Carbon
Total Dissolved Solids
Total Suspended Solids
Ammonia Nitrogen
pH
Arsenic, Total (as As)
Barium, Total (as Ba)
Cadmium, Total (as Cd)
Chromium, Total (as Cr)
Copper, Total (as Cu)
Iron, Total (as Fe)
Lead, Total (as Pb)
Mercury, Total (as Hg)
Nickel, Total (as Ni)
Selenium, Total (as Se)
Silver, Total (as Ag)
Zinc, Total (as Zn)

Volatile Compounds

Method of Analysis: EPA Methods 624 or 1624

Chloromethane (methyl chloride)
Bromomethane (methyl bromide)
Vinyl chloride
Chloroethane (ethyl chloride)
Methylene chloride (dichloromethane)
1,1-Dichloroethene (1,1-dichloroethylene)
1,1-Dichloroethane
1,2-Dichloroethene (1,2-dichloroethylene)
Chloroform
1,2-Dichloroethane
1,1,1-Trichloroethane (methyl chloroform)
Carbon tetrachloride
Bromodichloromethane
1,1,2,2-Tetrachloroethane
1,2-Dichloropropane
1,3-Dichloropropene
Trichloroethene
Dibromochloromethane
1,1,2-Trichloroethane
Benzene
2-Chloroethyl vinyl ether
Bromoform
Tetrachloroethene
Toluene
Chlorobenzene
Ethylbenzene

Acid Extractible Cmpounds

Method of Analysis: EPA Methods 625 or 1625

2-Chlorophenol
2-Nitrophenol
2,4-Dimethylphenol
Benzoic acid
2,4-Dichlorophenol
4-Chloro-3-methylphenol
2,4,6-Trichlorophenol
2,4,5-Trichlorophenol
2,4-Dinitrophenol
4-Nitrophenol
4,6-Dinitro-2-methylphenol
Pentachlorophenol

Chlorinated Hydrocarbon Insecticides

Methods of Analysis: EPA Methods 608 or 625

Beta BHC
Delta BHC
Gamma BHC
Heptachlor
Aldrin
Heptachlor epoxide
Endosulfan
Dieldrin
4,4'-DDE
Endrin
Endosulfan II
4,4'-DDD
Endosulfan sulfate
4,4'-DDT
Endrin aldehyde
Chlordane
Toxaphene

Polychlorinated Biphenyls

Methods of Analysis: EPA Methods 608 or 625

Arochlor-1016
Arochlor-1221
Arochlor-1232
Arochlor-1242
Arochlor-1248
Arochlor-1254
Arochlor-1260

Base/Neutral Compounds

Methods of Analysis: EPA Methods 625 or 1625

bis (2-chloroethyl) ether
1,3-Dichlorobenzene
1,4-Dichlorobenzene
Benzyl alcohol
1,2-Dichlorobenzene
bis (2-chloroisopropyl) ether
N-Nitroso-dipropylamine
Hexachloroethane
Nitrobenzene
Isophorone
bis (2-chloroethoxy) methane
1,2,4-Trichlorobenzene
Naphthalene
Hexachlorobutadiene
Hexachlorocyclopentadiene
2-Chloronaphthalene
Dimethyl phthalate
Acenaphthylene
Acenaphthene
Dibenzofuran
2,4-Dinitrotoluene
2,6-Dinitrotoluene
Diethyl phthalate
4-Chlorophenyl phenyl ether
Fluorene
N-Nitrosodiphenylamine
4-Bromophenyl phenyl ether
Hexachlorobenzene
Phenanthrene
Anthracene
Di-n-butyl phthalate
Fluoranthene
Pyrene
Butyl benzyl phthalate
3,3'-Dichlorobenzidine
Benzo (a) anthracene
bis (2-ethylhexyl) phthalate
Chrysene
Di-n-octyl phthalate
Benzo (b) fluoranthene
Benzo (k) fluoranthene
Benzo (a) pyrene
Indeno (1,2,3-cd) pyrene
Dibenz (a,h) anthracene
Benzo (g,h,i) perylene

Facility Name: INDIANOLA CITY OF STP (NORTH)

Permit Number: 9133001

Outfall Number: 001

Ceriodaphnia and Pimephales Toxicity Effluent Testing

1. For facilities that have not been required to conduct toxicity testing by a previous NPDES permit, the initial annual toxicity test shall be conducted within three (3) months of permit issuance. For facilities that have been required to conduct toxicity testing by a previous NPDES permit, the initial annual toxicity test shall be conducted within twelve months (12) of the last toxicity test.

2. The test organisms that are to be used for acute toxicity testing shall be Ceriodaphnia dubia and Pimephales promelas. The acute toxicity testing procedures used to demonstrate compliance with permit limits shall be those listed in 40 CFR Part 136 and adopted by reference in rule 567--63.1(1). The method for measuring acute toxicity is specified in USEPA. October 2002, Methods for Measuring the Acute Toxicity of Effluents and Receiving Waters to Freshwater and Marine Organisms, Fifth Edition. U.S. Environmental Protection Agency, Office of Water, Washington, D.C., EPA 821-R-02-012.

3. The diluted effluent sample must contain a minimum of 97.90 % effluent and no more than 2.10 % of culture water.

4. One valid positive toxicity result will require quarterly testing for effluent toxicity.

5. Two successive valid positive toxicity results or three positive results out of five successive valid effluent toxicity tests will require a toxic reduction evaluation to be completed to eliminate the toxicity.

6. A non-toxic test result shall be indicated as a "1" on the monthly operation report. A toxic test result shall be indicated as a "2" on the monthly operation report. DNR Form 542-1381 shall also be submitted to the DNR field office along with the monthly operation report.

Ceriodaphnia and Pimephales Toxicity Effluent Limits

The 30 day average mass limit of "1" for the parameters Acute Toxicity, Ceriodaphnia and Acute Toxicity, Pimephales means no positive toxicity results.

Definition: "Positive toxicity result" means a statistical difference of mortality rate between the control and the diluted effluent sample. For more information see USEPA. October 2002, Methods for Measuring the Acute Toxicity of Effluents and Receiving Waters to Freshwater and Marine Organisms, Fifth Edition, U.S. Environmental Protection Agency, Office of Water, Washington, D.C. EPA 821-R-01-012.

SLUDGE HANDLING AND DISPOSAL REQUIREMENTS

1. The permittee shall comply with all existing Federal and State laws and regulations that apply to the use and disposal of sewage sludge and with technical standards developed pursuant to Section 405(d) of the Clean Water Act when such standards are promulgated. If an applicable numerical limit or management practice for pollutants in sewage sludge is promulgated after issuance of this permit that is more stringent than a sludge pollutant limit or management practice specified in existing Federal or State laws or regulations, this permit shall be modified, or revoked and reissued, to conform to the regulations promulgated under Section 405(d) of the Clean Water Act. The permittee shall comply with the limitation no later than the compliance deadline specified in the applicable regulations.

2. The permittee shall provide written notice to the Department of Natural Resources prior to any planned changes in sludge disposal practices.

3. Land application of municipal sewage sludge shall be conducted in accordance with criteria established rule IAC 567--67.1 through 67.11(455B).

MAJOR CONTRIBUTING INDUSTRIES
LIMITATIONS, MONITORING AND REPORTING REQUIREMENTS

1. You are required to notify the department, in writing, of any of the following:

   (a) 180 days prior to the introduction of pollutants to your facility from a major contributing industry. A major contributing industry means an industrial user of a treatment works that:

      (1) Has a flow of 50,000 gallons or more per average work day;

      (2) Has a flow greater than five percent (5%) of the flow carried by the treatment works receiving the waste;

      (3) Has in its waste a toxic pollutant in toxic amounts as defined in standards issued under Section 307 (a) of the Clean Water Act and adopted by reference in Rule 62.5(455B); or

      (4) Is found by the department in connection with the issuance of an NPDES permit to have a significant impact, either alone or in combination with other contributing industries, on the treatment works or on the quality of effluent from the treatment works.

   (b) 60 days prior to a proposed expansion, production increase or process modification that may result in the discharge of a new pollutant or a discharge in excess of limitations stated in the existing treatment agreement.

   (c) 10 days prior to any commitment by you to accept waste from any new major contributing industry.

   Your written notification must include a new or revised treatment agreement in accordance with rule 64.3(5)(455B).

2. You shall require all users of your facility to comply with Sections 204(b), 307 and 308 of the Clean Water Act.

   Section 204(b) requires that all users of the treatment works constructed with funds provided under Sections 201(g) or 601 of the Act to pay their proportionate share of the costs of operation, maintenance and replacement of the treatment works.

   Section 307 of the Act requires users to comply with pretreatment standards promulgated by EPA for pollutants that would cause interference with the treatment process or would pass through the treatment works.

   Section 308 of the Act requires users to allow access at reasonable times to state and EPA inspectors for the purpose of sampling the discharge and reviewing and copying records.

3. You shall limit and monitor pollutants for each major contributing industry as required eslewhere in this permit, and submit sample results to the department monthly. Your report shall be submitted by the fifteenth day of the following month.

Revised: August 18, 1993 cwf

STANDARD CONDITIONS

1. DEFINITIONS

(a) 7 day average means the sum of the total daily discharges by mass, volume or concentration during a 7 consecutive day period, divided by the total number of days during the period that measurements were made. Four 7 consecutive day periods shall be used each month to calculate the 7-day average. The first 7-day period shall begin with the first day of the month.

(b) 30 day average means the sum of the total daily discharges by mass, volume or concentration during a calendar month, divided by the total number of days during the month that measurements were made.

(c) daily maximum means the total discharge by mass, volume or concentration during a twenty-four hour period.

2. DUTY TO COMPLY

You must comply with all conditions of this permit. Any permit noncompliance constitutes a violation of the Clean Water Act and is grounds for enforcement action; permit termination, revocation and reissuance, or modification; or denial of a permit renewal application. Issuance of this permit does not relieve you of the responsibility to comply with all local, state and federal laws, ordinances, regulations or other legal requirements applying to the operation of your facility.
{See 40 CFR 122.41(a) and 567-64.3(11) IAC}

3. DUTY TO REAPPLY

If you wish to continue to discharge after the expiration date of this permit you must file an application for reissuance at least 180 days prior to the expiration date of this permit.
{See 567-64.8(1) IAC}

4. NEED TO HALT OR REDUCE ACTIVITY

It shall not be a defense for a permittee in an enforcement action that it would have been necessary to halt or reduce the permitted activity in order to maintain compliance with the conditions of this permit.
{See 567-64.7(5)(j) IAC}

5. DUTY TO MITIGATE

You shall take all reasonable steps to minimize or prevent any discharge in violation of this permit which has a reasonable likelihood of adversely affecting human health or the environment.
{See 567-64.7(5)(i) IAC}

6. PROPERTY RIGHTS

This permit does not convey any property rights of any sort or any exclusive privileges.

7. TRANSFER OF TITLE

If title to your facility, or any part of it, is transferred the new owner shall be subject to this permit.
{See 567-64.14 IAC}

You are required to notify the new owner of the requirements of this permit in writing prior to any transfer of title. The Director shall be notified in writing within 30 days of the transfer

8. PROPER OPERATION AND MAINTENANCE

All facilities and control systems shall be operated as efficiently as possible and maintained in good working order. A sufficient number of staff, adequately trained and knowledgeable in the operation of your facility shall be retained at all times and adequate laboratory controls and appropriate quality assurance procedures shall be provided to maintain compliance with the conditions of this permit.
{See 40 CFR 122.41(e) and 567 64.7(5)(f) IAC}

9. DUTY TO PROVIDE INFORMATION

You must furnish to the Director, within a reasonable time, any information the Director may request to determine whether cause exists for modifying, revoking and reissuing, or terminating this permit or to determine compliance with this permit. You must also furnish to the Director, upon request, copies of any records required to be kept by this permit.

10. MAINTENANCE OF RECORDS

You are required to maintain records of your operation in accordance with 567-63.2 IAC.

11. PERMIT MODIFICATION, SUSPENSION OR REVOCATION

(a) This permit may be modified, suspended, or revoked and reissued for cause including but not limited to those specified in 567-64.3(11) IAC.

(b) This permit may be modified due to conditions or information on which this permit is based, including any new standard the department may adopt that would change the required effluent limits.
{See 567-64.3(11)© IAC}

(c) If a toxic pollutant is present in your discharge and more stringent standards for toxic pollutants are established under Section 307(a) of the Clean Water Act, this permit will be modified in accordance with the new standards.
{See 567-64.7(5)(g) IAC}

The filing of a request for a permit modification, revocation or suspension, or a notification of planned changes or anticipated noncompliance does not stay any permit condition.

12. SEVERABILITY

The provisions of this permit are severable and if any provision or application of any provision to any circumstance is found to be invalid by this department or a court of law, the application of such provision to other circumstances, and the remainder of this permit, shall not be affected by such finding.

13. **INSPECTION OF PREMISES, RECORDS, EQUIPMENT, METHODS AND DISCHARGES**
You are required to permit authorized personnel to:

(a) Enter upon the premises where a regulated facility or activity is located or conducted or where records are kept under conditions of this permit.

(b) Have access to and copy, at reasonable times, any records that must be kept under the conditions of this permit.

(c) Inspect, at reasonable times, any facilities, equipment, practices or operations regulated or required under this permit.

(d) Sample or monitor, at reasonable times, for the purpose of assuring compliance or as otherwise authorized by the Clean Water Act.

14. **TWENTY-FOUR HOUR REPORTING**
You shall report any noncompliance that may endanger human health or the environment. Information shall be provided orally within 24 hours from the time you become aware of the circumstances. A written submission that includes a description of noncompliance and its cause; the period of noncompliance including exact dates and times, whether the noncompliance has been corrected or the anticipated time it is expected to continue; and the steps taken or planned to reduce, eliminate, and prevent a reoccurrence of the noncompliance must be provided within 5 days of the occurrence. The following instances of noncompliance must be reported within 24 hours of occurrence:

(a) Any unanticipated bypass which exceeds any effluent limitation in the permit.
{See 40 CFR 122.44(g)}

(b) Any upset which exceeds any effluent limitation in the permit.
{See 40 CFR 122.44(n)}

(c) Any violation of a maximum daily discharge limit for any of the pollutants listed by the Director in the permit to be reported within 24 hours.
{See 40 CFR 122.44(g)}

15. **OTHER NONCOMPLIANCE**
You shall report all instances of noncompliance not reported under Condition #14 at the time monitoring reports are submitted.

16. **ADMINISTRATIVE RULES**
Rules of this Department which govern the operation of your facility in connection with this permit are published in Part 567 of the Iowa Administrative Code (IAC) in Chapters 60-64 and 120-122. Reference to the term "rule" in this permit means the designated provision of Part 567 of the Iowa Administrative Code.

17. **NOTICE OF CHANGED CONDITIONS**
You are required to report any changes in existing conditions or information on which this permit is based:

(a) Facility expansions, production increases or process modifications which may result in new or increased discharges of pollutants must be reported to the Director in advance. If such discharges will exceed effluent limitations, your report must include an application for a new permit.
{See 567-64.7(5)(a) IAC}

(b) If any modification of, addition to, or construction of a disposal system is to be made, you must first obtain a written permit from this Department.
{See 567-64.2 IAC}

(c) If your facility is a publicly owned treatment works or otherwise may accept waste for treatment from industrial contributors see 567-64.3(5) IAC for further notice requirements.

(d) You shall notify the Director as soon as you know or have reason to believe that any activity has occurred or will occur which would result in the discharge of any toxic pollutant which is not limited in this permit.
{See 40 CFR 122.42(a)}

You must also notify the Director if you have begun or will begin to use or manufacture as an intermediate or final product or byproduct any toxic pollutant which was not reported in the permit application

18. **OTHER INFORMATION**
Where you become aware that you failed to submit any relevant facts in a permit application, or submitted incorrect information in a permit application or in any report, you must promptly submit such facts or information.

19. UPSET PROVISION

(a) Definition - "Upset" means an exceptional incident in which there is unintentional and temporary noncompliance with technology based permit effluent limitations because of factors beyond the reasonable control of the permittee. An upset does not include noncompliance to the extent caused by operational error, improperly designed treatment facilities, inadequate treatment facilities, lack of preventive maintenance, or careless or improper operation.

(b) Effect of an upset. An upset constitutes an affirmative defense in an action brought for noncompliance with such technology based permit effluent limitations if the requirements of paragraph *"c"* of this condition are met. No determination made during administrative review of claims that noncompliance was caused by upset, and before an action for noncompliance, is final administrative action subject to judicial review.

(c) Conditions necessary for demonstration of an upset. A permittee who wishes to establish the affirmative defense of upset shall demonstrate through properly signed, contemporaneous operating logs, or other relevant evidence that;

(1) An upset occurred and that the permittee can identify the cause(s) of the upset.

(2) The permitted facility was at the time being properly operated; and

(3) The permittee submitted notice of the upset to the Department in accordance with 40 CFR 122.41(l)(6)(ii)(B).

(4) The permittee complied with any remedial measures required by Item #5 of the Standard Conditions of this permit.

(d) Burden of Proof. In any enforcement proceeding, the permittee seeking to establish the occurrence of an upset has the burden of proof.

20. FAILURE TO SUBMIT FEES

This permit may be revoked, in whole or in part, if the appropriate permit fees are not submitted within thirty (30) days of the date of notification that such fees are due.

21. BYPASSES

(a) Definition - Bypass means the intentional diversion of waste streams from any portion of a treatment facility.

(b) Prohibition of bypass, Bypass is prohibited and the department may take enforcement action against a permittee for bypass unless:

(1) Bypass was unavoidable to prevent loss of life, personal injury, or severe property damage;

(2) There were no feasible alternatives to the bypass, such as the use of auxiliary treatment facilities, retention of untreated wastes, or maintenance during normal periods of equipment downtime. This condition is not satisfied if adequate backup equipment should have been installed in the exercise of reasonable engineering judgement to prevent a bypass which occurred during normal periods of equipment downtime or preventive maintenance;

(3) The permittee submitted notices as required by paragraph *"d"* of this section.

(c) The Director may approve an anticipated bypass after considering its adverse effects if the Director determines that it will meet the three conditions listed above.

(d) Reporting bypasses. Bypasses shall be reported in accordance with 567-63.6 IAC.

22. SIGNATORY REQUIREMENTS

Applications, reports or other information submitted to the Department in connection with this permit must be signed and certified as required by 567-64.3(8) IAC.

23. USE OF CERTIFIED LABORATORIES

Effective October 1, 1996, analyses of wastewater, groundwater or sewage sludge that are required to be submitted to the department as a result of this permit must be performed by a laboratory certified by the State of Iowa. Routine, on-site monitoring for pH, temperature, dissolved oxygen, total residual chlorine and other pollutants that must be analyzed immediately upon sample collection, settleable solids, physical measurements, and operational monitoring tests specified in 567-63.3(4) are excluded from this requirement.

APPENDIX B
Waste Load Allocation – Cavitt Creek and Middle River

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. They are currently considering two different outfall locations. 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:
Downstream of the proposed outfall, the Middle River is an A1, B(WW-1) HH designated use waterbody. Downstream of the mouth of the Middle River, the Des Moines River is an A1, B(WW-1) HH designated use waterbody before switching to an A1, B(WW-1) HH Class C designated use waterbody due to 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 (or just upstream of) the proposed discharge point are estimated based on the drainage area ratio method and flow statistics obtained at USGS gage station 05486490, which is located approximately one mile downstream of the proposed discharge point on the Middle River near Indianola, Iowa. The drainage area at the proposed discharge point was found using DEM data (WLA GIS Tool) and adjusted based on the drainage area of the nearby USGS gage 05486490.

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
USGS Gage 05486490 (Middle River near Indianola, IA)	503	20.8$^{\$}$	1.200$^{\$}$	1.600$^{\$}$	2.800$^{\$}$
The Middle River at (or just upstream of) the proposed outfall	501.25	20.7$^{@}$	1.196$^{@}$	1.594$^{@}$	2.790$^{@}$

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

Downstream of the mouth of the Middle River, the Class C segment of the Des Moines River begins at the mouth of Cedar Creek. The annual critical low flows in the Des Moines River at (or just upstream of) the mouth of Cedar Creek are determined so that the limits for the protection of the Class C segment 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 (Des Moines River near Tracy, IA)	12,479	1,670$	221.000$	249.000$	310.000$
The Des Moines River at (or just upstream of) the mouth of Cedar Creek	12,503.56	1,673.3@	221.435@	249.490@	310.610@

$: 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 ☐	1: Existing design capacity sheets are attached (supporting document and permit rationale for the current NPDES permit) 2: Proposed design capacity shown on the request form
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 Permit limits?	Yes ☒, No ☐, or Not Applicable ☐	1: Current limits sheet attached
5	Outfall Location Change	Yes ☒, No ☐, or Not Applicable ☐	
Conclusion and discussion: Due to Items 1, 4, and 5, a tier II antidegradation review is required.			

