Village of Beecher, Illinois

Water & Sanitary Sewer Master Plan Update

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LIST OF ABBREVIATIONS

avg	-	average
cfs	-	cubic feet per second
EPA	-	Environmental Protection Agency
ft	-	feet
ft2	-	square feet
ft3	-	cubic feet
gcd	-	gallons per capita per day
gpd	-	gallons per day
gpm	-	gallons per minute
hr	-	hour
in	-	inches
I/I	-	infiltration/inflow
max	-	maximum
MG	-	million gallons (or mil gal)
MGD	-	million gallons per day
MH	-	manhole
min	-	minimum
PE	-	population equivalent
PS	-	pumping station
SSES	-	sanitary sewer evaluation survey
WWTP	-	wastewater treatment plant

LIST OF DEFINITIONS

Average DWF (gpm)

This is the average dry weather flow for a trunk sewer section.

Peak DWF (gpm) –

This is the peak flow rate tributary to the trunk sewer section. This is determined by multiplying the average flow rate (100 gallons per capita per day) by the IEPA peaking factor, which is:

$$\text{peaking factor} = \frac{18 + (P.E. / 1000)^{1/2}}{4 + (P.E. / 1000)^{1/2}}$$

10-yr I/I Tributary (gpm) –

This column lists the design Inflow and Infiltration for the trunk sewer section. This was calculated using flow data from the two-year storm on May 9, 2003. The I/I data from this storm was extrapolated to a 10-year storm using Bulletin 70 rainfall data.

IEPA Peaking Factor (also noted as Peak DWF gpm)

The ratio of peak wastewater flow to daily average wastewater flow. This ratio is determined by the Illinois Environmental Protection Agency equation:

$$\frac{\text{Peak Flow Rate}}{\text{Average Daily Flow Rate}} = \frac{18 + (P)^{½}}{4 + (P)^{½}}$$

Where P is Population Equivalent in 1,000 PE

Infiltration

Water other than wastewater that enters a sewage collection system (including sewer service connections) from the ground through such sources as defective pipes, pipe joints, connections, or manholes. Infiltration does not include, and is distinguished from, inflow.

Inflow

Water other than wastewater that enters a sewage collection system (including sewer service connections) from sources such as roof leaders, cellar drains, yard drains, area drains, foundation drains, drains from springs and swampy areas, manhole covers, cross connections between storm sewers and sanitary sewers, catch basins, cooling towers, storm water, surface runoff, street wash waters, or drainage. Inflow does not include, and is distinguished from, infiltration.

Population Equivalent (P.E.)

The term used to evaluate the impact of industrial or other waste on a treatment works or sanitary sewer. One population equivalent is 100 gallons of sewage per day, roughly the amount generated by a typical resident.

Pumping Station

A structure that contains pumps, piping, valves and other mechanical and electrical equipment for pumping wastewater.

Reserve Capacity (gpm) –

This column subtracts the total peak flow from the full flow capacity to determine the reserve capacity in each section of pipe.

Reserve Capacity (with E.F. Pumps) (gpm) –

This column adds the 2,500 gpm capacity of the excess flow pumps to the reserve capacity to determine reserve capacity of the system.

Surcharged

Condition where the wastewater flow depth is above the top of the sewer.

Total Peak Flow (gpm)

This column adds the peak dry weather flow and the 10-year I/I flow to get the total design peak flow for each section of trunk sewer.

Trunk Sewer

Large diameter sewer collecting wastewater from a region and conveying it to a wastewater treatment plant. For this report, trunk sewers were considered to be those twelve inches in diameter or larger.

Wastewater Collection System

The network of sewers, manholes, pumping stations and force mains that collect and convey wastewater to a wastewater treatment plant.

EXECUTIVE SUMMARY

Scope of Water and Sanitary Sewer Master Plan

The purpose of a Water & Sewer Master Plan is three-fold: 1) to assess a community's water and sanitary sewer infrastructure's ability to adequately serve the current demands of the community, 2) to provide a systematic plan to expand a community's water and sanitary sewer infrastructure to address any identified deficiencies, and 3) to provide a systematic plan to expand a community's water & sanitary sewer infrastructure to meet future needs of areas planned to be served by a community.

The Village's latest Water Master Plan was completed by Baxter & Woodman, Inc. in 2007 & the latest Sanitary Sewer Master Plan was completed by Baxter & Woodman, Inc. in 2004 and included the areas generally bound by Ashland Avenue to the west, one-half mile north of Eagle Lake Road to the north, Cottage Grove Avenue to the east and Corning Road to the south. Since the time of the 2007 Water & Sanitary Sewer Master Plan report, the Village has experienced significant population growth and the Illiana Expressway and the Chicagoland third airport are both in planning stages. The Village has contracted Baxter & Woodman, Inc. to complete a Water & Sanitary Sewer Master Plan Update with an expanded Planning Area which includes the areas generally bound by Crawford Avenue to the west, Offner Road to the North, Stateline Road to the east, and County Line Road to the south.

General Items

1. The Village's current planning area is approximately 3,900 acres, and the new planning area is approximately 36,500 acres.

2. This Master Plan includes proposed water and sanitary sewer demands based upon projected P.E. loadings of the areas in the Planning Area as presented in the 2008 Eastern Will County Wastewater Planning Study. Using data provided in this report, the plan can easily be modified to reflect changing development patterns and water supply needs.

Water Master Plan Executive Summary

1. The Village of Beecher operates and maintains three water supply wells, one elevated storage tank, one ground storage tank with associated booster pumping station, and a water distribution system that includes approximately 32 miles of water main.

2. The current water supply capacity and storage capacity are adequate to meet the immediate needs of the Village.

3. The older water mains in the Village have a history of more frequent water main breaks and the Village's efforts to replace these water mains should be continued.

4. Numerous additional wells and storage facilities will be required to serve the future growth within the Planning Area. The locations of the new facilities should be planned out as development occurs.

5. The existing well supply capacity is adequate to serve the immediate needs of the Village until the maximum daily water demand reaches approximately 2.88 million gallons per day. New water supply wells can be expected to serve between 3,400 and 6,800 people depending on the capacity of the well. The locations of the new wells should be strategically located to coincide with areas of new developments but must be located at least a half mile from one another to prevent interference. We recommend a groundwater capacity evaluation to determine the long term capacity of the limestone aquifer.