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 that TMDL, the Indianola wastewater treatment facility was assigned *E. coli* wasteload allocations, as discussed in the *E. coli* section below. There are no TMDLs currently scheduled for segments in the route of flow.

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/Environment/WaterQuality/WatershedImprovement/WatershedResearchData.aspx.

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

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.

To protect the downstream Class C use:
The Middle River enters the Des Moines River over 30 miles upstream of the beginning of the Des Moines River Class C stream segment; therefore, the Des Moines River is assumed to be fully mixed at the beginning of the Class C stream segment.

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 mouth of Cedar Creek.

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 mouth of Cedar Creek.

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. Except for chloride and sulfate (discussed below), 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	22.2	0.5
October	8	12.3	0.5
November	8.1	6	0.5
December	8	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	8.8	15.3	6.6
February	14.4	10.0	14.3	7.4
March	14.9	4.6	14.8	3.5
April	15.8	3.3	15.8	2.6
May	15.3	2.9	15.2	2.2
June	14.6	2.0	14.5	1.6
July	17.7	1.9	17.6	1.4
August	16.4	1.8	16.3	1.3
September	16.6	2.3	16.6	1.8
October	15.9	4.6	15.8	3.5
November	14.8	5.7	14.7	4.3
December	16.1	6.7	16.0	5.0

*: 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	8.8	**14.6**	6.6
February	14.4	10.0	14.3	7.4
March	14.9	4.6	**12.9**	3.5
April	**12.9**	3.3	**8.4**	2.6
May	**8.6**	2.9	**5.6**	2.2
June	**5.8**	2.0	**3.8**	1.6
July	**4.4**	1.9	**2.9**	1.4
August	**4.6**	1.8	**2.9**	1.3
September	**5.2**	2.3	**3.5**	1.8
October	**9.5**	4.6	**5.7**	3.5
November	14.8	5.7	**8.5**	4.3
December	16.1	6.7	**12.2**	5.0

*: **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.

Field Use Attainability Assessment (UAA) had three sites along the Middle River downstream of the proposed outfall. Two observations of stream width, average depth, and velocity were made at each site. Based on these UAA data, the stream average width, depth, and velocity at annual 7Q10 + ADW and annual 7Q10 + AWW conditions are estimated and are shown in Table 6. The spreadsheet for the estimate is attached.

Table 6: Stream Width, Depth and Velocity

Flow condition	Flow (cfs)	Width (ft)	Depth (in)	Velocity (fps)
Annual 7Q10 + ADW	5.153	46.9	5.86	0.23
Annual 7Q10 + AWW	10.737	51.2	8.15	0.31

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.

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, the recent chapter 62 revision that became effective on Oct. 14, 2009 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 to this facility.

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 both based on an actual sampling frequency of 1/month, although a monitoring frequency of 4/month was used in the calculations.

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:
The criteria apply at the end of the ZID. The ratio is calculated using ADW design flow and 2.5% of 1Q10 as the ZID. Therefore, use 99.2% of effluent and 0.8% of dilution water for the testing.

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.

Jan. 21, 2016 *NPDES # 6- 91-33-0-01*

Indianola, City of STP (North)

Proposed new outfall on Cavitt Creek

(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	5.2	**8.7**	254.7	**423.6**	--
February	5.8	**9.9**	286.4	**482.9**	--
March	4.5	**8.0**	187.6	**388.2**	--
April	2.1	**5.6**	103.4	**267.1**	--
May	1.8	**3.7**	90.4	**178.5**	--
June	1.3	**2.5**	66.3	**121.7**	--
July	1.1	**1.9**	53.3	**88.8**	--
August	1.0	**1.8**	48.6	**85.6**	--
September	1.5	**2.2**	73.1	**111.0**	--
October	2.8	**3.3**	138.4	**157.6**	--
November	3.4	**5.0**	167.8	**240.9**	--
December	4.0	**7.2**	194.8	**347.2**	--
Bacteria	Geometric Mean (#org/100 ml)		March 15th – November 15th		--
*E. coli***	126				
Chloride***	389	629	19,156	30,996	1/ month
Sulfate***	1,514	1,514	74,609	74,609	1/ month
TRC****	0.0078	0.0190	0.387	0.936	5/week
pH	6.5 – 9.0 Standard Units				--

For the major facility acute WET testing, use 100% of effluent and 0% of dilution water

Stream Network/Classification of Receiving Stream: Cavitt Creek (A2, B(WW-2) to A3, B(WW-2) to A2, B(WW-2)) to the Middle River (A1, B(WW-1) HH) to the Des Moines River (A1, B(WW-1) HH to A1, B(WW-1) HH Class C)

Date Done: Jan. 21, 2016

Annual critical low flow in Cavitt Creek at (or just upstream of) the proposed outfall
30Q10 flow 0 cfs, 7Q10 flow 0 cfs, 1Q10 flow 0 cfs

Annual critical low flow in the Middle River at (or just upstream of) the mouth of Cavitt Creek
30Q10 flow 2.808 cfs, 7Q10 flow 1.605 cfs, 1Q10 flow 1.204 cfs, harmonic mean flow 20.9

Annual critical low flow in the Des Moines River at (or just upstream of) the mouth of Cedar Creek
30Q10 flow 310.610 cfs, 7Q10 flow 249.490 cfs, 1Q10 flow 221.435 cfs, harmonic mean flow 1,673.3 cfs

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

Performed by: Ian Paul Willard Approved by: Connie Dou

* **Bold** values are governed by CBOD5/DO modeling, while the others are based on ammonia nitrogen toxicity.
** Due to a recent revision to IAC567.62 (Chapter 62), sample maximum limit for bacteria is no longer required. Only geometric mean is required.
*** Chloride/sulfate limits are based on the new chloride/sulfate criteria that took effective on Nov. 11, 2009. Chloride/sulfate criteria are hardness dependent and the default hardness has been changed from 100 mg/l to 200 mg/l, effective Nov. 11, 2009.
**** 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.

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	1.422E+01	2.640E+01	2.789E+02	1.301E+03	1/week
1,1,2-Trichloroethane	2.828E+00	2.828E+00	5.442E+01	5.442E+01	1/week
1,1-Dichloroethylene	1.753E+01	5.400E+01	5.500E+02	2.662E+03	1/week
1,2,4-Trichlorobenzene	4.978E+00	4.978E+00	9.760E+01	9.760E+01	1/week
1,2-Dichloroethane	9.133E-01	5.900E+01	2.866E+01	2.908E+03	1/week
1,2-Dichloropropane	3.703E-01	3.703E-01	1.162E+01	1.162E+01	1/week
2,3,7,8-TCDD (Dioxin)	1.259E-10	1.259E-10	3.950E-09	3.950E-09	1/week
2,4,5-TP (Silvex)	7.112E-01	7.112E-01	1.394E+01	1.394E+01	1/week
2,4-D	7.112E+00	7.112E+00	1.394E+02	1.394E+02	1/week
3,3-Dichlorobenzidine	6.912E-04	6.912E-04	2.169E-02	2.169E-02	1/week
4,4' DDT	1.000E-06	1.100E-03	4.929E-05	5.422E-02	1/week
Alachlor	1.422E-01	1.422E-01	2.789E+00	2.789E+00	1/week
Aldrin	1.234E-06	3.000E-03	3.873E-05	1.479E-01	1/week
Aluminum	8.700E-02	7.500E-01	4.288E+00	3.697E+01	1/week
Antimony	3.983E-01	1.100E+01	7.808E+00	5.422E+02	1/week
Arsenic (III)	8.483E-02	3.400E-01	1.633E+00	1.676E+01	1/week
Asbestos	4.978E-01	4.978E-01	9.760E+00	9.760E+00	1/week
Atrazine	2.134E-01	2.134E-01	4.183E+00	4.183E+00	1/week
Barium	7.112E+01	2.050E+02	1.394E+03	1.010E+04	1/week
Benzene	1.259E+00	1.650E+01	3.950E+01	8.133E+02	1/week
Benzo(a)Pyrene	4.443E-04	4.443E-04	1.394E-02	1.394E-02	1/week
Beryllium	2.845E-01	5.000E-01	5.577E+00	2.464E+01	1/week
Bromoform	3.456E+00	3.456E+00	1.084E+02	1.084E+02	1/week
Cadmium	4.523E-04	4.316E-03	2.229E-02	2.127E-01	1/week
Carbofuran	2.845E+00	2.845E+00	5.577E+01	5.577E+01	1/week
Carbon Tetrachloride	3.950E-02	2.155E+01	1.239E+00	1.062E+03	1/week
Chlordane	4.300E-06	2.400E-03	2.119E-04	1.183E-01	1/week
Chloride	3.89E+02	6.29E+02	1.9156E+04	3.0996E+04	1/month
Chlorobenzene	1.780E+00	1.610E+01	8.232E+01	7.936E+02	1/week
Chlorodibromomethane	3.209E-01	3.209E-01	1.007E+01	1.007E+01	1/week
Chloroform	1.160E+01	1.160E+01	3.641E+02	3.641E+02	1/week
Chloropyrifos	4.100E-05	8.300E-05	2.021E-03	4.091E-03	1/week
Chromium (VI)	1.100E-02	1.600E-02	5.422E-01	7.886E-01	1/week
cis-1,2-Dichloroethylene	4.978E+00	4.978E+00	9.760E+01	9.760E+01	1/week
Copper	1.687E-02	2.690E-02	8.314E-01	1.326E+00	1/week
Cyanide	5.200E-03	2.200E-02	2.563E-01	1.084E+00	1/week
Dalapon	1.422E+01	1.422E+01	2.789E+02	2.789E+02	1/week
Di(2-ethylhexyl)adipate	2.845E+01	2.845E+01	5.577E+02	5.577E+02	1/week
Bis(2-ethylhexyl)phthalate	2.448E-02	2.448E-02	1.132E+00	1.132E+00	1/week
Dibromochloropropane	1.422E-02	1.422E-02	2.789E-01	2.789E-01	1/week
Dichlorobromomethane	4.196E-01	4.196E-01	1.317E+01	1.317E+01	1/week

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	3.556E-01	3.556E-01	6.972E+00	6.972E+00	1/week
Dieldrin	1.333E-06	2.400E-04	4.183E-05	1.183E-02	1/week
Dinoseb	4.978E-01	4.978E-01	9.760E+00	9.760E+00	1/week
Diquat	1.422E+00	1.422E+00	2.789E+01	2.789E+01	1/week
Endosulfan	5.600E-05	2.200E-04	2.760E-03	1.084E-02	1/week
Endothall	7.112E+00	7.112E+00	1.394E+02	1.394E+02	1/week
Endrin	3.600E-05	8.600E-05	1.774E-03	4.239E-03	1/week
Ethylbenzene	2.337E+00	2.265E+01	1.081E+02	1.116E+03	1/week
Ethylene dibromide	3.556E-03	3.556E-03	6.972E-02	6.972E-02	1/week
Fluoride	8.077E+00	8.077E+00	3.981E+02	3.981E+02	1/week
gamma-Hexachlorocyclohexane (Lindane)	9.500E-04	9.500E-04	4.682E-02	4.682E-02	1/week
Glyphosate	4.978E+01	4.978E+01	9.760E+02	9.760E+02	1/week
Heptachlor	1.950E-06	5.200E-04	6.119E-05	2.563E-02	1/week
Heptachlor epoxide	9.627E-07	5.200E-04	3.021E-05	2.563E-02	1/week
Hexachlorobenzene	7.159E-06	7.159E-06	2.246E-04	2.246E-04	1/week
Hexachlorocyclopentadiene	1.224E+00	1.224E+00	5.577E+01	5.577E+01	1/week
Iron	1.000E+00	1.000E+00	4.929E+01	4.929E+01	1/week
Lead	7.693E-03	1.974E-01	3.792E-01	9.730E+00	1/week
Mercury (II)	1.669E-04	1.640E-03	7.718E-03	8.083E-02	1/week
Methoxychlor	7.112E+00	7.112E+00	1.394E+02	1.394E+02	1/week
Nickel	9.376E-02	8.433E-01	4.622E+00	4.157E+01	1/week
Nitrate as N	3.200E+02	3.200E+02	1.394E+04	1.577E+04	1/week
Nitrate+Nitrite as N	3.200E+02	3.200E+02	1.394E+04	1.577E+04	1/week
Nitrite as N	7.112E+01	7.112E+01	1.394E+03	1.394E+03	1/week
o-Dichlorobenzene	4.267E+01	4.267E+01	8.366E+02	8.366E+02	1/week
Oxamyl (Vydate)	1.422E+01	1.422E+01	2.789E+02	2.789E+02	1/week
para-Dichlorobenzene	2.114E-01	2.000E+00	9.776E+00	9.858E+01	1/week
Parathion	1.300E-05	6.500E-05	6.408E-04	3.204E-03	1/week
Pentachlorophenol (PCP)	1.828E-02	2.383E-02	9.012E-01	1.175E+00	1/week
Phenols	5.000E-02	2.500E+00	2.464E+00	1.232E+02	1/week
Picloram	3.556E+01	3.556E+01	6.972E+02	6.972E+02	1/week
Polychlorinated Biphenyls (PCBs)	1.580E-06	2.000E-03	4.957E-05	9.858E-02	1/week
Polynuclear Aromatic Hydrocarbons (PAHs)	3.338E-05	3.000E-02	1.544E-03	1.479E+00	1/week
Selenium	5.000E-03	1.930E-02	2.464E-01	9.513E-01	1/week
Silver	3.800E-03	3.800E-03	1.873E-01	1.873E-01	1/week
Simazine	2.845E-01	2.845E-01	5.577E+00	5.577E+00	1/week
Styrene	7.112E+00	7.112E+00	1.394E+02	1.394E+02	1/week
Sulfate	1.514E+03	1.514E+03	7.4609E+04	7.4609E+04	1/month

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	3.672E-02	3.672E-02	1.698E+00	1.698E+00	1/week
Thallium	5.230E-04	5.980E-01	2.418E-02	2.948E+01	1/week
Toluene	5.564E-02	2.521E+00	2.573E+00	1.236E+02	1/week
Total Residual Chlorine (TRC)	7.8E-03	1.90E-02	3.87E-01	9.36E-01	5/week
Toxaphene	2.000E-06	7.300E-04	9.858E-05	3.598E-02	1/week
trans-1,2-Dichloroethylene	1.558E-01	1.558E-01	7.203E+00	7.203E+00	1/week
Trichloroethylene (TCE)	8.000E-02	4.000E+00	3.943E+00	1.972E+02	1/week
Trihalomethanes (total)	5.690E+00	5.690E+00	1.115E+02	1.115E+02	1/week
Vinyl Chloride	1.778E-02	1.778E-02	1.235E+00	1.235E+00	1/week
Xylenes (Total)	7.112E+02	7.112E+02	1.394E+04	1.394E+04	1/week
Zinc	2.156E-01	2.156E-01	1.063E+01	1.063E+01	1/week

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. They are currently considering two different outfall locations. This wasteload allocation is for a proposed outfall into Cavitt Creek (at 41° 24' 54" N, 93° 35' 41" W).

Route of Flow and Use Designations:
Downstream of the proposed outfall, Cavitt Creek is an A2, B(WW-2) designated use waterbody before switching to an A3, B(WW-2) designated use waterbody and then back to an A2, B(WW-2) designated use waterbody. Downstream of the mouth of Cavitt Creek, the Middle River is an A1, B(WW-1) HH designated use waterbody. Downstream of the mouth of the Middle River, the Des Moines River is an A1, B(WW-1) HH designated use waterbody before switching to an A1, B(WW-1) HH Class C designated use waterbody due to 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 7Q10 in Cavitt Creek at (or just upstream of) the proposed discharge point is estimated by multiplying its Plate 4 7Q10 coefficient with its drainage area. Because the Plate 4 7Q10 coefficient is zero, the annual critical 7Q10 is zero. In cases where the annual critical 7Q10 is not zero, the 7Q10 ratio method is used (using data from a streamgage) to determine the annual critical 1Q10 and 30Q10. However, because the annual critical 7Q10 is zero, a streamgage does not need to be used to determine that the annual critical 1Q10 and 30Q10 will also be zero.

Table 1a: Annual Critical Low Flows in Cavitt Creek

Location	7Q10 Coefficient in Plate 4 (cfs/mi^2)	Drainage Area (mi^2)	Annual 7Q10 (cfs)	Annual critical low flows (cfs)		
				1Q10	7Q10	30Q10
Cavitt Creek at (or just upstream of) the proposed outfall	0	8.92	0*	0	0*	0

*: Estimated based on 7Q10 coefficient in Plate 4 and drainage area values

At the mouth of Cavitt Creek, the Middle River has a Class HH (human health) designation. The annual critical low flows in the Middle River at (or just upstream of) the mouth of Cavitt Creek are determined so that the limits for the protection of the A1, B(WW-1) HH segment of the Middle River can be calculated.

The annual critical low flows in the Middle River at (or just upstream of) the mouth of Cavitt Creek are estimated based on the drainage area ratio method and flow statistics obtained at USGS gage station 05486490, which is located approximately 1,500 ft upstream of the mouth of Cavitt Creek on the Middle River near Indianola, Iowa. The drainage area at the mouth of Cavitt Creek was found using DEM data (WLA GIS Tool) and adjusted based on the drainage area of the nearby USGS gage 05486490.

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

Location	Drainage Area (mi^2)	Harmonic Mean (cfs)	Annual critical low flows (cfs)		
			1Q10	7Q10	30Q10
USGS Gage 05486490 (Middle River near Indianola, IA)	503	20.8$	1.200$	1.600$	2.800$
The Middle River at (or just upstream of) the mouth of Cavitt Creek	504.52	20.9@	1.204@	1.605@	2.808@

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

Downstream of the mouth of the Middle River, the Class C segment of the Des Moines River begins at the mouth of Cedar Creek. The annual critical low flows in the Des Moines River at (or just upstream of) the mouth of Cedar Creek are determined so that the limits for the protection of the Class C segment 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 1c: 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 (Des Moines River near Tracy, IA)	12,479	1,670$	221.000$	249.000$	310.000$
The Des Moines River at (or just upstream of) the mouth of Cedar Creek	12,503.56	1,673.3@	221.435@	249.490@	310.610@

$: 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 ☐	1: Existing design capacity sheets are attached (supporting document and permit rationale for the current NPDES permit) 2: Proposed design capacity shown on the request form
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 Permit limits?	Yes ☒, No ☐, or Not Applicable ☐	1: Current limits sheet attached
5	Outfall Location Change	Yes ☒, No ☐, or Not Applicable ☐	
Conclusion and discussion: Due to Items 1, 4, and 5, a tier II antidegradation review is required.			

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 that TMDL, the Indianola wastewater treatment facility was assigned *E. coli* wasteload allocations, as discussed in the *E. coli* section below. There are no TMDLs currently scheduled for segments in the route of flow.

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/Environment/WaterQuality/WatershedImprovement/WatershedResearchData.aspx.

4. CALCULATIONS:

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

<u>To protect the aquatic life use</u>:
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, since the annual critical low flows in the receiving stream (Cavitt Creek) are all zero, the criteria apply at “end-of-pipe” instead of the end of the Mixing Zone (MZ) and Zone of Initial Dilution (ZID).

<u>To protect the downstream Class HH use</u>:
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 mouth of Cavitt Creek.

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 in the Middle River at mouth of Cavitt Creek.

<u>To protect the downstream Class C use</u>:
The Middle River enters the Des Moines River over 30 miles upstream of the beginning of the Des Moines River Class C stream segment; therefore, the Des Moines River is assumed to be fully mixed at the beginning of the Class C stream segment.

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 mouth of Cedar Creek.

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 mouth of Cedar Creek.

<u>Final limits</u>:
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. Except for chloride and sulfate (discussed below), 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 Cavitt Creek. Since the annual critical low flows in the receiving stream are all zero, the criteria apply at "end-of-pipe" instead of the end of the MZ and the ZID. Cavitt Creek is a B(WW-2) stream; therefore, early life protection will begin in April and run through September.

Because the Middle River is an A1, B(WW-1) HH stream at the mouth of Cavitt Creek, the wasteload allocations for the protection of the Middle River are also calculated. By the time the effluent reaches the Middle River, it is assumed to be at equilibrium with the environment; therefore, standard stream background temperatures, pH, and concentrations of NH3-N are used. Ammonia nitrogen decay in Cavitt Creek from the proposed outfall to its mouth (11,026 ft) is also considered, using flow velocities of 0.29 fps for annual 7Q10 + ADW conditions and 0.44 fps for annual 7Q10 + AWW conditions (as discussed in the CBOD5/Total Dissolved Oxygen section below). Based on the ratio of the stream flow to the discharging flow, 5% of the 1Q10 and 100% of the 30Q10 flow in the Middle River at (or just upstream of) the mouth of Cavitt Creek 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 wasteload allocations for the protection of Cavitt Creek and the Middle River were then compared and the more stringent values were selected (Table 5a) and used in CBOD5/DO modeling.

The monthly background pH, temperatures, 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	22.2	0.5
October	8	12.3	0.5
November	8.1	6	0.5
December	8	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	13.5	5.2	13.0	5.2
February	14.2	5.8	14.2	5.8
March	11.7	4.5	11.1	3.8
April	8.9	2.1	8.1	2.1
May	9.9	1.8	8.7	1.8
June	11.5	1.3	9.7	1.3
July	13.5	1.1	10.7	1.1
August	10.6	1.0	8.5	1.0
September	14.4	1.5	12.0	1.5
October	10.8	2.8	9.9	2.8
November	8.2	3.4	7.7	3.4
December	9.5	4.0	9.0	4.0

*: 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	**8.7**	5.2	**8.6**	5.2
February	**9.9**	5.8	**9.8**	5.8
March	**8.0**	4.5	**7.9**	3.8
April	**5.6**	2.1	**5.4**	2.1
May	**3.7**	1.8	**3.6**	1.8
June	**2.5**	1.3	**2.5**	1.3
July	**1.9**	1.1	**1.8**	1.1
August	**1.8**	1.0	**1.7**	1.0
September	**2.2**	1.5	**2.3**	1.5
October	**3.3**	2.8	**3.2**	2.8
November	**5.0**	3.4	**4.9**	3.4
December	**7.2**	4.0	**7.0**	4.0

*: **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 0.00098 (the water channel elevation changes from 792 ft to 786 ft over a distance of approximately 6,140 ft), estimated based on the GIS LiDAR 2-ft contour coverage.

Field Use Attainability Assessment (UAA) had one site along Cavitt Creek downstream of the proposed outfall. Two observations of stream width, average depth, and velocity were made at the site. Based on these UAA data, the stream average width, depth, and velocity at annual 7Q10 + ADW and annual 7Q10 + AWW conditions are estimated and are shown in Table 6. The spreadsheet for the estimate is attached.

Table 6: Stream Width, Depth and Velocity

Flow condition	Flow (cfs)	Width (ft)	Depth (in)	Velocity (fps)
Annual 7Q10 + ADW	3.558	13.0	11.35	0.29
Annual 7Q10 + AWW	9.143	14.5	17.35	0.44

Reaeration:
Based on aerial imagery, Cavitt Creek appears to be fairly meandering downstream of the proposed outfall. Therefore, the USGS pool-riffle model (Melching and Flores 1999) is used.

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 (A2) water body. The water quality standard for *E. coli* in a Class (A2) water body is a geometric mean of 630 org./100 ml and a sample maximum of 2,880 org./100 ml from March 15th through November 15th. The criteria apply at “end-of-pipe”.

Additionally, approximately 2,000 ft downstream of the proposed outfall, Cavitt Creek switches from a Class (A2) designation to a Class (A3) designation. The water quality standard for *E. coli* in a Class (A3) 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. *E. coli* decay in the Class (A2) stream segment was calculated in order to determine the effluent limits necessary to meet the downstream Class (A3) water quality standard. A flow velocity of 0.44 fps for annual 7Q10 + AWW conditions (as discussed in the CBOD5/Total Dissolved Oxygen section above) and k = 5.28/day were used in the decay calculations. In order to meet the downstream Class (A3) *E. coli* standards, a geometric mean of 166 org./100 ml and a sample maximum of 309 org./100 ml are allowed in the effluent at the proposed outfall and apply at “end-of-pipe”.

Furthermore, 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”. Because these values are the most stringent, they govern the final limits.

However, the recent chapter 62 revision that became effective on Oct. 14, 2009 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 to this facility.

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, since the critical low flows in the receiving stream are all zero, the criteria apply at "end-of-pipe" instead of the boundaries of the MZ and the ZID.

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 both based on an actual sampling frequency of 1/month, although a monitoring frequency of 4/month was used in the calculations.

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, since the annual critical low flows in the receiving stream are all zero, the criterion applies at "end-of-pipe" instead of at the end of the ZID.

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, which in this case is not available since the critical low flows in the receiving stream are all zero. The criteria will apply at "end-of-pipe".

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:
The criteria apply at the end of the ZID, which in this case is not available since the critical low flows in the receiving stream are all zero. The criteria will apply at "end-of-pipe". Therefore, use 100% effluent for the major facility acute WET testing.

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 C
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.

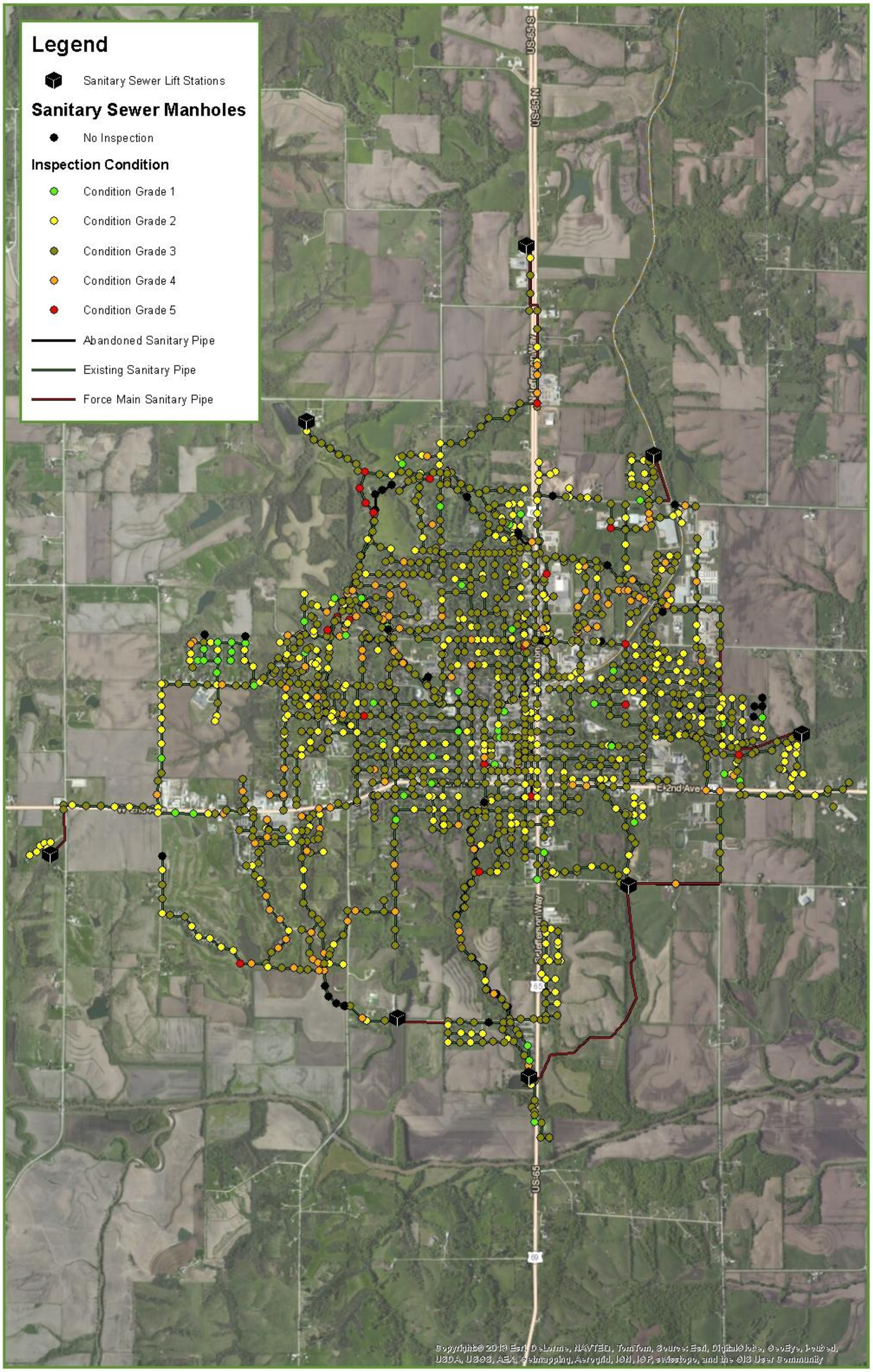

Figure 1: Manhole Condition Assessment Map

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.

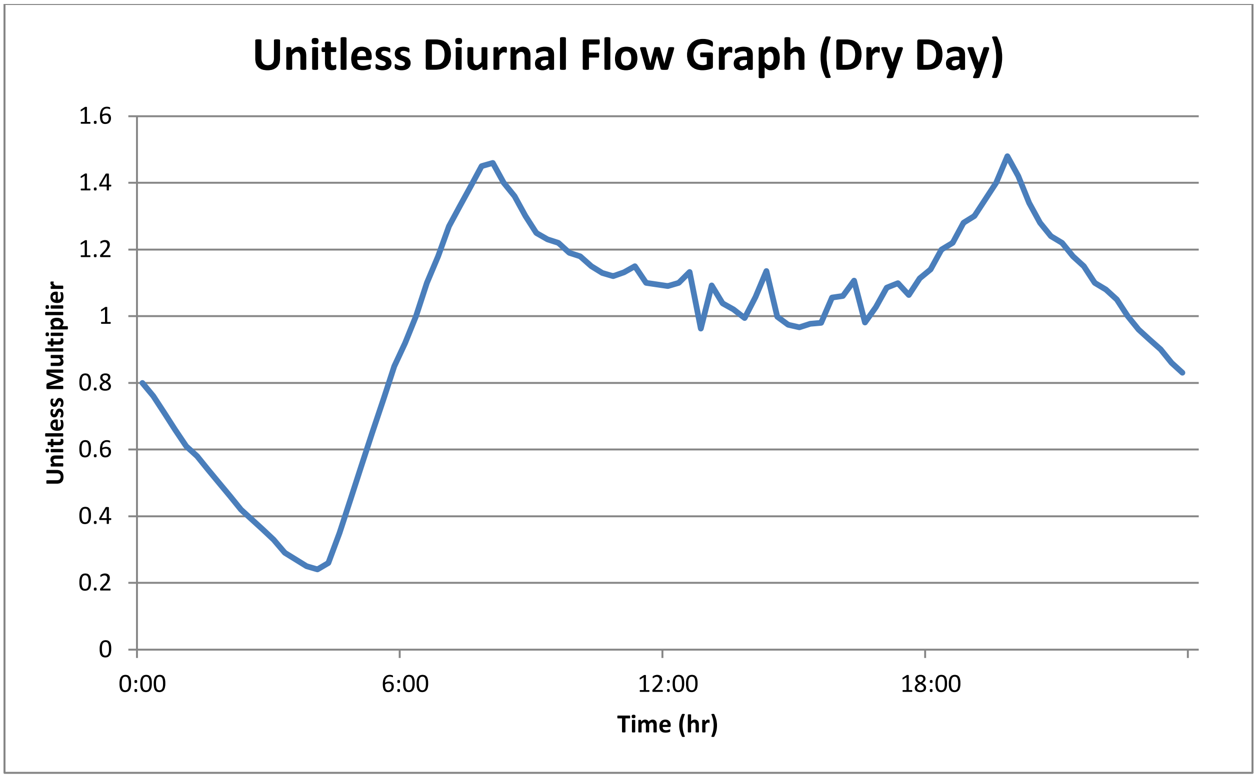

Figure 2: Adapted Unitless Diurnal Flow Pattern (Dry Day) – 12/10/2013

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.

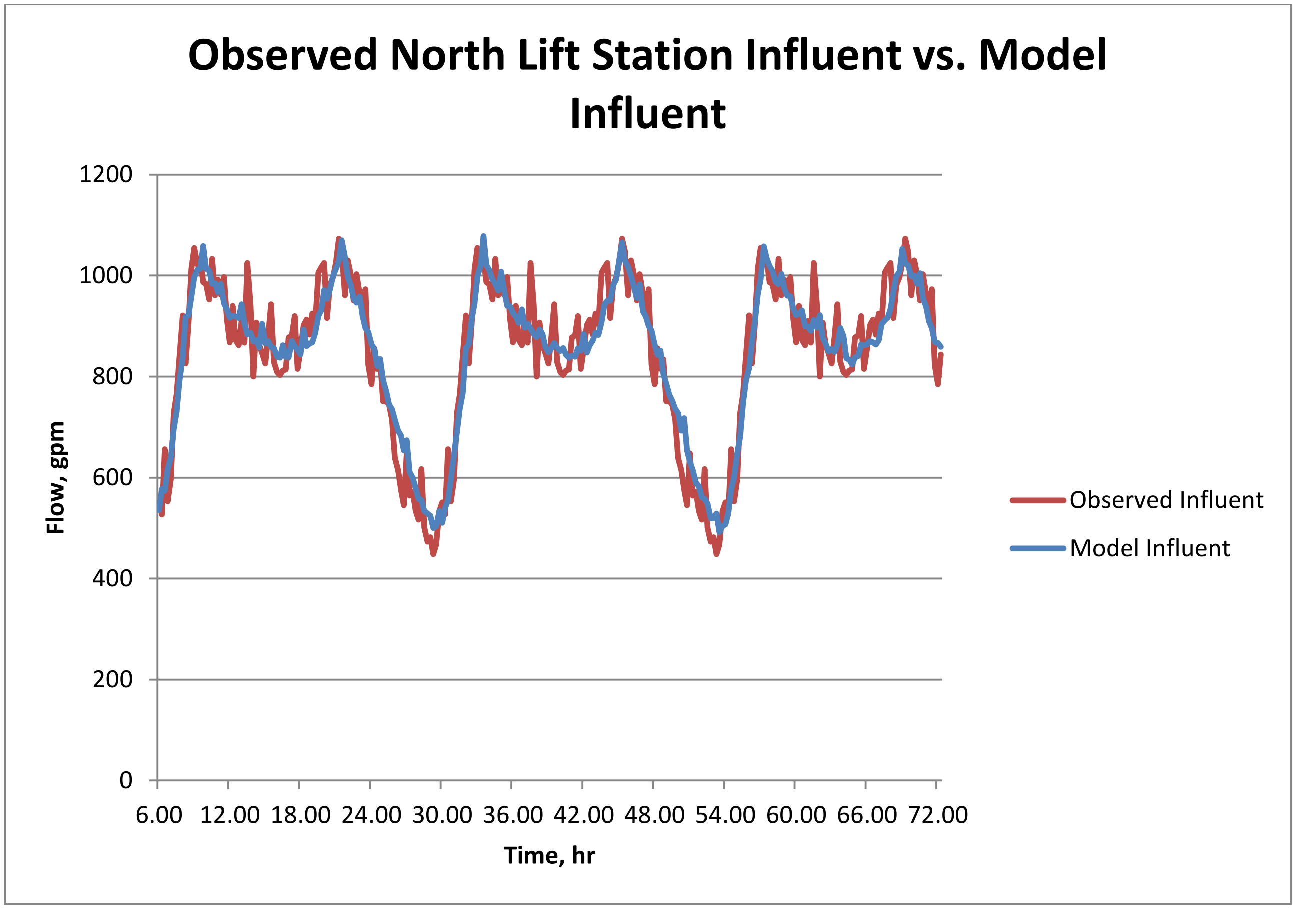

Figure 3: North Plant Influent Model Flows vs. North Plant Influent Observed Flow (Base Flow)

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.

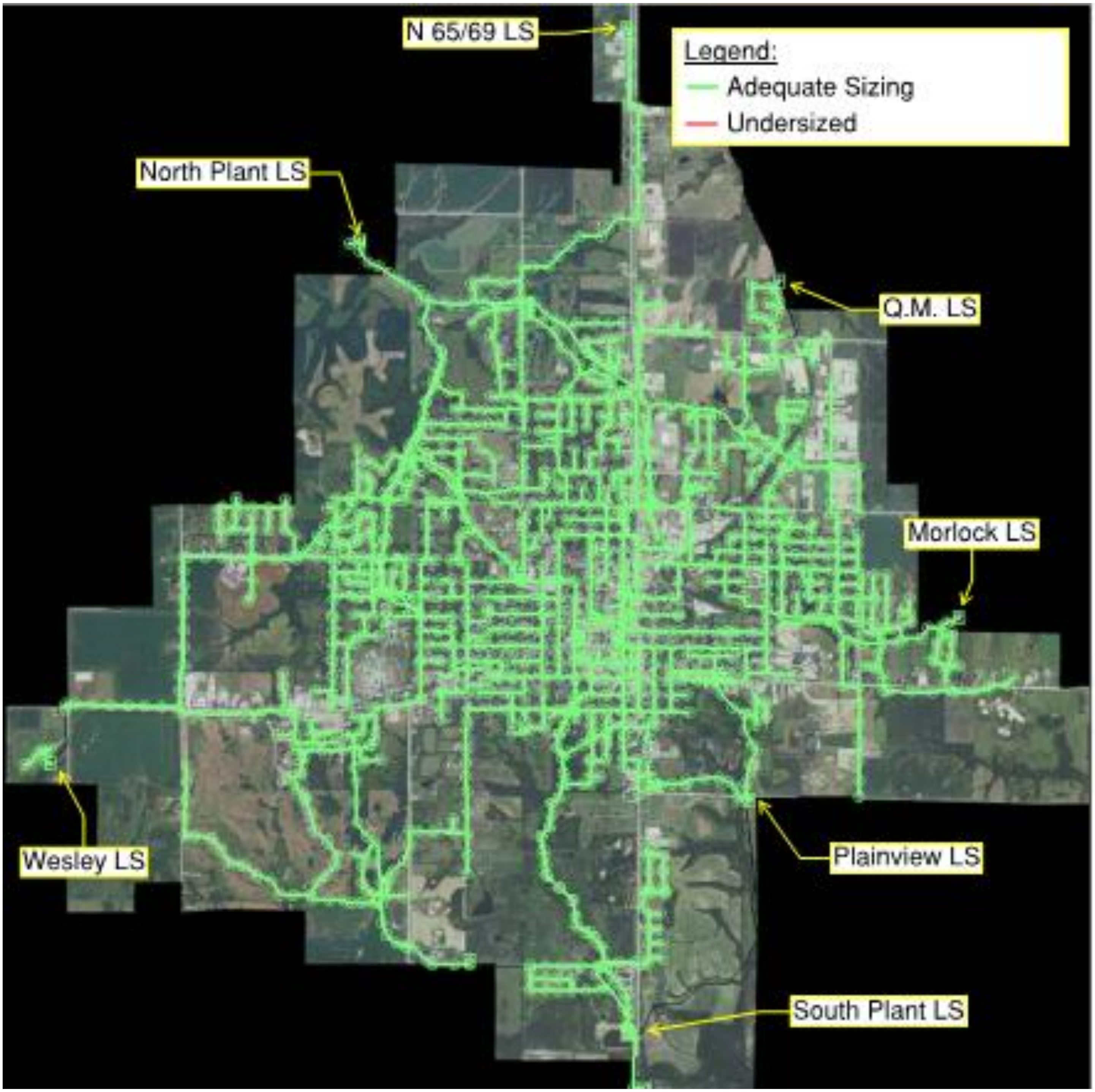

Figure 4: Model Output – Dry Day Base Flow

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.