6. The existing storage capacity is deficient by approximately 150,000 gallons. This deficiency is currently being supplemented by the excess well capacity, which is adequate for the short term; however, the Village must begin planning for a new storage facility in the near future. Depending on the capacity of the new storage facilities, they can be expected to serve approximately 12,000 to 24,000 people. As with the new wells, the storage facilities should also coincide with areas of new development.

7. The impact on existing facilities from additional water demands generated by future growth will be minimal as long as additional facilities are constructed as needed. Improvements to the existing distribution system may be necessary to accommodate immediate growth near the outer edges of the Village to ensure adequate flow and pressures are available further away from the existing supply and storage. This must be evaluated on a case-by-case basis depending on the size and type of developments planned.

8. The distribution system to serve the entire Planning Area should consist of a network of large diameter distribution mains located on no greater than a one mile by one mile grid. The sizing of the distribution mains must be evaluated as development occurs and should be selected based on several factors including size of development, type of development, distance for supply and storage facilities, and future considerations. Smaller diameter water mains can be used to serve local developments as needed. A separate pressure zone will need to be created to serve the far northwest corner of the Planning Area.

Sanitary Sewer Master Plan Executive Summary:

1. The Village of Beecher operates one wastewater treatment plant (WWTP), and maintains a sanitary sewer system that includes four pumping stations. The Village's current sanitary collection system and treatment plant has capacity to meet the current needs of the Village; however numerous trunk sewers and the treatment plant are nearing capacity.

2. Trunk sewer upsizing (or installation of relief sewers) and a wastewater treatment plant expansion are projects needed in the immediate future to ensure the Village can continue to meet the sanitary sewer service needs of the Village.

3. The older sewers in the Village (the sewers constructed in the 1960s and 1970s) have significant I/I (as high as 25 times the Average Daily Flow (ADF) during a 2-year storm event) and the Village's efforts to identify and remove the I/I from these sewers should be continued. The location of these sewers are generally located upstream of the Excess Flow Pump Station and west of Fireman's Park.

4. Four new regional pumping stations, in addition to four existing stations, are required to serve the future growth within the Planning Area. The locations of the new regional stations were determined to maximize the use of gravity sewers and to minimize the number of new pumping stations. Temporary pump stations may be needed in the future to accommodate the actual development pattern until the regional facilities can be constructed. These should be evaluated as development occurs.

5. The impact on existing trunk sewers and pumping stations from wastewater flows generated by future growth was evaluated. Several improvements to the existing system are necessary to accommodate future flow rates:
   a. The Excess Flow Pump Station will need to be upgraded to a permanent submersible station with more capacity.
   b. The 18-inch Trim Creek Trunk Sewer from manhole 19, (located on the East side of Trim Creek between Indiana Ave. and Pasadena Ave) to the WWTP will need to be supplemented to provide capacity for future development. A 21-inch relief sewer should be constructed in two phases to provide additional capacity.
   c. One section of the 8-inch sewer and one section of the 12-inch sewer on Indiana Avenue will need to be supplemented with a 12-inch sewer to accommodate the Cardinal Creek development. Ultimately, an 18-inch sewer will be required from Highlington Court to the WWTP to convey the flow from Cardinal Creek Pumping Station.

1. INTRODUCTION

1.1 Water System Study Purpose and Scope

The water system portion of this report analyzes and evaluates the capability of the Village of Beecher's water supply, storage, and distribution facilities to meet the present and future water needs of the Village. This report expands the Village of Beecher's Water System Planning Report Update (February 2007) to include the area that may be served as a result of two potential projects that would have a dramatic impact on the region: the third airport and the Illiana Expressway. The water system section of the report consists of the following:

1. Assessment of the current and future water demands.

2. Evaluation of the capabilities of the existing supply, storage, and distribution facilities to meet the current and future needs of the Village.

3. Identification and evaluation of necessary improvements to the supply and storage systems to meet the present and future needs of the Village.

4. Conceptual plan of necessary improvements to the distribution system to support future growth.

1.2 Sanitary Sewer System Study Purpose and Scope

The sanitary sewer evaluation analyzes the capability of the Village of Beecher's wastewater collection system to serve the present and future needs of the Village. Similar to the water system evaluation, this report updates the Sanitary Sewer Master Plan (2004) to determine the impact of the third regional airport and Illiana Expressway. The purposes of the sewer system section of the report are to:

1. Evaluate the adequacy of the Village of Beecher's existing wastewater collection system to handle the existing wastewater flows.

2. Determine the general location, sizes, and capacities of the trunk sewers, pumping stations, and treatment plants to serve future growth, and develop opinions of probable cost for the trunk sewer improvements.

3. Analyze the impact of future development on the existing wastewater collection system, including determining the size and opinion of probable cost of relief sewers.

The Sanitary Sewer System Master Plan is intended to serve as a planning tool for the Village of Beecher. This report will provide the sizes and general locations of the sanitary trunk sewers, pumping stations, treatment plants, and force mains to serve future growth within the Planning

Area. This plan will be valuable to the Village during pre-annexation and preliminary platting discussions with developers to identify the sanitary sewer requirements to accommodate these future developments.

Minimizing the number of new pumping stations added to the sanitary sewer system is critical to the Village of Beecher. The Sanitary Sewer Master Plan was developed to identify the locations of regional pumping stations so as to minimize the number of new pumping stations.

1.3 Location and Planning Area

The Village of Beecher is located in the southeast portion of Will County near the Indiana state line. The Village is 38 miles south of the Chicago Loop and is located on Illinois Route 1 (Dixie Highway) which runs north-south through the central part of the Village.

The existing Beecher Facilities Planning Area (FPA) was established to provide the planning boundaries for the Village's wastewater treatment facilities and consists of the Village of Beecher and certain unincorporated areas in Washington Township, Will County, totaling about 3,900 acres. The Planning Boundary for this report is the area considered for water and wastewater service, which is generally described as the area encompassing the following limits: County Line Road to the south, Offner Road to the north, Crawford Avenue to the west, and State Line Road to the east. This update will expand the original study area from approximately 3,900 acres to 36,500 acres. Exhibit A delineates this boundary.