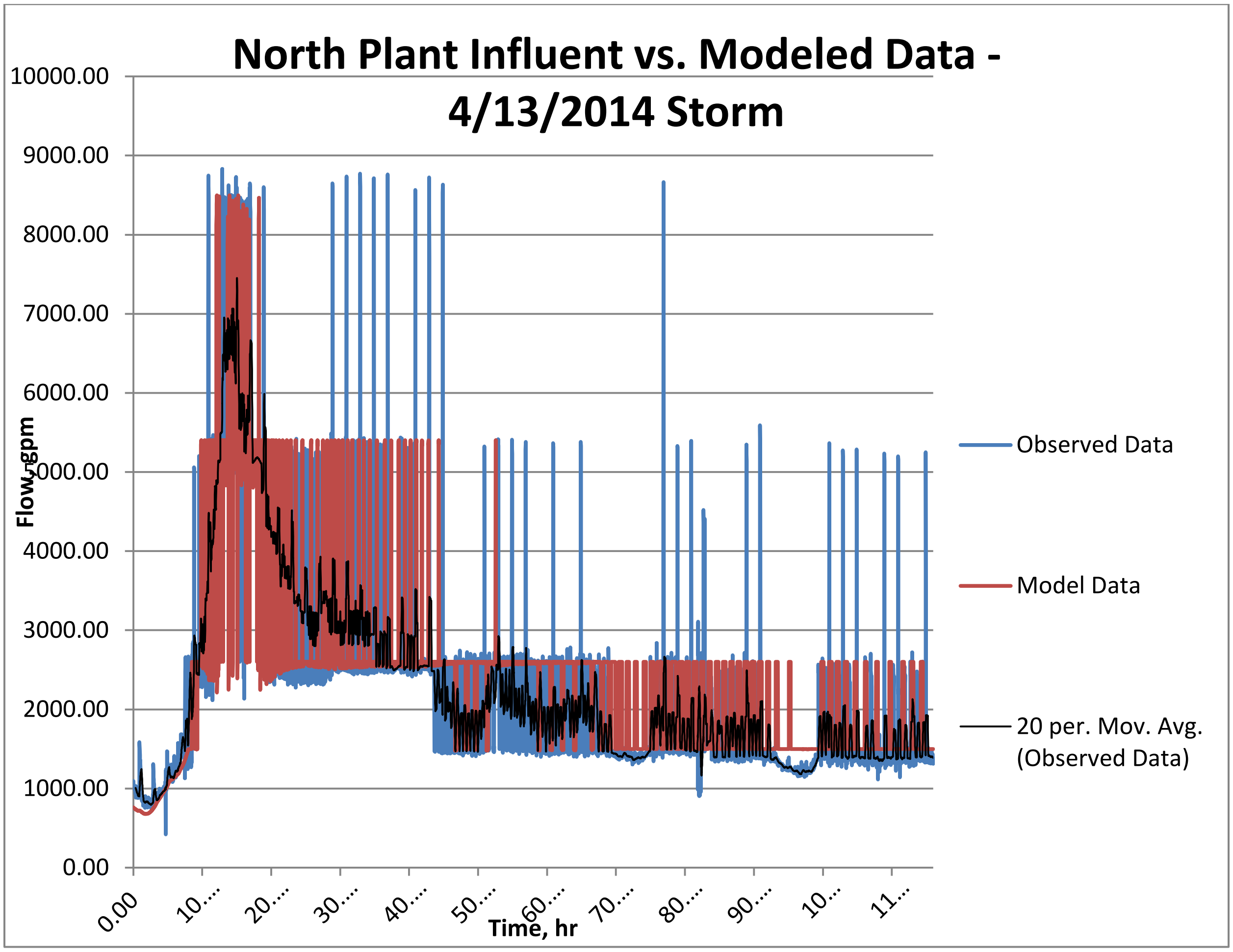

Figure 5: North Plant Lift Station Effluent Model Flows vs. Observed Flows (April 13, 2014 Rainfall Event)

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.

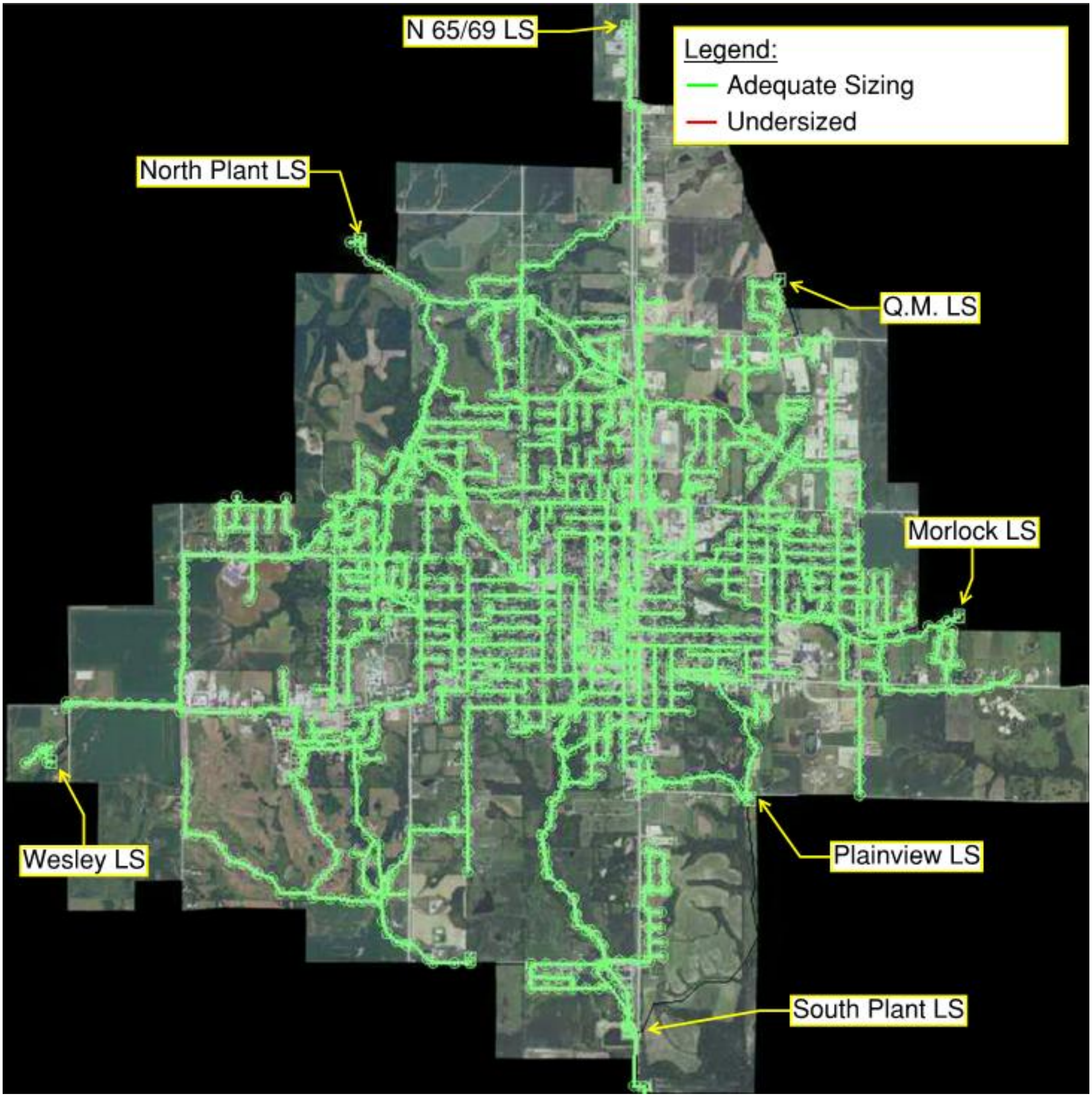

Figure 6: Model Output – April 13, 2014 Rainfall Event (2.65 inch rainfall)

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.

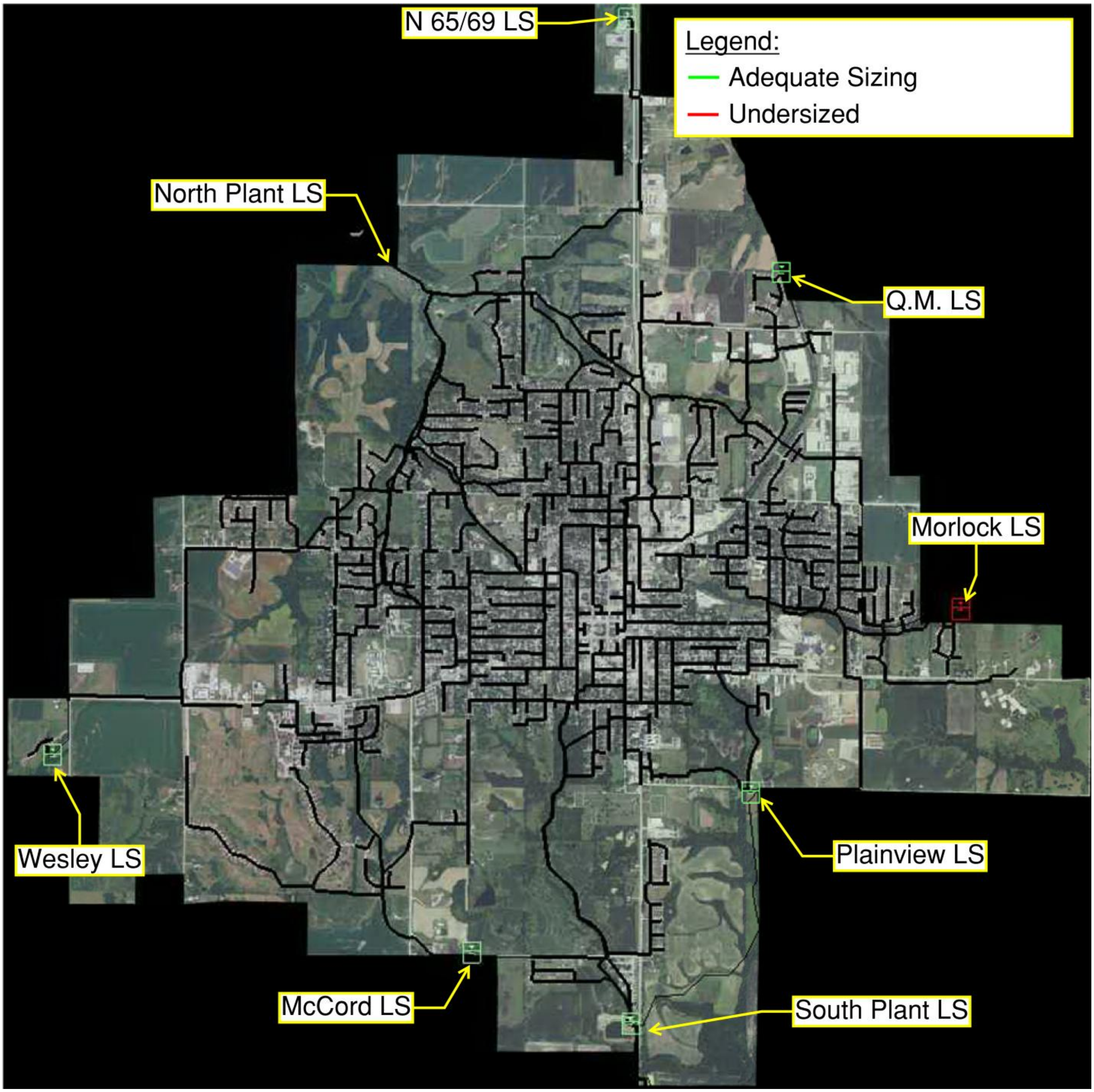

Figure 7: Model Output – 10-yr, 24-hr Storm, Lift Station Analysis

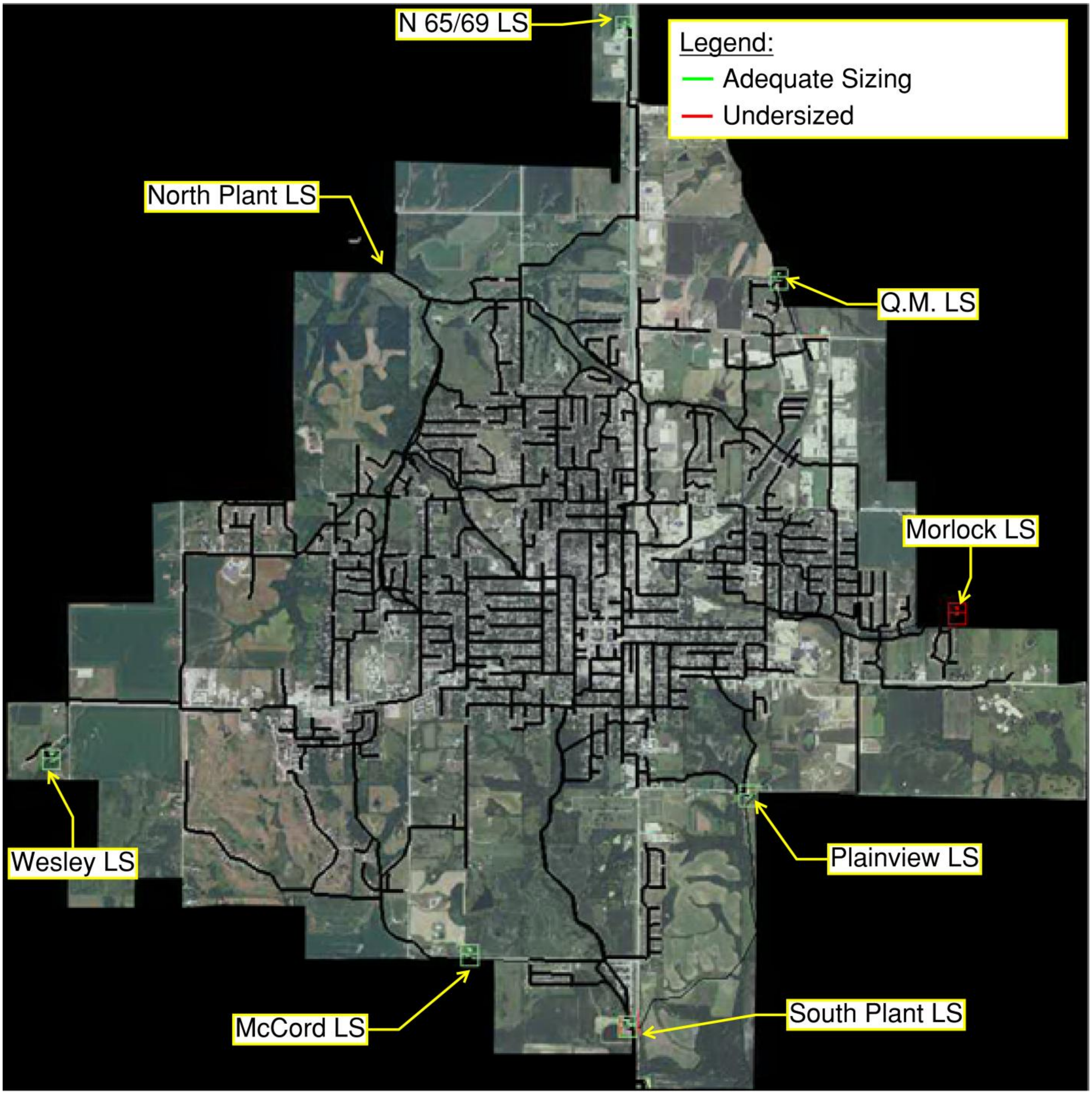

Figure 8: Model Output – 25-yr, 24-hr Storm, Lift Station Analysis

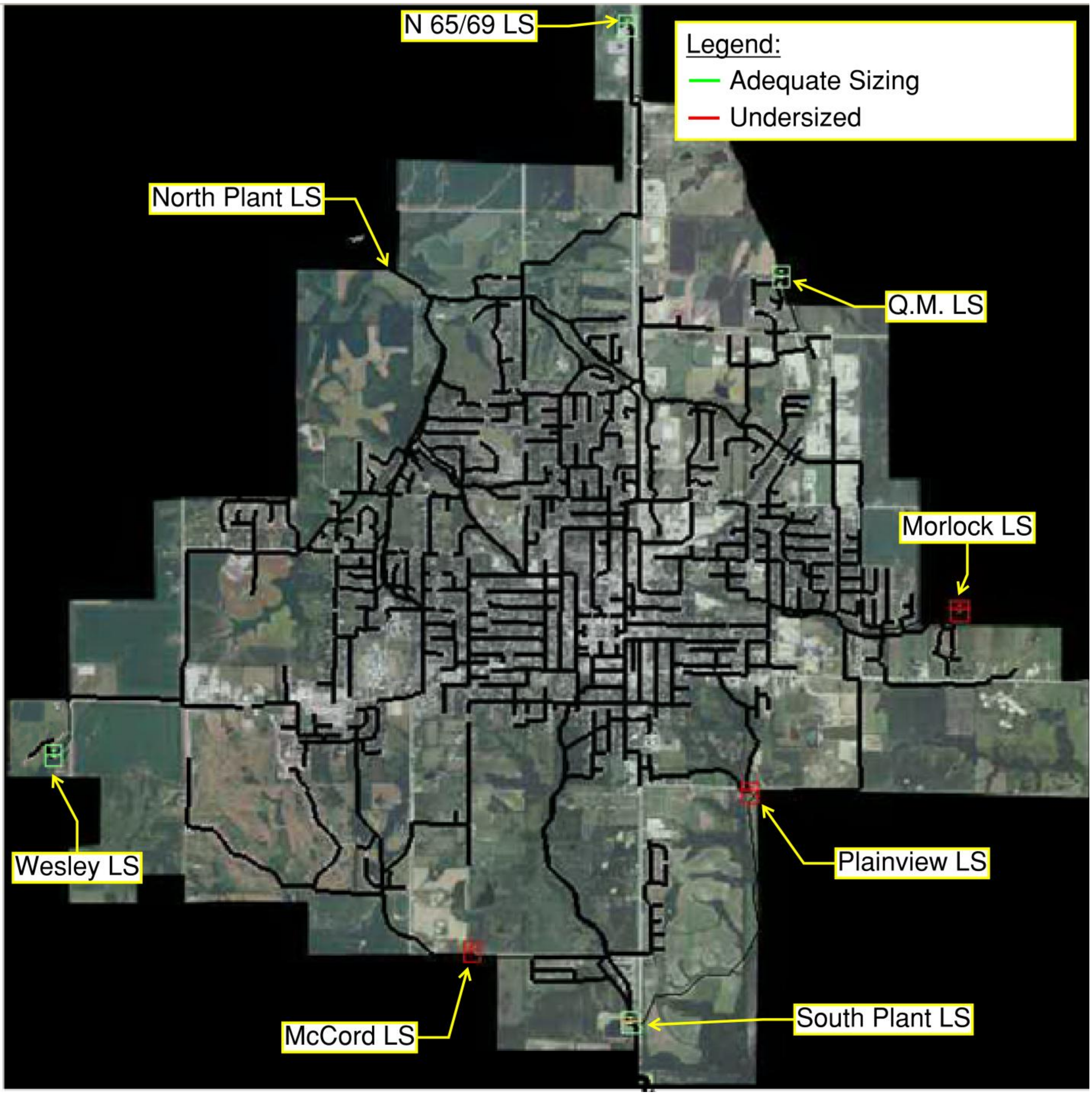

Figure 9: Model Output – 100-yr, 24-hr Storm, Lift Station Analysis

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

Conveyance System Improvements:

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.

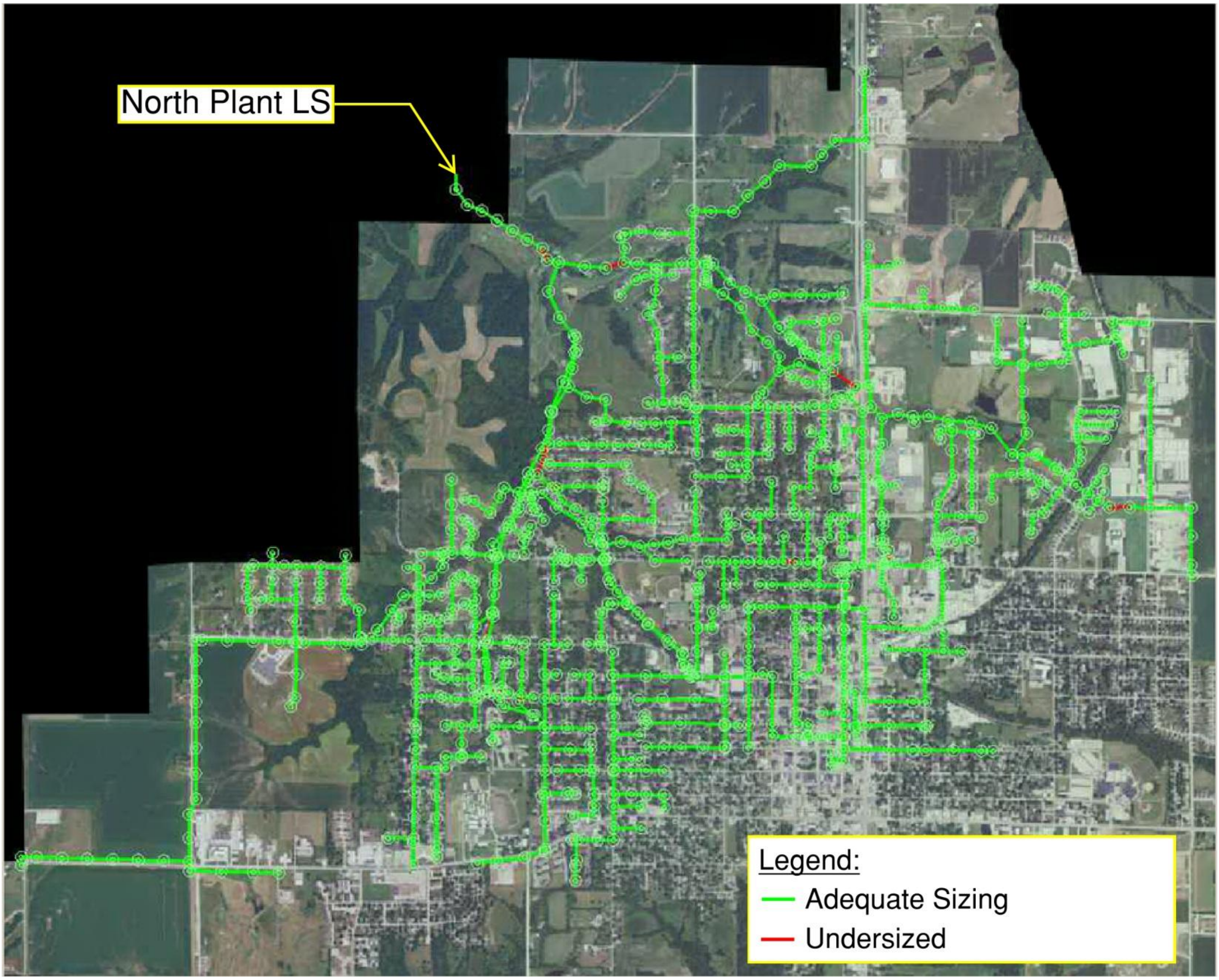

Figure 10: North Plant Lift Station Catchment Area, 10-yr, 24-hr Storm

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.