The Village of Beecher has not participated fully in the growth that occurred in suburbs closer to Chicago because of its location at the outer limits of the Chicago metropolitan area. However, the Village has been growing at an increased rate over the past ten years. The U.S. Census Bureau data showing the Village's population for the 40-year period from 1970 to 2010 is shown in Table 1 below. Factors that may loom important to Beecher's future growth and development the potential for the Illiana Expressway to the south and nearby location of Chicago's third regional airport immediately to the north and west of the Village of Beecher.

TABLE 1

U.S. Census Bureau Data

Year	Population
1970	1,770
1980	2,024
1990	2,032
2000	2,033
2010	4,359

2. WATER SYSTEM MASTER PLAN UPDATE

2.1 Background Information

The Village is presently served by three limestone wells which provide an excellent, inexpensive source of water. Although all three wells are capable of supplying enough water to meet the present demand, growth within the Planning Boundary will require additional sources of water in the future. The existing storage and distribution system are satisfactorily serving the Village, but will need to be expanded to meet the future growth within the Planning Boundary.

The Village of Beecher owns and operates the existing water supply, storage, and distribution facilities. The Village employs three licensed operators to oversee the operation of these facilities. The Village is a municipal corporation of the State of Illinois, and is the agency responsible for the facilities planning of the water system.

2.2 Water Consumption and Demand

The evaluation of the normal water consumption and the peak demand for water is an essential step in the planning of water system improvements. A water system must perform two vital services for a community, including: (1) supplying clean, safe water in sufficient quantities to meet the everyday needs of the residential, commercial, institutional, and industrial consumers in the service area, and (2) delivering water to any part of the system to meet the fire protection requirements of that particular area.

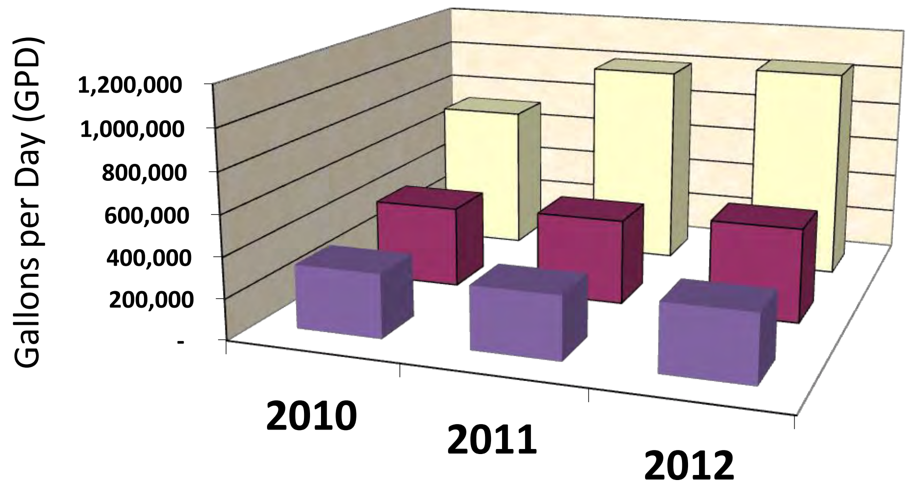
2.2.1 Residential, Commercial, Institutional, and Industrial Consumption

Village Water Consumption – The normal consumption for a community consists of the quantity of water needed for residential, commercial, institutional, and industrial customers. This includes not only the quantity of water actually metered and billed to each customer, but also the amount of water which leaks out of the mains, water which is not registered on individual meters, and the amount of water used for public needs by the municipality. The annual water consumption of a municipality is equal to the total water pumpage in a year.

The actual water consumption summary for the years 2010, 2011, and 2012 is shown in Figure 1.

FIGURE 1

Existing Water Consumption



	2010	2011	2012
Total Billed Amount (gpd)	314,442	309,929	331,815
Average Daily Pumpage (gpd)	401,688	420,416	464,516
Maximum Daily Pumpage (gpd)	724,000	1,002,000	1,047,000

Village records historically indicated that approximately 84% of the annual water pumped is billed to residential customers, leaving approximately 16% to the commercial, institutional, and industrial customers. Using these figures, the average per capita consumption for the years 2010 through 2012 varied from a low of 77 gallons per day in 2010 to a high of 89 gallons per day in 2012. This is for residential use only.

For planning purposes, residential drinking water demands are typically based on 70 gpcd and wastewater flows are based on 100 gpcd; a 70% ratio. In Beecher, this ratio ranged from a low of 55% in 2011 to 89% in 2012 with an average of 69% over the three-year period. This Master Plan uses proposed wastewater flows based upon projected P.E. loadings of the areas in the Planning Area as presented in the 2008 Eastern Will County Wastewater Planning Study. Therefore, the future residential pumpage was calculated as 70% of the planned wastewater flow as determined in the Planning Study.

The future residential consumption was based on the same ratios for pumpage versus consumption as the 2010 through 2012 period. The future commercial, institutional, and industrial consumption projections were calculated based on the increase being proportional to the residential growth. It should be noted that these anticipated increases in commercial, institutional and industrial demands may vary significantly depending on the type of establishments that are built in the future and their water requirements. Should a large water-using industry build in the community, an adjustment to the future projections may be necessary.

Unbilled Water – Comparison of the total pumpage to the total billed amount for 2010 through 2012 reveals that approximately 74.5 percent of the water pumped is being billed. The 25.5 percent unbilled, which equals about 110,145 gallons per day, is unusually high and can be attributed to several factors. These factors include leakage in the distribution system, water main breaks, hydrant flushing, old meters, and unmetered water use by the Village. Replacing water mains in the Village, which are up to 95 years old has and will continue to reduce leakage from the distribution system and also reduce the amount of water lost during main breaks. Looping the distribution system will continue to improve circulation, which may reduce the frequency of hydrant flushing.

Maximum Day and Peak Hour Consumption – In addition to supplying the average daily consumption, a water supply system must also be capable of delivering the water demands during maximum days and peak hours. The maximum days and peak hours typically occur during dry weather periods in the summer. As can be seen in Figure 1, the maximum daily consumption to average daily consumption ratio for the years 2010 through 2012 varied from a low of 1.80 in 2010 to a high of 2.38 in 2011 with a three-year average of 2.15 times the average daily consumption. Experience in other communities indicates that a peak hourly rate of 1.5 times the maximum daily rate would be realistic.