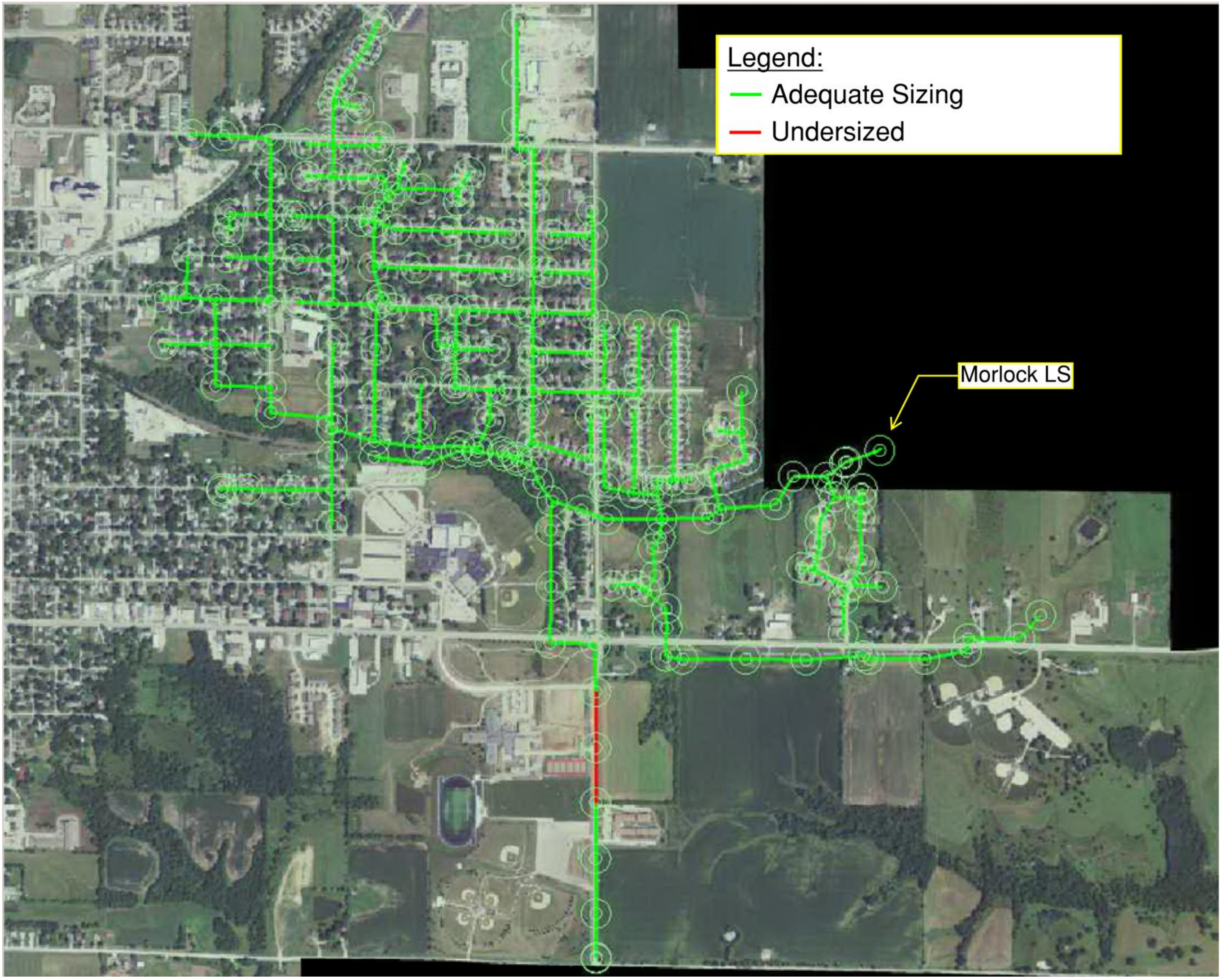

Figure 11: Morlock Lift Station Catchment Area, 10-yr, 24-hr Storm

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

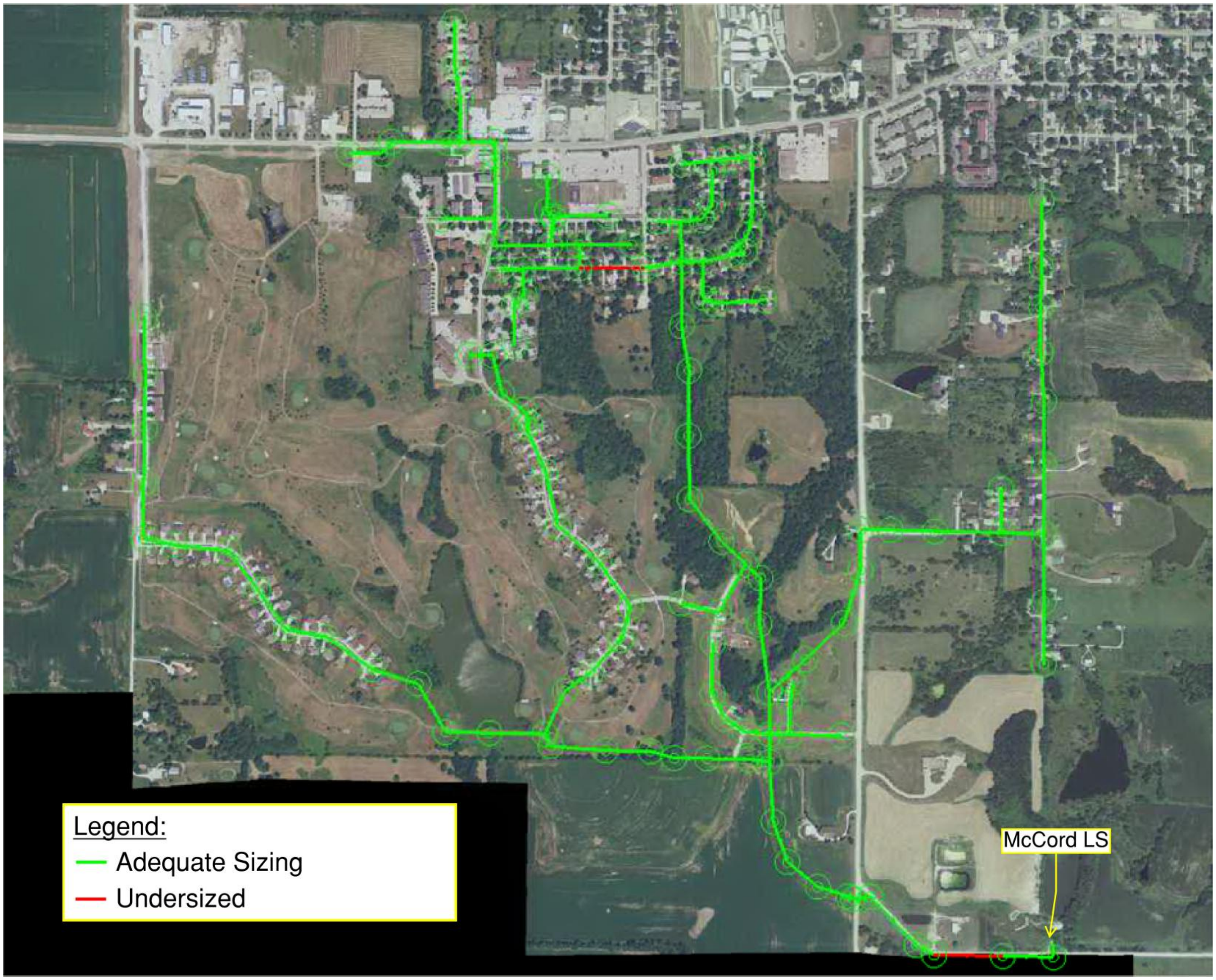

Figure 13: McCord Lift Station Catchment Area, 10-yr, 24-hr Storm

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.

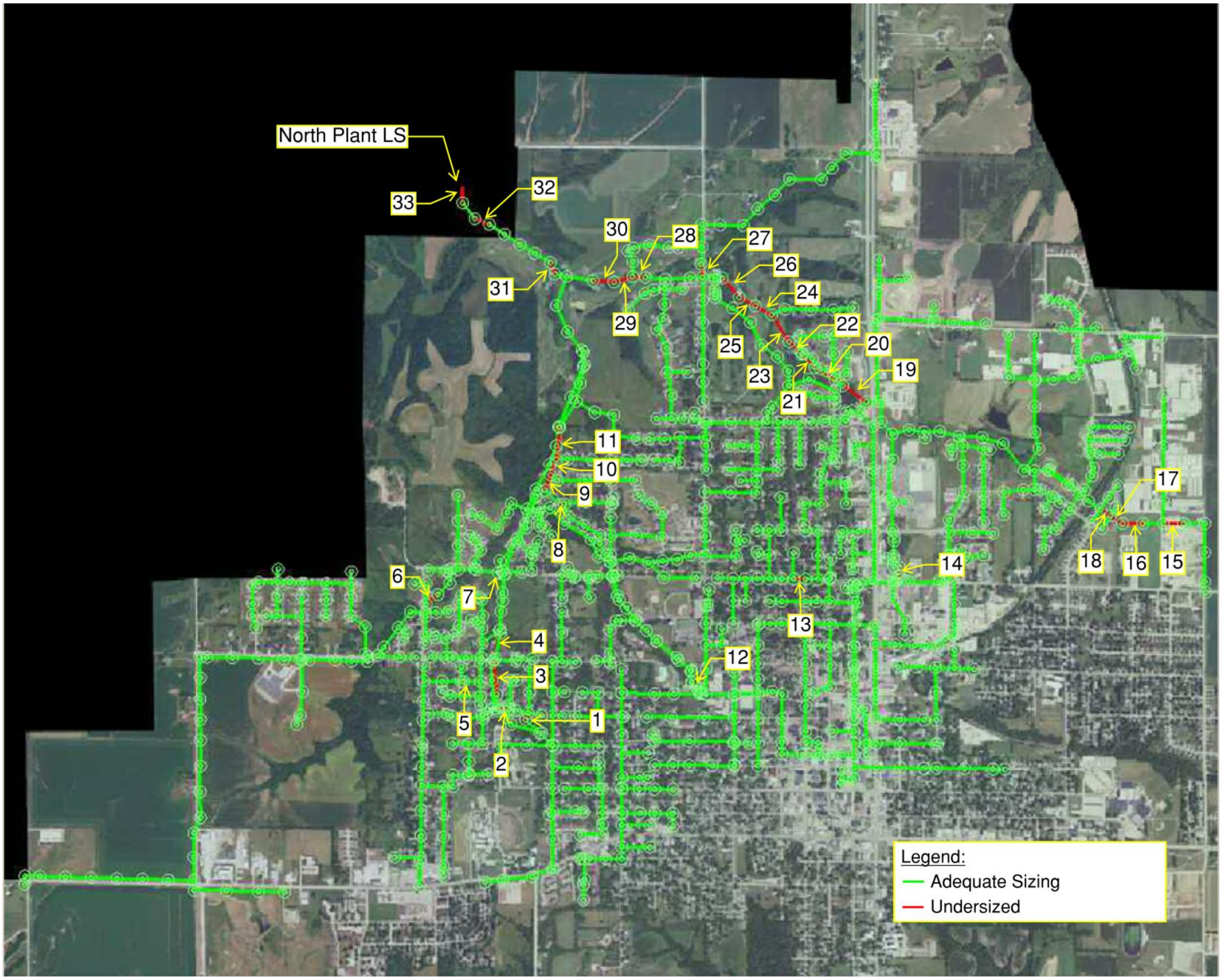

Figure 15: North Plant Lift Station Catchment Area, 25-yr, 24-hr Storm

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.