2.2.2 Fire Demand

Fire Protection – In addition to providing water for the domestic, commercial, institutional and industrial needs of a community, a water supply system must also provide water for fire protection. Because the water required for fire protection must be supplied at very high flow rates for relatively short periods of time, fire demand is an extremely important factor in water system planning. The Insurance Services Office (ISO) establishes the fire protection requirements for a community and evaluates the capability of the community's water system and fire department to meet those requirements.

There are many considerations that ISO uses when evaluating a community's firefighting capabilities. In addition to water supply, ISO considers the size of the fire department, number of pumper engines, telephone and alarm system, and firefighting procedures in their evaluation. Thus, providing the highest fire flow requirement will not necessarily ensure the most desirable insurance rate for a community. There are also general guidelines that are used in planning studies to estimate how much water is needed to fight fires.

Fire Flow Rates – ISO required fire flow rates vary by specific building based upon building materials, size, use, type, separation, and the type of materials stored in the building. For this reason, it is difficult to generalize about ISO fire flow requirements for buildings in the Village.

However, commonly used values for generalized building types can be used to approximate fire flow requirements, and are provided in Table 2 below.

TABLE 2

Fire Flow Requirements

Type of Development	Rate (gpm)	Duration (hours)
Single Family Residential	1,500	2
Multiple Family Residential	2,500	2
Commercial/Business	2,500	2
Office/Research	2,500	2
Public/Institutional (Schools, etc.)	3,000	3
General Manufacturing	3,000	3
Large Scale Manufacturing	3,500	3

Based on potential for commercial and industrial growth in the Village, we recommend providing a maximum fire flow rate of 3,500 gpm, the most conservative flow that will be needed. This will ensure that the Village is prepared to provide adequate fire protection for future commercial and industrial growth.

Maximum Water Demands **–** The maximum water demands for the past three years within the Village and those anticipated for the entire planning area are summarized in Table 3. These are the rates on which the existing system should be judged, and for which the proposed improvements should be designed. For instance, the supply facilities should be able to provide the rate of use on a maximum day; the storage facilities should be able to provide the rate of use during a peak hour, in addition to providing the fire demand rate and also reserve capacity; and the distribution facilities should be able to supply the rate of use on a maximum day in addition to the fire demand rate.

TABLE 3

Summary of Existing and Future Water Demands

Items	2010	2011	2012	25% Ultimate	50% Ultimate	Ultimate
Population[1]	4,359	4,332	4,394			
Avg. Daily Consumption (gpd)[2]	401,688	420,416	464,516	3,771,000	7,541,000	15,082,000
Max. Daily Consumption (gpd)[3]	724,000	1,002,000	1,047,000	8,107,000	16,213,000	32,427,000
Rate of Use on Max. Day (gpm)	503	696	727	5,630	11,259	22,519
Rate of Use on Peak Hour (gpm)[4]	755	1,044	1,091	8,445	16,889	33,779
Fire Demand Rates						
Recommended Max. Rate (gpm)	3,500	3,500	3,500	3,500	3,500	3,500
Max. Demand Rate						
w/ Recommended Fire Demand (gpm)	4,003	4,196	4,227	9,130	14,759	26,019

[1] 2010 population data from U.S. Census data. 2011 and 2012 population data estimated based on number of service connections as reported by the Village.
[2] 2010 – 2012 data from Village records for water pumped. Ultimate data calculated based on the Eastern Will County Wastewater Planning Study.
[3] 2010 – 2012 data from Village records. Ultimate data calculated using a Max. Day to Avg. Day ratio of 2.15.
[4] Peak hour use calculated using a Peak Hour to Max. Day ratio of 1.5.

2.3 Description of Existing Facilities

2.3.1 Supply Facilities

The Village's existing water supply comes from three limestone wells. A summary of the physical features of each well is provided in Table 4.

TABLE 4

Summary of Physical Features of Existing Wells

Item	Well No. 3	Well No. 4	Well No. 5
Year Drilled	1987	1993	2006
Depth (feet)	500	565	575
Diameter (inches)	16	10	20
Casing (diameter)	16" to 117'	12" to 124'	16" to 128'
Pumping Equipment	Lineshaft	Lineshaft	Submersible
Motor Horsepower	75	50	75
Capacity (gpm)	1,000	500	1,000

Well No. 3 **–** This well is located on Romans Road, southwest of the intersection of Church Road and Dixie Highway, in the far north central section of the Village. Treatment provided at this well consists of chlorination and fluoridation. In addition, Aquamag, an ortho/polyphosphate, is added to the water to sequester iron and prevent deposition within the water supply system.

Located at Well No. 3 is a 350 KVA generator driven by an 850cc Cummins Diesel engine, which provides an alternate power source in the event of a power failure. This generator is located inside the well house and was purchased when Well No. 3 was put into service in 1987. Village staff performs regular maintenance on the generator to ensure that it is operable in the event of a power outage.

Well No. 4 **–** This well is located next to the elevated tank on Gould Street between Hodges Street and Penfield Street in the west central section of the Village. Treatment provided at this well consists of chlorination and fluoridation. In addition, Aquamag is added to the water to sequester iron and prevent deposition within the water supply system. This well was designed to be able to be upgraded to be a 1,000 gpm well in the future.

Well No. 5 **–** This well is located on Rolling Pass Road in the northeast section of the Village. Treatment provided at this well consists of chlorination and fluoridation. In addition, Aquamag is added to the water to sequester iron and prevent deposition within the water supply system.

There is no generator at Well No. 5, however there is a pigtail electrical connection the Village uses to connect a rental generator to provide an alternate power source in the event of a power failure. The Village rents a generator from Godwin Pumps to power this well during power failures.

Well Operation – All of the Village's three wells are presently operating. Well No. 3 supplies water to a ground storage tank and is controlled by the elevation in that storage tank. Well No. 4 and Well No. 5 supply water directly to the distribution system and are controlled based on system pressures. A Supervisory Control and Data Acquisition (SCADA) system controls and monitors the operation of the water supply system using a radio telemetry system.

2.3.2 Storage Facilities

The storage facilities consist of a 100,000 gallon elevated tank, located adjacent to Well No. 4, west of Gould Street and between Hodges Street and Penfield Street. The tank was constructed in 1958 with a ground elevation of 736.13 feet, USGS datum. The bowl of the tank measures 28 feet in diameter and has a low water elevation of 844-feet and an overflow elevation 870 feet. The tank was repainted in 2001.