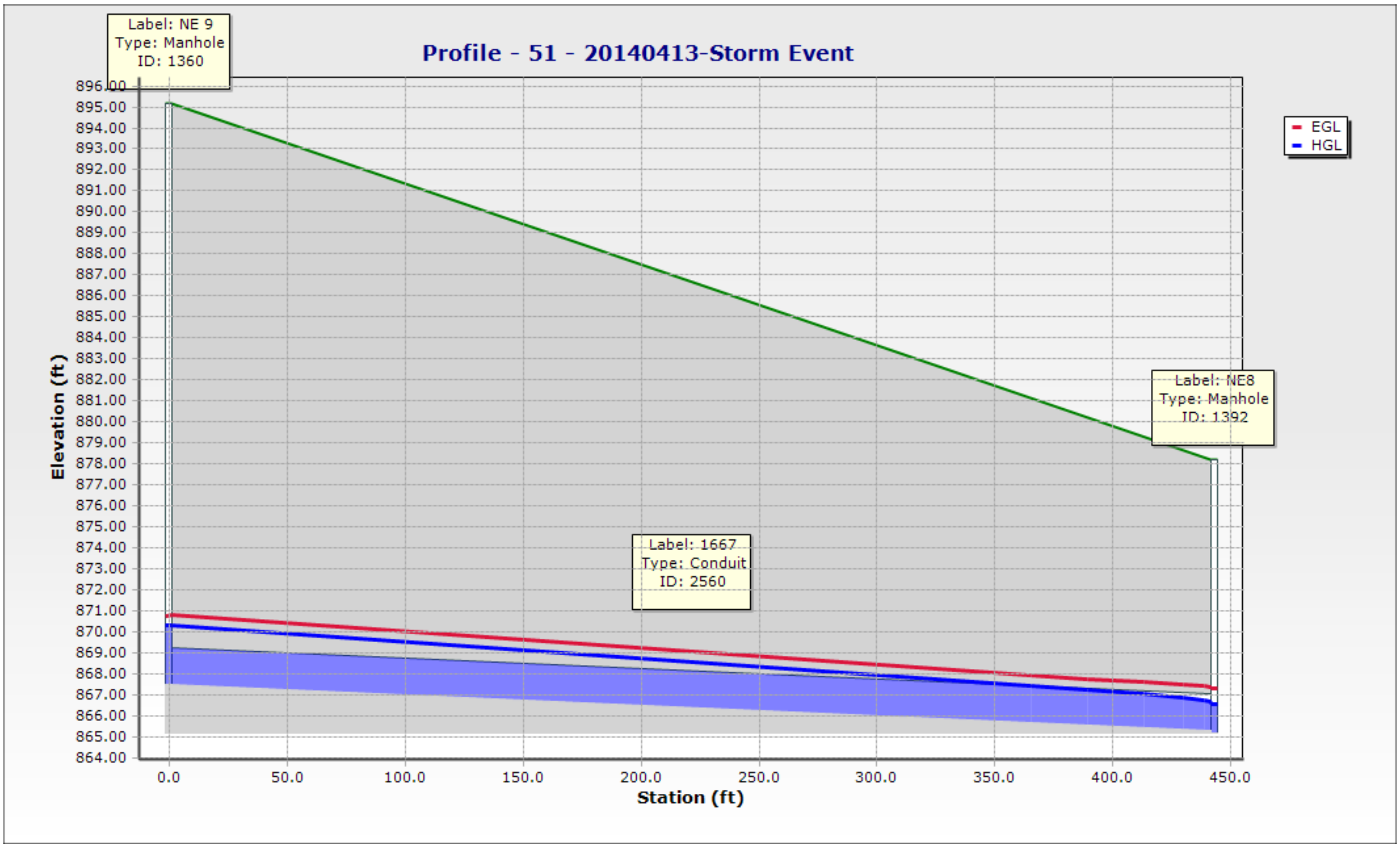

Figure 16: Minor Surcharging Pipe Section, 25-yr, 24-hr Storm

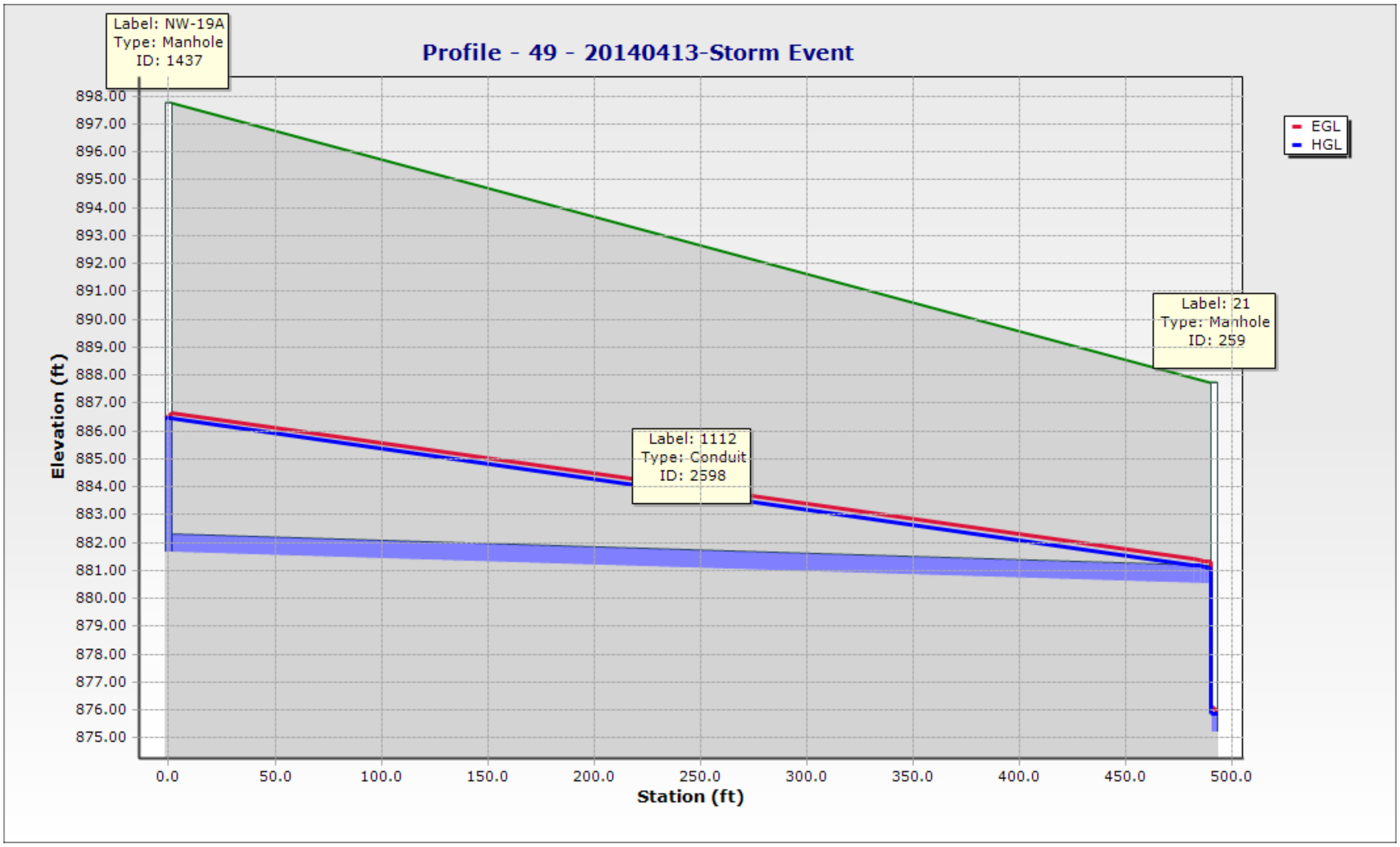

Figure 17: Surcharging Pipe Section, 25-yr, 24-hr Storm

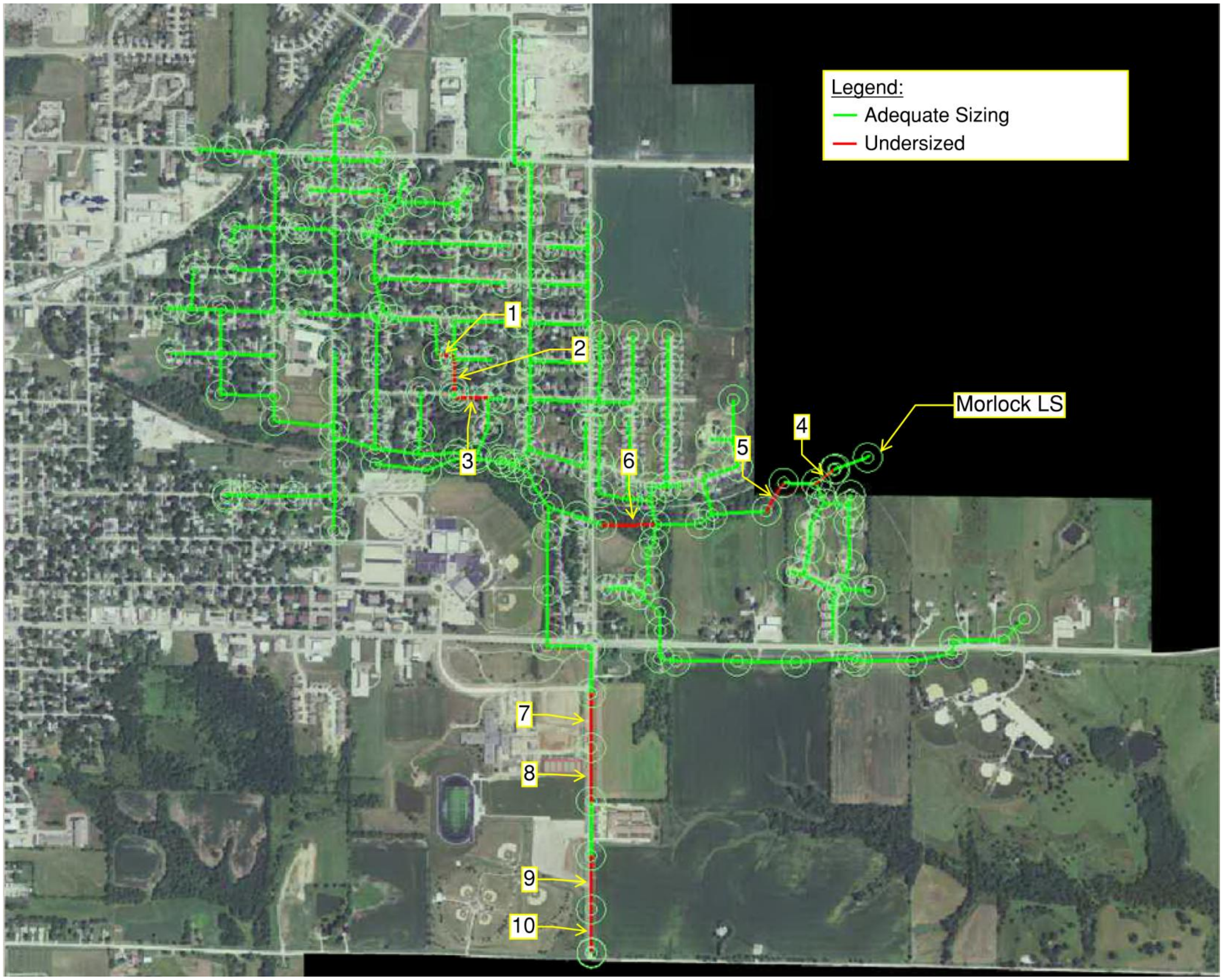

Figure 18: Morlock Lift Station Catchment Area, 25-yr, 24-hr Storm

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

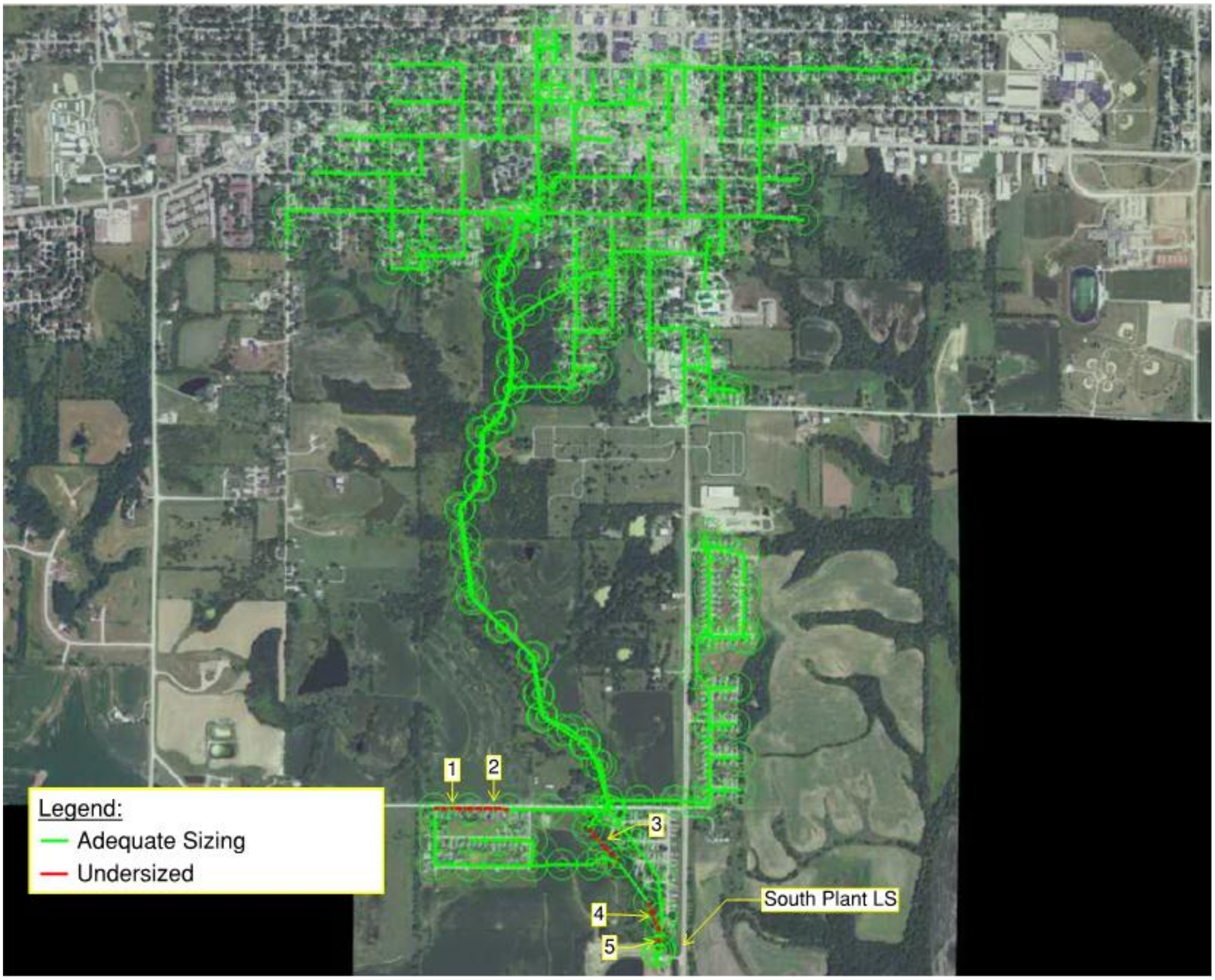

Figure 19: South Plant Lift Station Catchment Area, 25-yr, 24-hr Storm

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

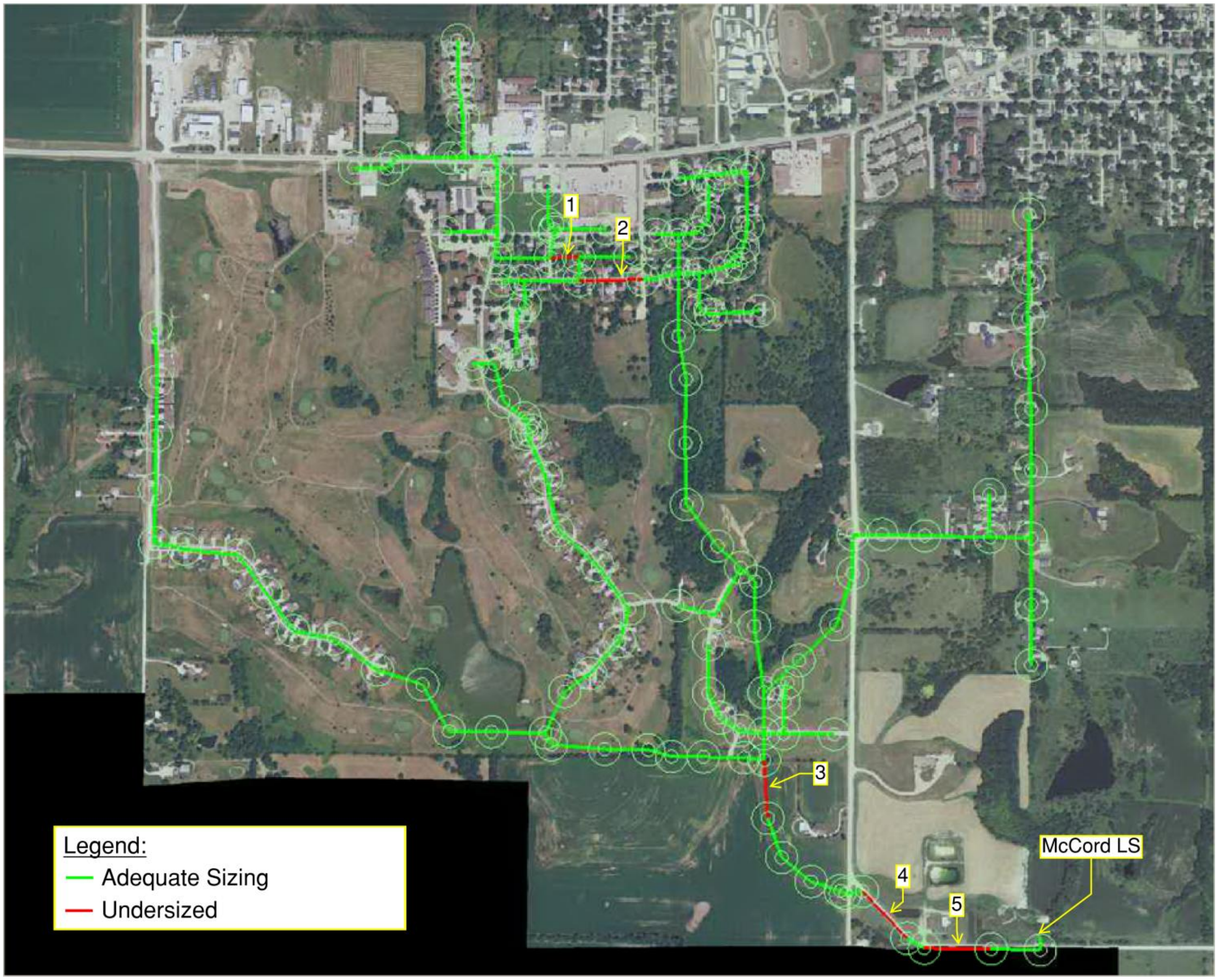

Figure 20: McCord Lift Station Catchment Area, 25-yr, 24-hr Storm

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.

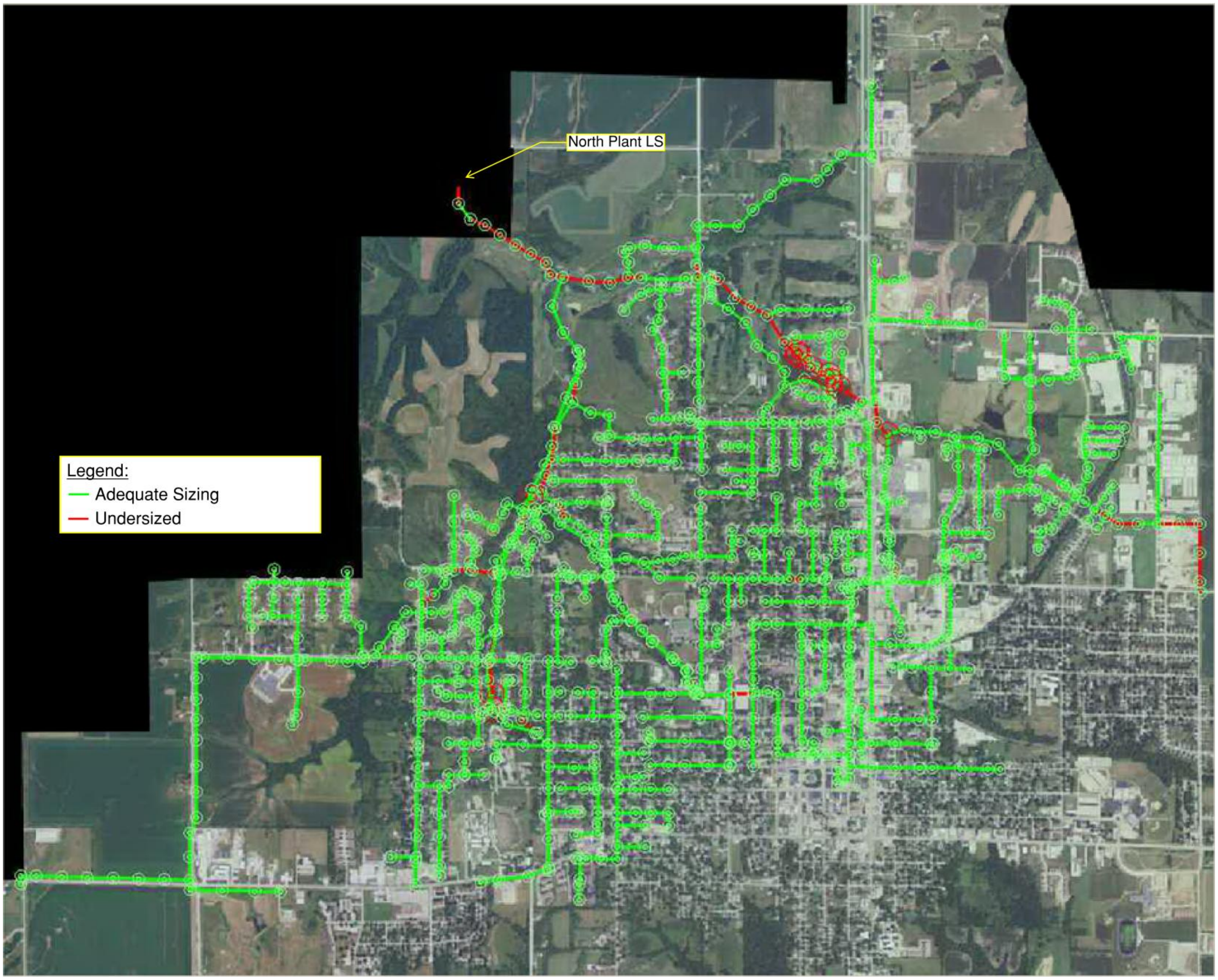

Figure 22: North Plant Lift Station Catchment Area, 100-yr, 24-hr Storm

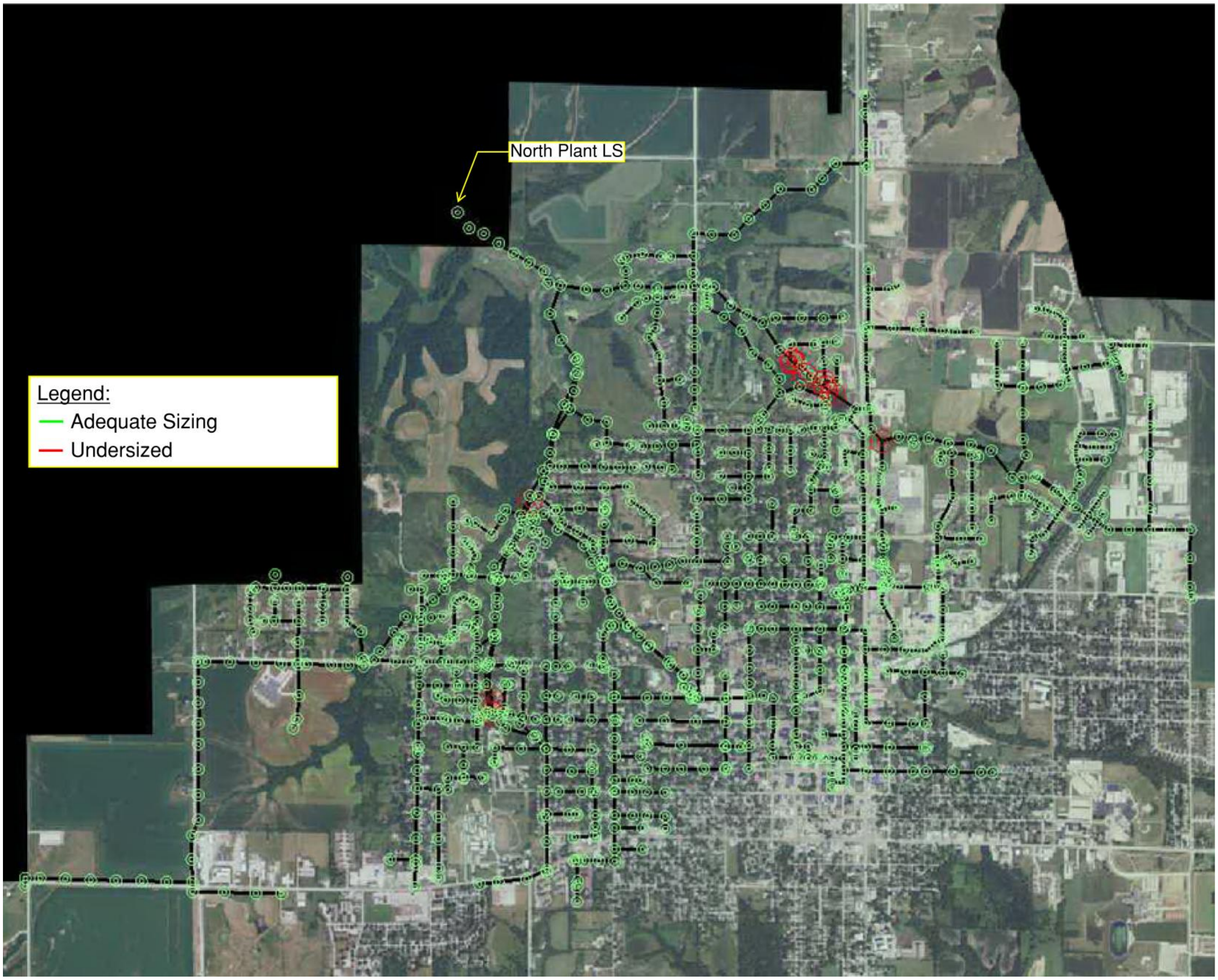

Figure 23: North Plant Lift Station Catchment Area Overflows, 100-yr, 24-hr Storm