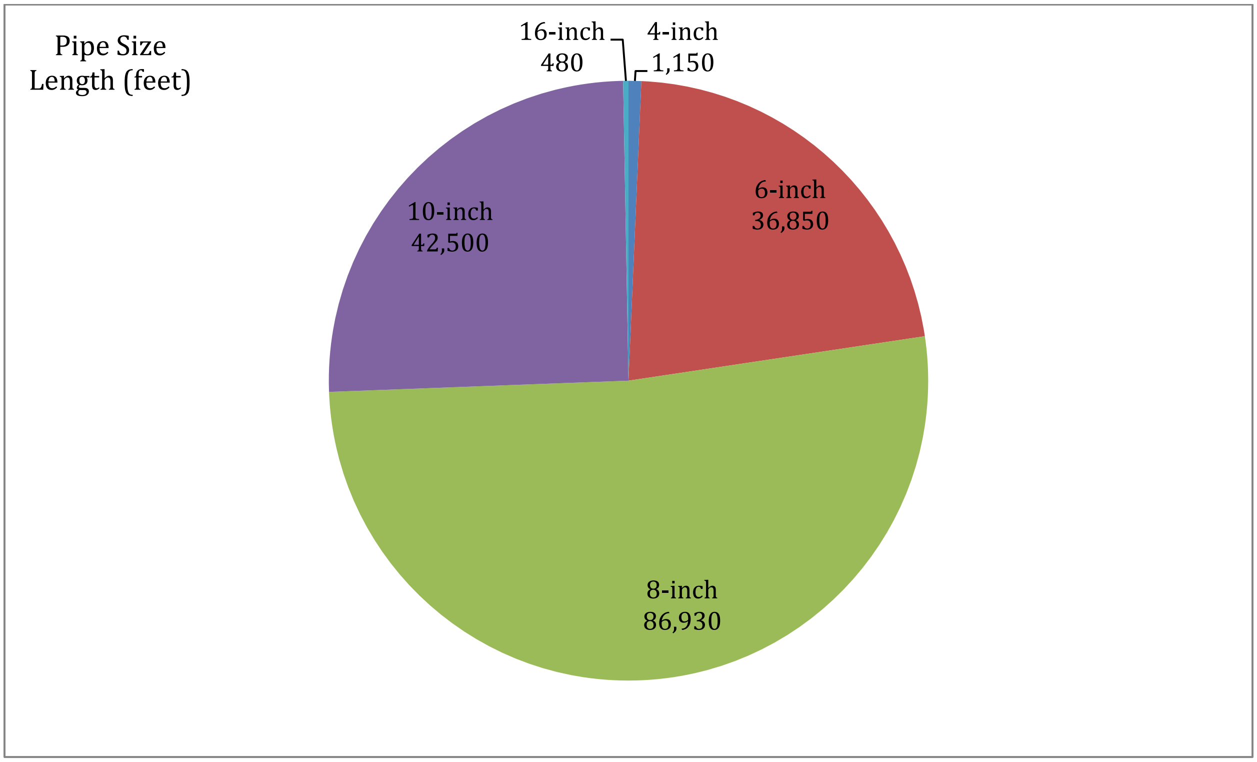
The Village also owns a 750,000 gallon ground storage tank, located at the southwest corner of Dixie Highway and Church Road, adjacent to Well No. 3. The tank was constructed in 2002 with a base elevation of 727 feet. The tank is a circular wire-wound, prestressed concrete tank, 59.5 feet in diameter and 36 feet in height (base to tank overflow). The tank utilizes a pumping station with two slow start pumps at a flow rate of 500 gpm each. A 2,000-gpm pump located at the pumping station is used to provide additional fire flow to the system.

2.3.3 Water Distribution System

The original water distribution system was constructed in the early 1900's and since that time has been extended to provide water service to the new areas of the Village. The present system consists of approximately 32 miles of water mains with shut-off valves and fire hydrants. The older water mains were constructed of sand cast pipe while the new mains are PVC pipe. Approximately 4,300-feet of 6-inch pipe has been replaced with 8-inch pipe in the past six years. In addition, new hydrants have been installed to replace old hydrants that do not have auxiliary valves and would not be reliable in the event of an emergency. The existing distribution system is shown on Exhibit E. An approximate breakdown of the distribution system pipe sizes is provided in Figure 2 below.

FIGURE 2

Summary of Water Distribution System



In past years it was not uncommon for the Village to experience 30 water main breaks in one year. However, recently the number of main breaks has decreased to approximately twenty per year. This decrease in main breaks can be attributed to several factors. One factor is the installation of two surge suppressors consisting of a fitting which provides an air cushion to reduce the effect of surges. These surge suppressors are located on the Pasadena Avenue water main just east of Maxwell Street and on the Dunbar Street water main just south of Penfield Street. The decrease in water main breaks may even more so be attributed to recent system improvements such as the replacement of old sand cast mains with new PVC mains, the interconnecting of dead end mains at strategic locations, and the use of steady flow mechanisms such as variable flow devices (VFD).

In past years, the Village water department received numerous water quality complaints from its customers. The majority of complaints were in regard to rusty water which resulted from poor circulation due to the dead end mains existing in the system. Over the last three years, the Village has received an average of three complaints per year. The water complaints were random with no specific geography and the complaints were usually the result of an internal plumbing issue such as an inoperable softener or water filter. This decrease in complaints can be directly attributed to the Village's recent improvements to the system such as the elimination of dead end mains. In addition, the Village feeds Aquamag, an ortho/polyphosphate, to sequester iron and prevent deposition.

Both of these enhancements have decreased the frequency that the Village flushes the mains, which have significantly reduced time expended by the operators as well the amount of water loss.

2.4 Evaluation of Existing Facilities

2.4.1 Supply Facilities

The supply facilities should be capable of providing the maximum daily demand of a community. It is common practice to evaluate the supply capacity assuming the largest unit is out of service, or the 'firm' capacity. The present combined capacity of the Village's three wells is 2,500-gpm and the 'firm' capacity is 1,500-gpm which is greater than the 2012 maximum daily demand of 727-gpm (see Table 3). Thus, the existing supply facilities are currently adequate to meet water consumption demands in the Village. Maximum daily demand projections (Table 3) indicate that additional supply facilities will be required to meet future water consumption demands. The deficiencies of the present supply system to meet future needs of the Village are summarized in Table 5 below.

TABLE 5

Supply Capacity Evaluation

Year	Population	Supply Capacity (gpm)	Firm Capacity (gpm)	Max. Daily Demand (gpm)	Deficiency (gpm)
2010	4,359	2,500	1,500	503	0
2011	4,332	2,500	1,500	696	0
2012	4,394	2,500	1,500	727	0
25% of Ultimate		2,500	1,500	5,630	4,130
50% of Ultimate		2,500	1,500	11,259	9,759
Ultimate		2,500	1,500	22,519	21,019

The above analysis reveals that numerous additional water supply wells will be required to meet the demands of the entire planning area. Well No. 4 was designed to be upgraded to 1,000 gpm in the future, which would increase the firm capacity to 2,000 gpm. This would provide sufficient capacity until the maximum daily water demand reaches 2.88 million gallons per day (MGD) or the Village population reaches approximately 13,600 people. There is currently no need to construct additional supply capacity at this time.

To meet fifty percent of the ultimate demand of the entire planning area, the Village will need a total of 10 to 20 additional wells depending on their output. There are two options to consider for the locations of the future wells. The first option would be to centrally locate the wells and pump the water out to the new areas of development. The second option is to locate the new wells in the vicinity of the new areas of development. Due to the large area to be served, the best option would be to locate the new wells near the new areas of development so the water in these areas will be fresh. Spacing the wells at consistent intervals will also eliminate the need to provide oversized transmission mains.

The Village's supply system provides to its customers water which meets all current water quality standards. The past water quality complaints received by the Village are a result of iron in the water and aggressive water causing corrosion within the mains and services. The Village is currently feeding Aquamag, an ortho/polyphosphate, into the system to sequester iron and inhibit corrosion. Aquamag sequesters iron by holding the iron in solution by means of a chemical reaction, which prevents the iron from settling out in the distribution system. Over a period of time, the chemical bond which holds the iron in suspension will break and allow the dissolved iron to be oxidized primarily by chlorine in the water supply and turned to a particle. These particles will then settle out in the main and become a source of red or rusty water complaints as water begins to flow again. This is of particular concern in dead end mains where water age becomes very high and water use is sporadic. Aquamag controls corrosion by the making the metal pipes become passive, which prevents the pipes from corroding.

Even though the hardness and iron concentrations in the Village's water supply are within the water quality standards, they do exceed what normally is considered aesthetically acceptable. Residents who wish to have better water quality have installed water softeners in their homes to reduce the hardness and iron concentrations. The Village has chosen not to provide softening and iron removal at the water source. Iron removal filters and ion exchange softeners at each of the three wells would require significant capital and operating expenditures, which would considerably increase the rates charged by the Village to supply water.

2.4.2 Storage Facilities

Water storage facilities provide the following: (1) water to meet the peak hourly demands, (2) water for fire protection, and (3) a reserve capacity for emergencies such as periods when the supply system is inoperable and when the tank is not completely full.

Peak Hourly Demand – The existing and future projected peak hourly demands for the Village are given in Table 3. The supply capacity should be capable of providing the maximum daily demand. Therefore, the storage facilities must provide the difference between the maximum daily rate and the maximum hourly rate. We recommend that the storage volume be sufficient to provide the difference between the maximum day rate and the maximum hourly rate for at least an eight hour period. Table 6 provides the required volume of water required to meet this eight-hour demand.

Fire Flow Demand – The storage volume required for fire protection is dependent on the fire flow rate and duration. The maximum fire flow rate is recommended to be 3,500-gpm for a duration of three hours to account for future commercial and industrial development. The required storage volume is currently 630,000 gallons (3,500-gpm x 180 minutes). Table 6 provides the required volume of water required to meet the fire flow demand.

***Reserve Capacity* –** The third purpose of storage is to provide a reserve supply of water to meet the demands during emergencies and to provide capacity in the event of a fire occurring when the tank is partially full. We recommend reserving 20 percent of the capacity of an elevated tank for this purpose. Table 6 provides the required volume of water required for reserve capacity supply.

***Required Storage* –** Table 6 presents the total required storage requirements for the Village from now through the ultimate development of the entire planning area. This analysis shows that the Village's current storage capacity is deficient by approximately 150,000 gallons, however, this deficiency can be made up for by the excess supply capacity. Using the excess supply capacity is acceptable for the short term, however, the Village should start planning to construct an additional storage tank in the near future. The next storage tank should be an elevated tank for redundancy. By the time the entire planning area is developed, the Village's storage capacity will be deficient by approximately 6.7 million gallons.

The additional storage necessary should be implemented in stages as the water system grows. Depending on the rate of development, elevated tanks of 500,000 to 1,000,000 gallons of capacity should be used. Larger tanks would tend to lead to water age issues and poor turnover, while smaller tanks would lead to additional maintenance because more tanks would be needed. The new towers should be located near the new areas of development to provide more reliable supply times and pressures during emergencies.

TABLE 6

Summary of Storage Volume Requirements

Items	2010	2011	2012	25% Ultimate	50% Ultimate	Ultimate
Population	4,359	4,332	4,394			
8-hour Maximum Demand						
Rate of Use on Max. Day (gpm)	503	696	727	5,630	11,259	22,519
Rate of Use on Peak Hour (gpm)	755	1,044	1,091	8,445	16,889	33,779
Required Storage Volume (gallons)	120,960	167,040	174,720	1,351,200	2,702,400	5,404,800
Fire Flow Demand						
Recommended Max. Rate (gpm)	3,500	3,500	3,500	3,500	3,500	3,500
Required Storage Volume (gallons)	630,000	630,000	630,000	630,000	630,000	630,000
Reserve Capacity (gallons)	187,740	199,260	201,180	495,300	833,100	1,508,700
Total Storage Volume Required (gallons)	938,700	996,300	1,005,900	2,476,500	4,165,500	7,543,500
Existing Storage Volume (gallons)	850,000	850,000	850,000	850,000	850,000	850,000
Storage Volume Deficiency (gallons)	88,700	146,300	155,900	1,626,500	3,315,500	6,693,500

2.4.3 Water Distribution System

Computer Model – The capability of the existing water distribution system to deliver the required flows to various sections of the Village was evaluated using the WaterGEMS computer program. The required input data for WaterGEMS is divided into two parts, including: (1) the physical characteristics of the system, and (2) water consumption data.