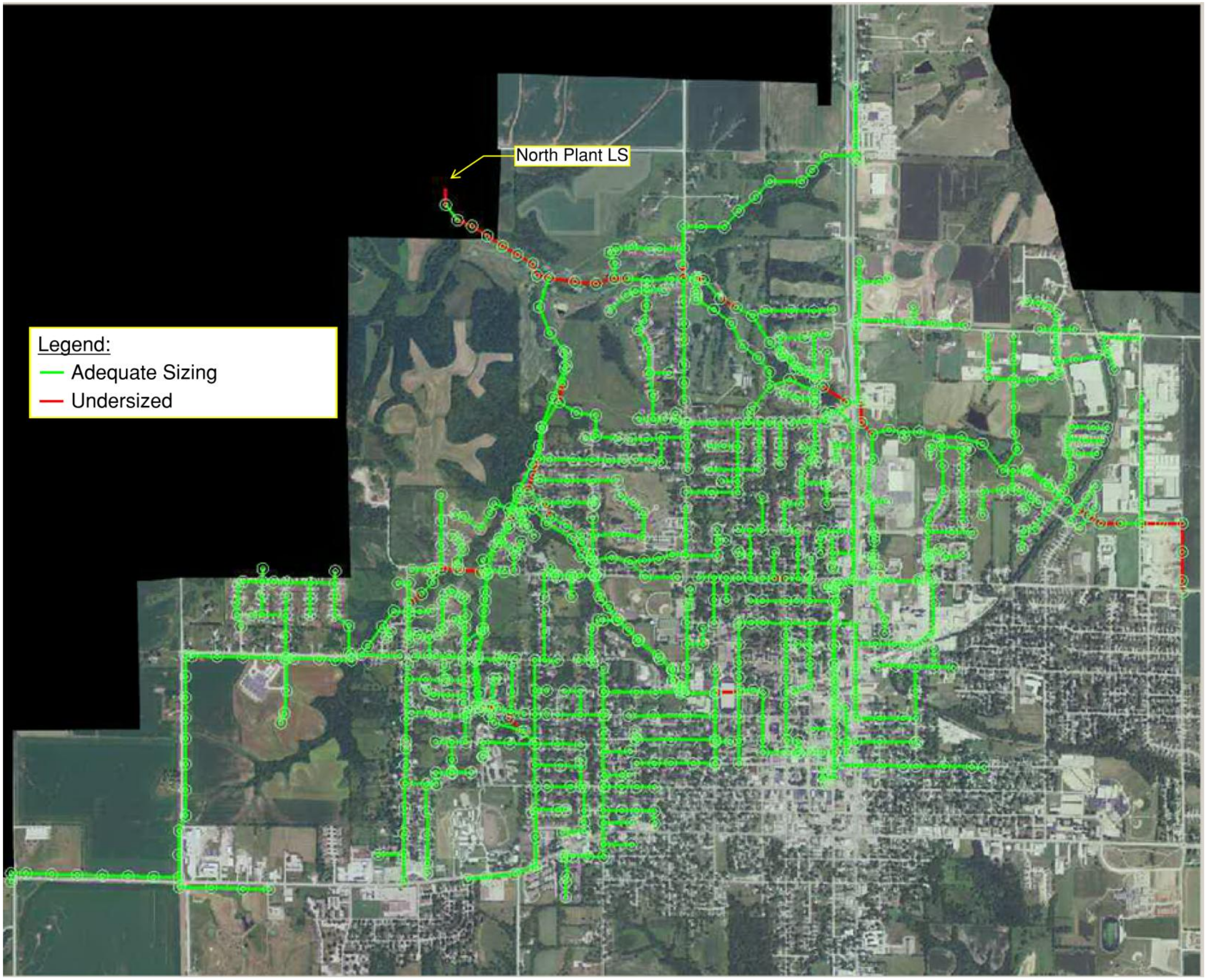

Figure 24: North Plant Lift Station Catchment Area with 25-yr Improvements, 100-yr, 24-hr Storm

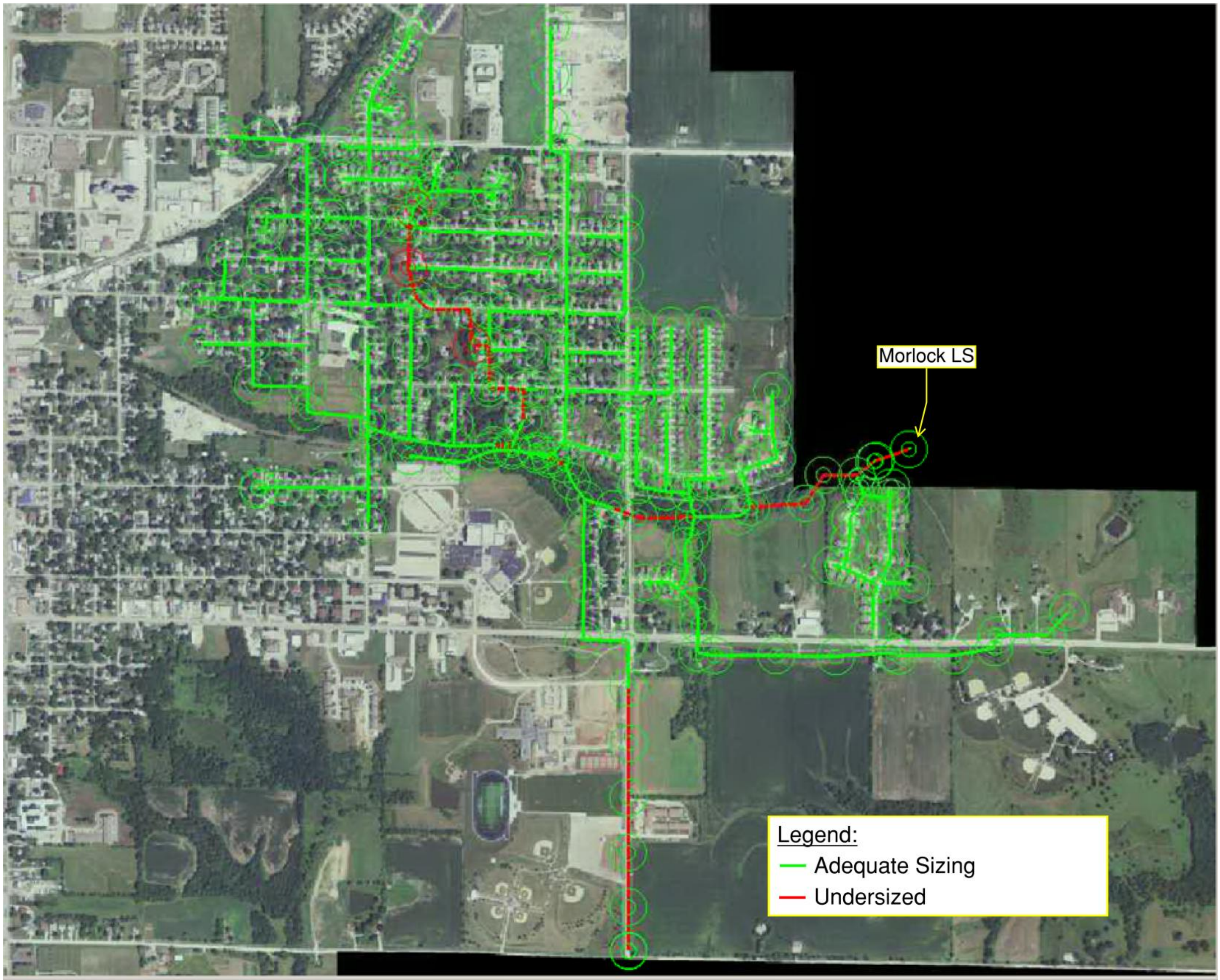

Figure 25: Morlock Lift Station Catchment Area, 100-yr, 24-hr Storm

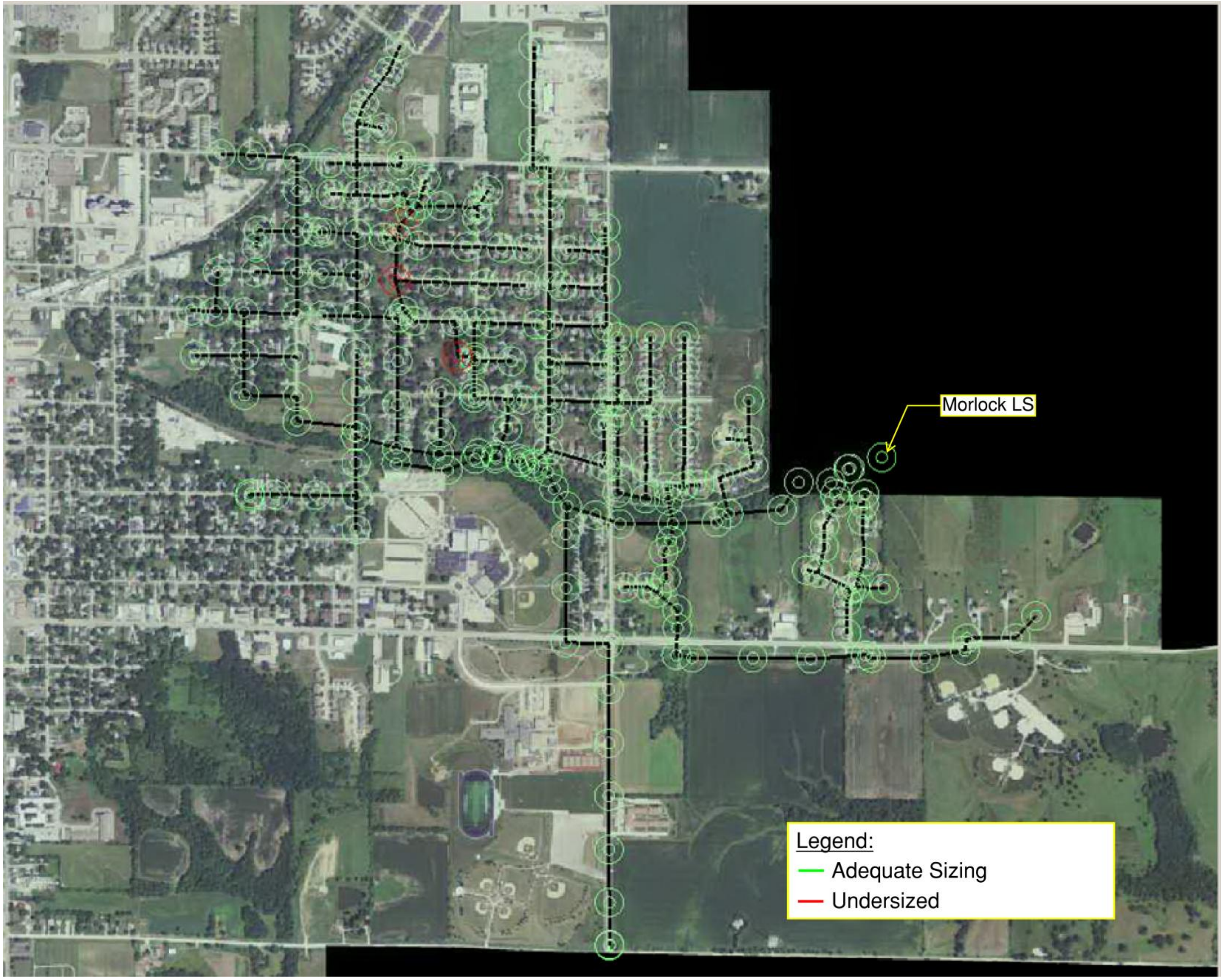

Figure 26: Morlock Lift Station Catchment Area Overflows, 100-yr, 24-hr Storm

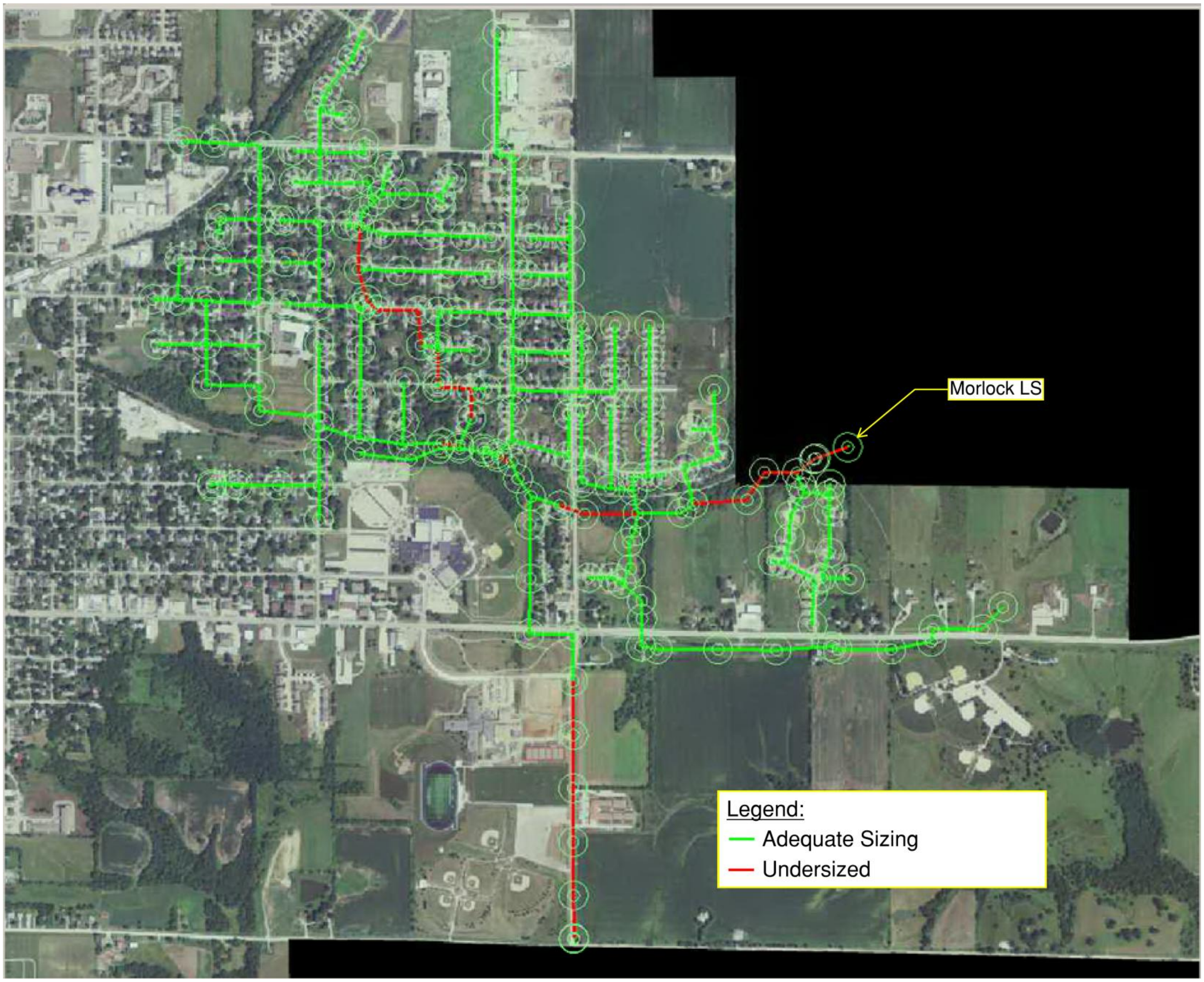

Figure 27: Morlock Lift Station Catchment Area with 25-yr Improvements, 100-yr, 24-hr Storm

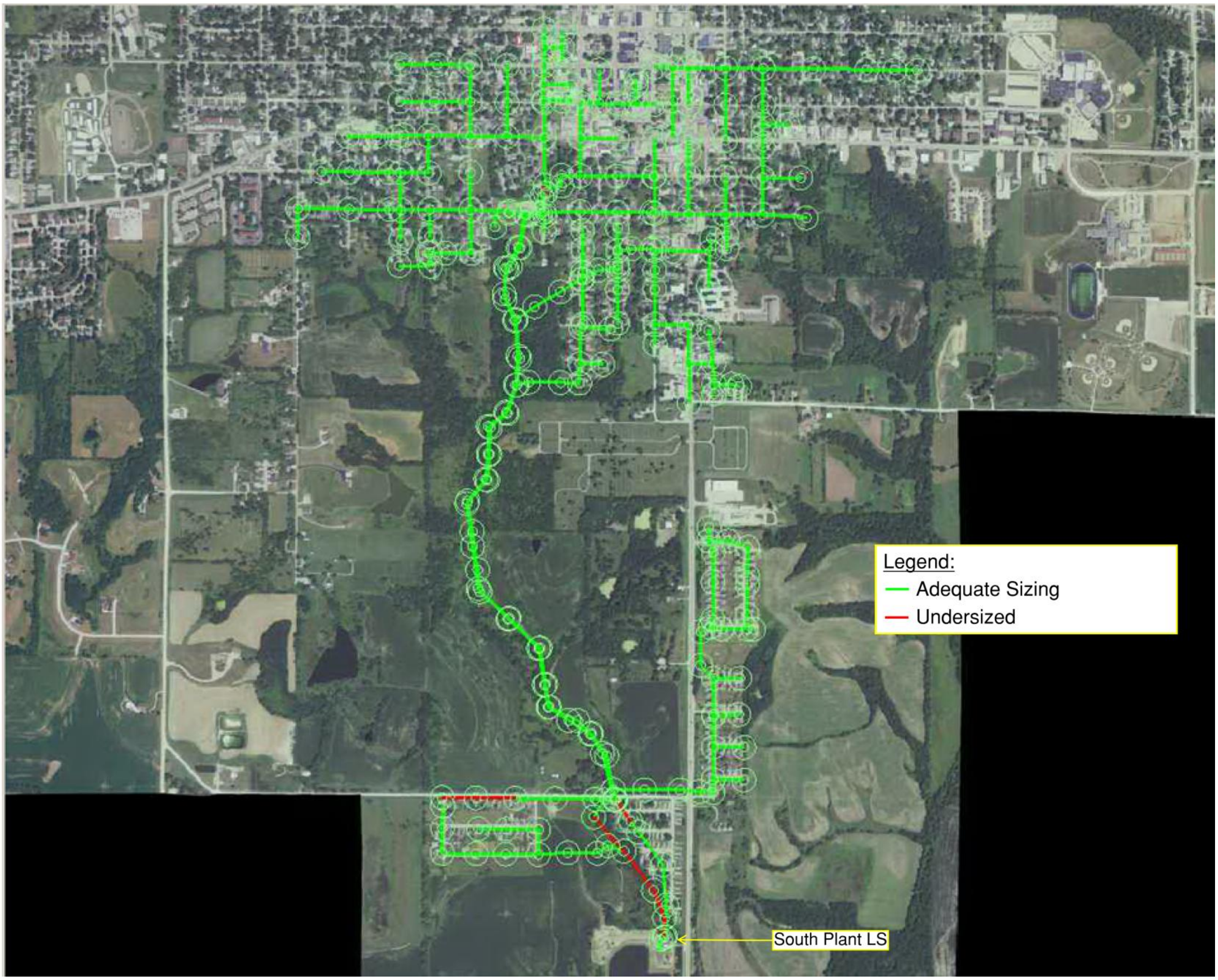

Figure 28: South Plant Lift Station Catchment Area, 100-yr, 24-hr Storm

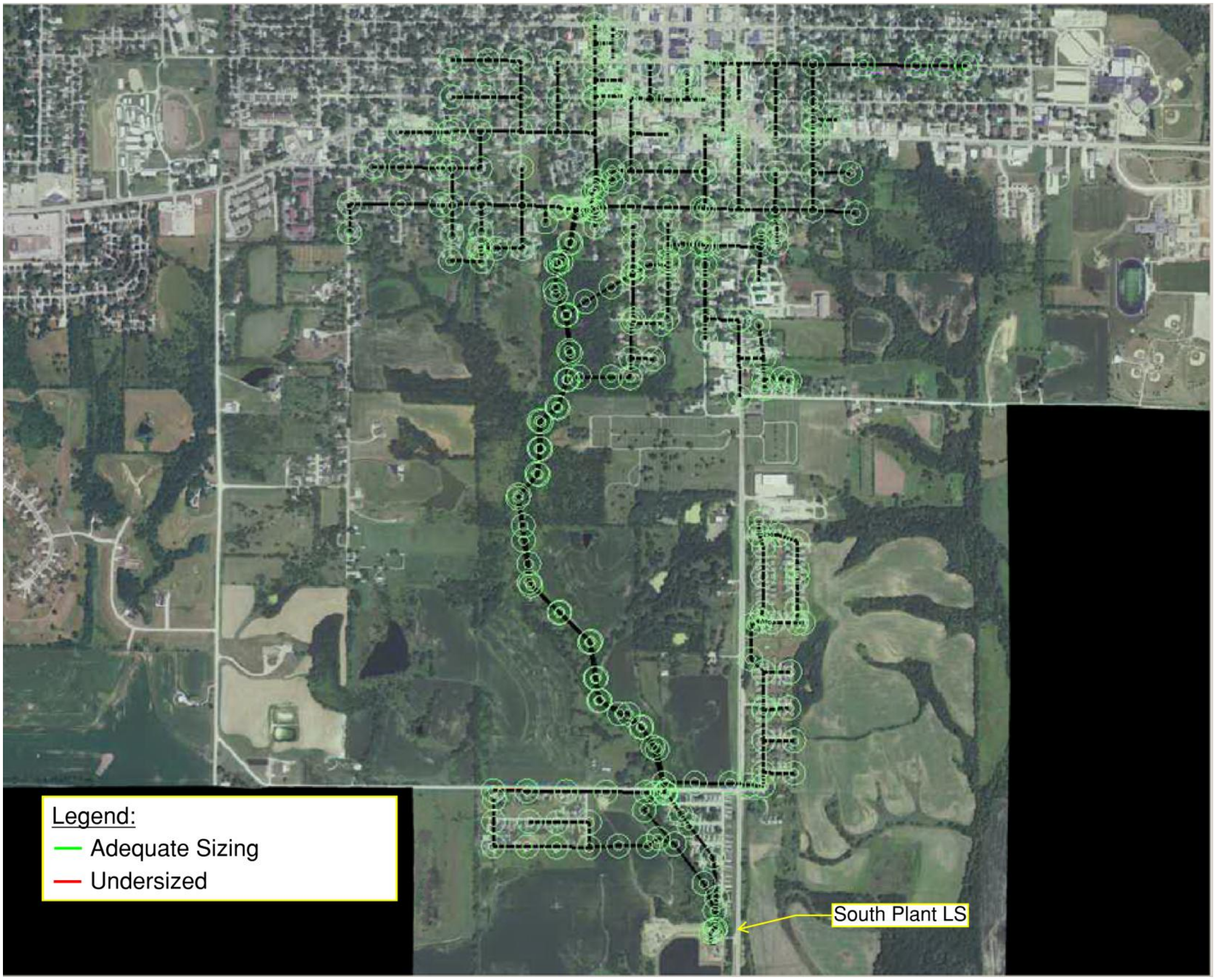

Figure 29: South Plant Lift Station Catchment Area Overflows, 100-yr, 24-hr Storm