The development of a computer model to simulate the water distribution system has several advantages. First, it can be used to determine the delivery capability at any point in the system without conducting hydrant tests. Second, the model can be utilized to identify inadequate water mains and evaluate proposed improvements to correct these inadequacies. Third, it can be used in the future to evaluate the adequacy of proposed water mains in new developments and to assess the impact of proposed developments on the existing system. We will keep this model on file for future use in evaluating the Village's distribution system.

Physical Characteristics – The physical characteristics of the system were determined from the Village's water main atlas, USGS maps, and consultation with Village staff. Size, length, and location of each water main were entered into the model. Digital topographic data was used to determine the ground elevations at various locations in the distribution system. Village staff previously assisted in determining the age and material of the water mains. This data was used to determine a roughness coefficient, or C-factor, for each of the pipes. A basic geographic information systems (GIS) database was created to aid in the WaterGEMS model creation and can be used by the Village as a starting point for a more in-depth GIS system if desired.

Water Consumption – The average and maximum daily water consumption demands were distributed throughout the system based on estimated flow to various sub-basins.

Calibration – The model was previously calibrated using hydrant tests by the Insurance Services Office (ISO) and by Baxter & Woodman, Inc. Re-calibration of the model was not included as part of this report. The calibration of the model has previously resulted in an accurate representation of the existing distribution system.

Evaluation of Existing System – In order to evaluate the adequacy of the existing distribution system, the model was run using maximum daily and fire flow demands. The fire flows were based on providing recommended fire flow rates as summarized in Table 3, which range from 1,500-gpm in residential areas to 3,500-gpm in public institution and general manufacturing areas.

The results of the computer model simulations indicate that the distribution system can generally deliver the recommended fire flow rates to most residential and commercial areas. However, deficiencies in the existing public institution and manufacturing areas were identified. These areas of deficiencies were identified in the previous report and the recommended improvements from that report should continue to be implemented to eliminate these deficiencies.

2.5 Evaluation of Alternative Improvements

2.5.1 Supply Facility Improvements

Numerous additional water supply wells will be required to meet the demands of the entire planning area. Currently, groundwater is the only feasible option for water supply for the Village. Surface water from Lake Michigan, the closest available sustainable surface water source, is not economically or politically feasible at this time.

Groundwater may be drawn from the shallow sand and gravel aquifer, the limestone aquifer or the deep sandstone aquifer. The shallow aquifer is self-replenishing and requires less power, but is more difficult to locate and typically provides less production per well. It is unlikely to drill a shallow well with sufficient capacity for municipal purposes in Eastern Will County. The existing wells are in the limestone aquifer, which is typically more productive and more consistent, but has higher drilling and operational cost and may require iron removal. The deep sandstone aquifer may produce yields high enough for municipal use but are costly to drill and operate. In addition, the deep aquifer likely contains radium which would need to be treated and removed prior to the distribution system. As the Village expands, an extensive well investigation study should be conducted to determine the potential locations for future wells and the long term capacity of the limestone aquifer.

Assuming each new well is capable of producing between 500 and 1,000 gpm, approximately 10 to 20 additional wells will be required to meet the demands of fifty percent of the entire planning area. New wells should be located in the vicinity of areas of new development as they occur. Although locating the exact site for each of these wells can not be done at this time, for planning purposes the Village should locate the wells at approximately ½ mile radius from each other. Each new well can serve anywhere from 3,400 to 6,800 people depending on the capacity of the well.

2.5.2 Storage Facility Improvements

Numerous additional storage facilities will be required to provide the required storage for the entire planning area. Storage facilities can either be elevated or ground storage tanks. Elevated storage tanks are typically more reliable and simple to operate, but cost more to construct and maintain. Ground storage tanks are less reliable but are cheaper to construct and maintain. As additional storage is required, all factors should be considered when deciding on which type of storage would be best suited for the Village's needs.

Assuming each new storage facility is between 500,000 and 1,000,000 gallons, anywhere from 4 to 8 additional storage facilities will be required to provide the required storage of fifty percent of the entire planning area. As with the wells, locating each of these new storage facilities at this time would be impractical. The new storage facilities should be located in the vicinity of areas of new development. The storage facilities should be spread out throughout the planning area to provide adequate flow and pressure to all areas. For planning purposes, assume a new storage facility will be required for every 12,000 to 24,000 people depending on the storage volume.

2.5.3 Distribution System Improvements

The distribution system improvements required to serve the planning area depend entirely on the development that occurs. As development occurs, new water mains will need to be installed to supply water to areas that were previously unserved. The size of the water main required to serve new areas will depend on several factors including the location of the development, the size of the development, and the type of development.

In general, a network of large diameter distribution mains along the major roads, at no greater than a one mile by one mile grid, with smaller local mains to serve local roads would be ideal. Preliminary water modeling indicates the use of 12-inch diameter distribution mains, however, larger 16-inch or 20-inch diameter distribution mains may be required in the event of a large user or the distance from supply or storage facilities is excessive. Undersizing the water mains may limit future development and lead to low flow and pressure throughout the new development whereas oversizing the water mains may lead to water age and quality problems. As new development occurs, a thorough distribution system analysis should be performed to determine the proper pipe sizing.

Overall, a majority of the planning area can be served with one pressure zone. However, a separate pressure zone will be required in the northwest portion of the planning area due to higher elevations. This area generally includes the land north of West Eagle Lake Road and west of South Ashland Avenue and includes elevations above 760 feet. A booster pumping station will be required to boost the water pressure from the lower zone into the upper zone along with a series of pressure reducing valves to allow water from the upper zone back into the lower zone. At least one storage tank is recommended within this upper pressure zone to provide emergency storage in the event that the booster pumping station fails.

A conceptual plan for the ultimate water system layout is shown on Exhibit F. This plan is conceptual only based on the recommended spacing for new facilities and future commercial/industrial water users that are consistent with existing use. Future improvements must be planned on a case-by-case basis and the overall concept plan should continually be updated as development occurs.

3. SEWER SYSTEM MASTER PLAN UPDATE

3.1 Existing Conditions

3.1.1 Topography and Drainage

The Village of Beecher, and most of its planning area, is located in the watershed of Trim Creek, Pike Creek, and Exline Slough. The tributaries of these watersheds, help to create three regional sewer planning areas. The regional sewer planning areas are shown on Exhibit C.