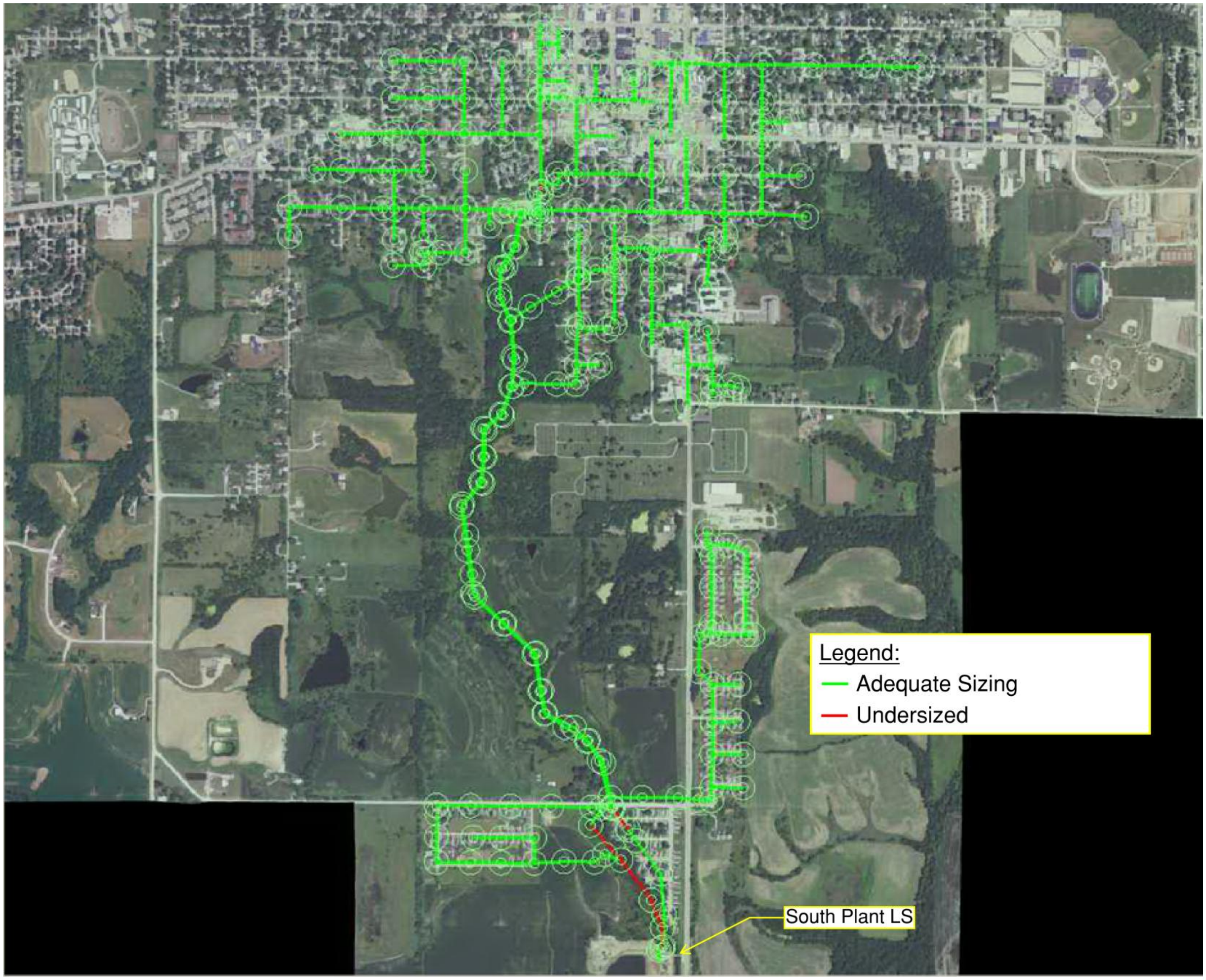

Figure 30: South Plant Lift Station Catchment Area with 25-yr Improvements, 100-yr, 24-hr Storm

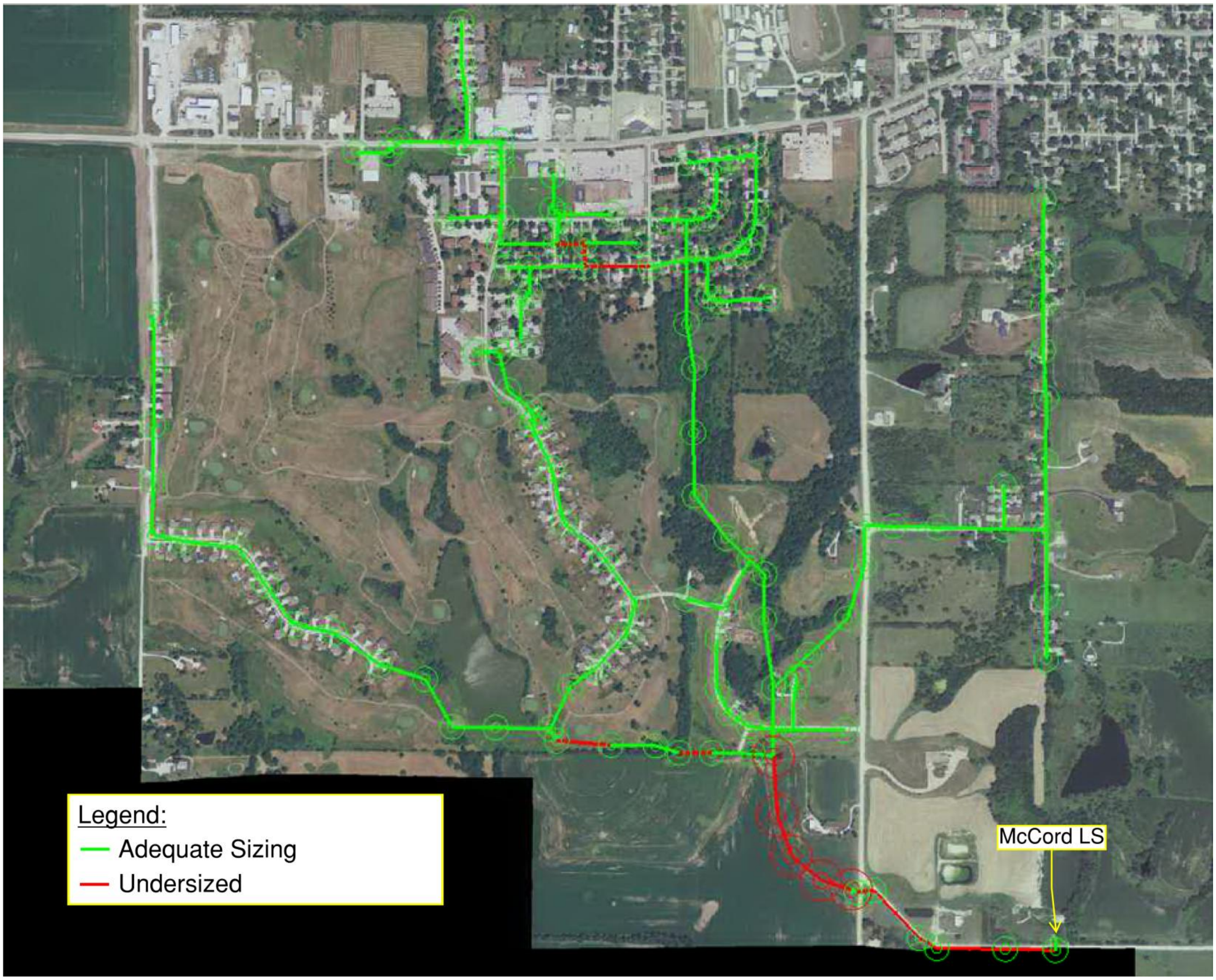

Figure 31: McCord Lift Station Catchment Area, 100-yr, 24-hr Storm

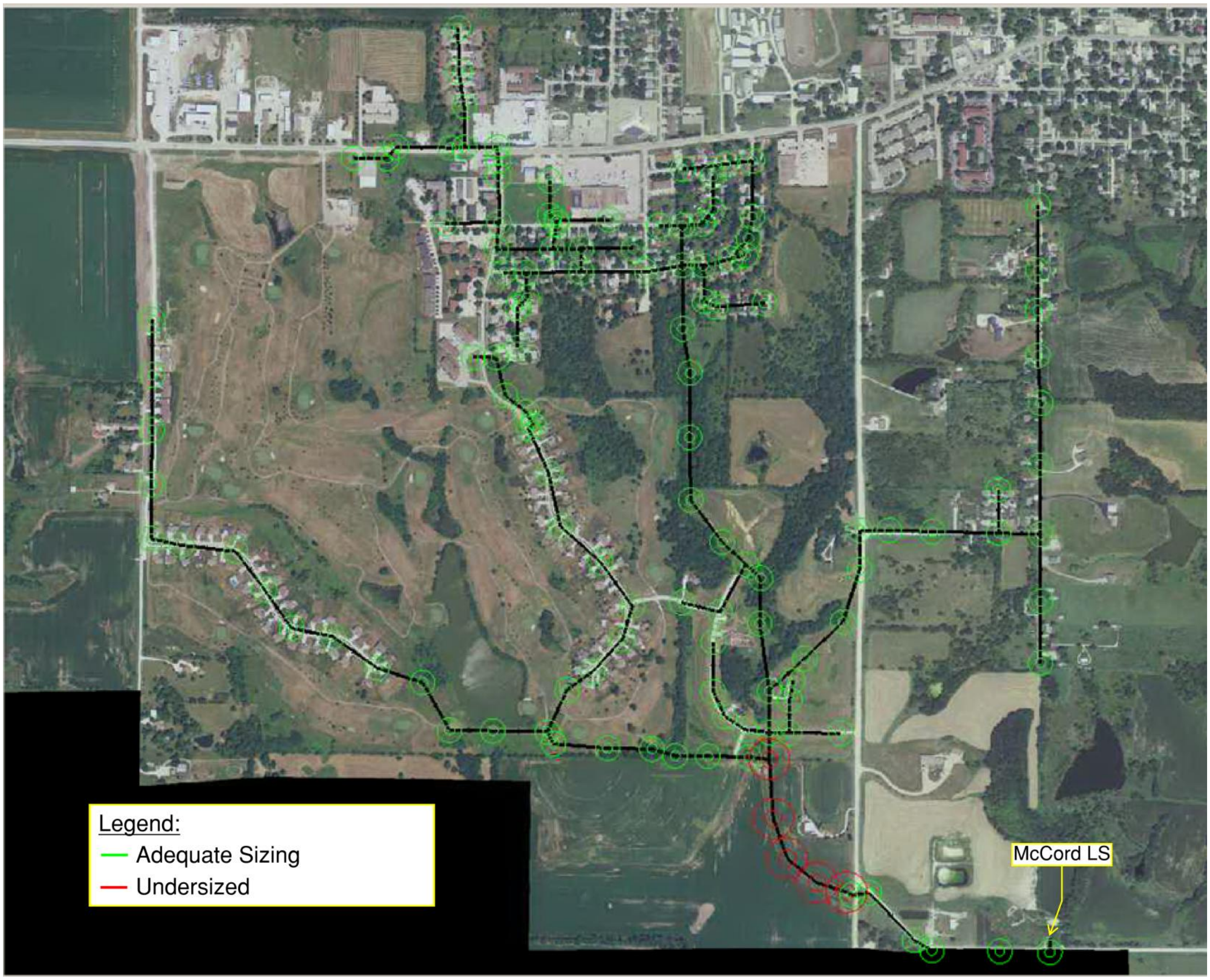

Figure 32: McCord Lift Station Catchment Area Overflows, 100-yr, 24-hr Storm

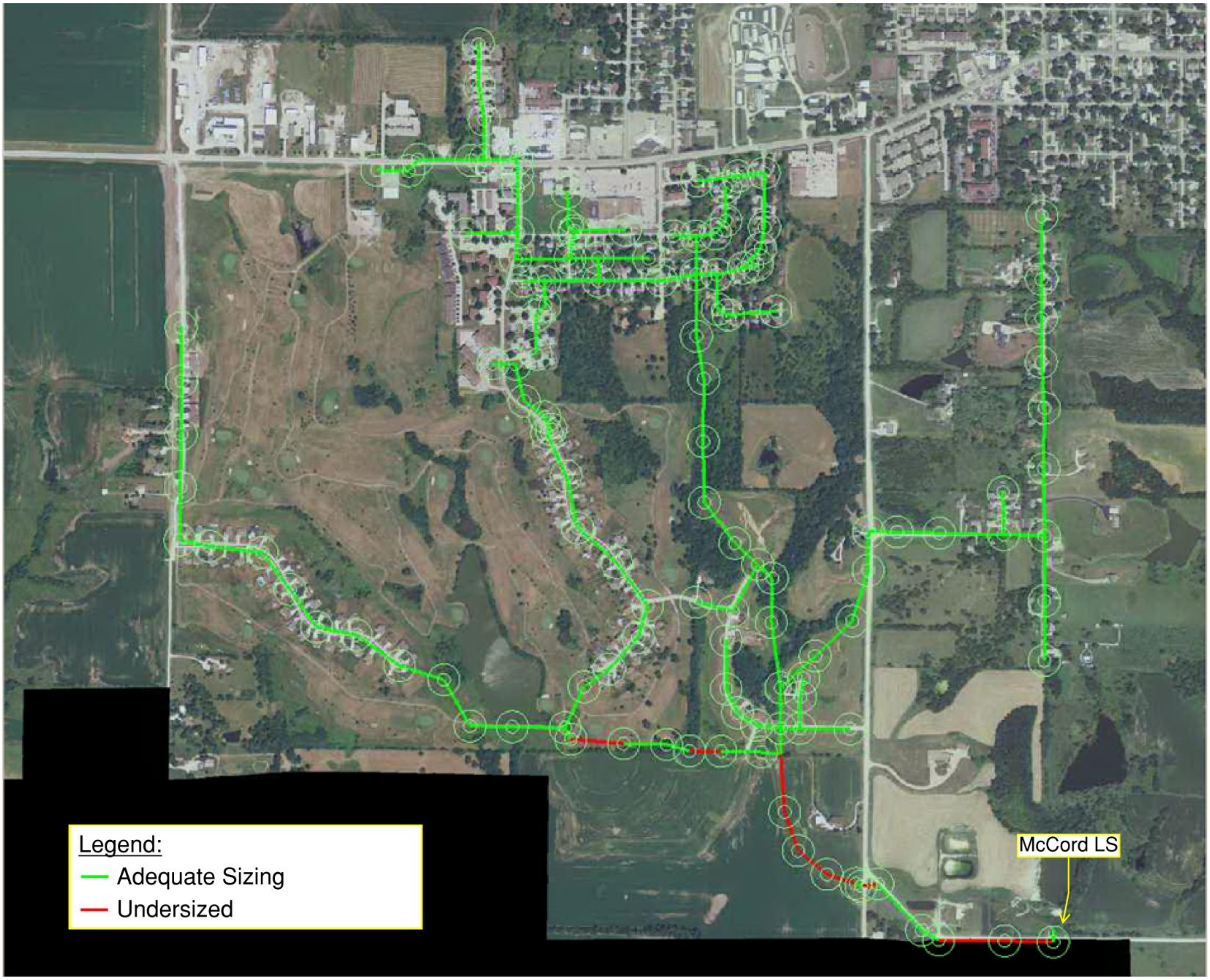

Figure 33: McCord Lift Station Catchment Area with 25-yr Improvements, 100-yr, 24-hr Storm

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

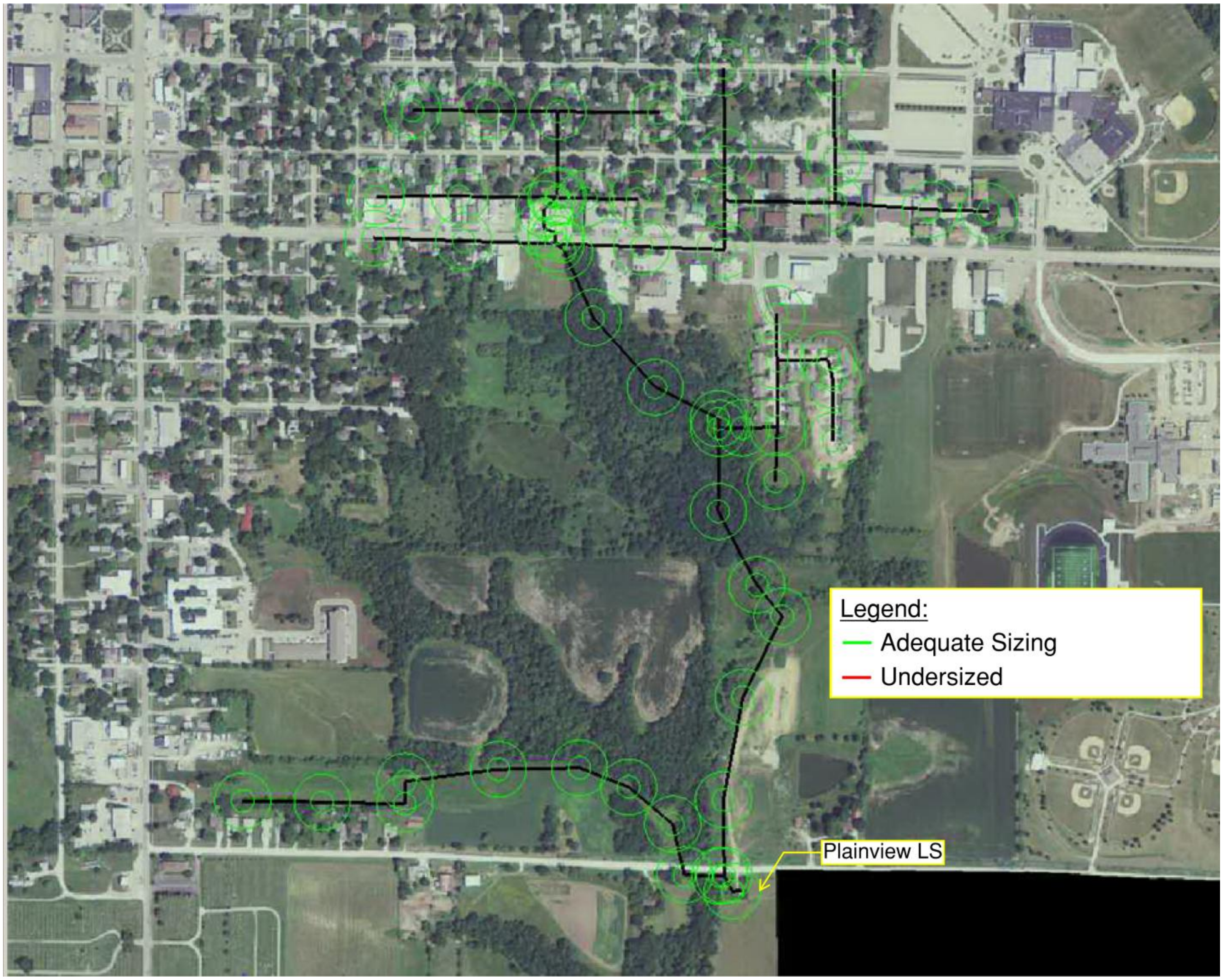

Figure 35: Plainview Lift Station Catchment Area Overflows, 100-yr, 24-hr Storm

APPENDIX D
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

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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 Departmen*t. *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 E
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.