The West Region is generally delineated by Crawford Avenue to the west, County Line Road to the South, Ashland Avenue to the east, and Offner Road to the north. The Central region is generally delineated by Ashland Avenue to the west, County Line Road to the south, Cottage Grove to the east, and Offner Road to the north. The East Region is generally delineated by Cottage Grove to the west, County Line Road to the south, State Line Road to the east and Church Road to the north.

The Central Region includes the existing areas that are served by the sanitary sewer, which are all tributary to the Trim Creek Trunk Sewer. All of the existing sewered areas of the Village drain to the tributary of Trim Creek which flows though the center of the Village.

Sanitary sewers are generally installed to follow the natural topography of the service area. Pumping stations are installed at low points of the service area to pump wastewater to higher elevations so that it can flow by gravity to the treatment plant, without requiring substantial cuts to install the sewers. The low points in Beecher's topography are located on streams along the northern, southern and eastern boundaries of the Planning Area. As a result, pumping stations are required in the northern, eastern, and southern portions of the planning area to pump wastewater to the Village's treatment facilities.

3.1.2 Existing Population

The 2010 census indicated the residential population of Beecher was 4,359. Of this population, nearly 100 percent was served by the current sanitary sewer system.

3.1.3 Existing Treatment Works

The Village of Beecher currently operates one WWTP. The plant is located on the east bank of the west tributary of Trim Creek approximately one quarter of a mile south of Indiana Avenue on Ahrens Drive. It has a design capacity of 6,000 P.E. with a design average flow of 0.6 MGD and a design maximum flow of 1.5 MGD. Originally constructed in 1963, the WWTP was upgraded in 1996. The WWTP property is large enough to expand the WWTP to double its capacity, which would be a total design capacity of 12,000 P.E. with a design average flow of 1.2 MGD and a design maximum flow of 3.0 MGD.

3.1.4 Pumping Stations

The existing collection system includes four pumping stations as shown on Exhibit B. The year of construction, the type and number of pumps, and the capacity of the existing pumping stations are listed below:

TABLE 7

Existing Pumping Stations

Pumping Station	Year Constructed	Type	Number of Pumps	Capacity (gpm)
Cardinal Creek	2005	Submersible	2	500/1240*
Trim Creek-Excess Flow	2000	Portable	1	2,500
Fairway Drive	1980	Submersible	2	80
Miller Street	1970's	Submersible	2	75

* Cardinal Creek Pumping Station existing pumps have a capacity of 500 gpm and per the annexation agreement, the developer is required to upsize these pumps to 1240 gpm pumps upon issuance of the 251st building permit for the subdivision.

3.1.5 Existing Sanitary Sewer System

This section evaluates the ability of the existing sanitary sewer system to convey current flows to the WWTP. Exhibit B shows the location of the existing sanitary sewers, trunk sewers, force mains and pumping stations.

Beecher's original collection system was constructed in the 1960's. In general, the construction materials used for sewers installed in this era were of poorer quality than the construction materials for newer sewers. In addition, sewers were not inspected during construction as rigorously as they are today. As a result, older sewers tend to be more susceptible to inflow and infiltration (I/I) than the newer ones. Large amounts of I/I in a sanitary sewer system can result in flows that exceed the capacity of the sewers, resulting in basement backups or discharge of raw sewage through surcharged manholes. Similarly, peak wastewater flows resulting from I/I can exceed the capacity of the WWTP, causing the release of partially treated sewage into the receiving creek. In addition to its environmental impact, I/I reduces the effective capacity of the collection system and treatment works. An analysis of the amount of I/I received by Beecher's sanitary sewer system and its effect on the sanitary sewer system was included in the Sanitary Sewer Master Plan dated February 2004.

Flow data for the existing trunk sewers is summarized in Table 2. Pipe capacities were determined using available data for the size and slope of the sewer and the previously completed I/I study. If current information for the slope was not available, record drawings were used to estimate the existing slope. Average Daily Dry Weather Flow is considered the average daily flow rate recorded during a five day dry weather period. Peak Daily Dry Weather Flow is the highest rate that was recorded during a dry weather period, which typically occurs in the morning during the highest water usage period

Appendix C further examines the reserve capacity in the following existing trunk sewers: the 12-inch Cardinal Creek Trunk Sewer, the 12-inch Church Road Trunk Sewer, the 12-inch Dutch American Way Trunk Sewer, the 8-inch and 12-inch Indiana Avenue Trunk Sewer, and the 15-inch and 18-inch Trim Creek Trunk Sewer. The capacity of each trunk sewer is examined for both existing and future service areas. The system was analyzed from the WWTP upstream to Eagle Lake Road. Location, size, length, slope, and full flow capacity information is shown for each section of trunk sewer.

3.1.6 Conclusions

The following conclusions can be drawn from the analysis of the flow monitoring data and reserve capacity evaluation:

1. The 15-inch trunk sewer at Miller Street and Catalpa Street, the 8-inch sewer located in Fireman's Park, and the 18-inch trunk sewer north of the WWTP each receive large amounts of I/I during storm events.

2. The newer trunk sewers along Trim Creek and Church Road receive less I/I per tributary footage than the collector sewers in the older areas. This can be attributed to the better construction materials and inspection procedures used for the installation of these sewers.

3. A flow monitoring program should be conducted every few years to monitor wastewater flow rates and reserve capacities, and will allow the analysis of the impact of sewer rehabilitation.

4. All three flow monitoring locations experienced surcharged conditions. During the two-year event on May 9, 2003, the depth of the 15-inch on Miller was 10.5 feet deep, the 8-inch in Fireman's Park was 7.25 feet deep, and the 18-inch north of the WWTP was 12.75 feet deep. Village staff has indicated that there are no known overflows of sanitary sewage to the ground surface. However, basement backups do occur during high flows. If not addressed, this will only worsen as new development is incorporated into the sewer system upstream.

5. The peak flow to the 15-inch Trim Creek Trunk Sewer south of MH 19 (located on the east side of Trim Creek in the alley between Indiana Ave. and Pasadena Ave.) exceeds its capacity. I/I reduction or construction of a supplemental sewer are two alternatives available to provide capacity for future upstream development. Relief sewer alternatives will be discussed in more detail in Section 3.4.

3.2 Future Expansion of Sanitary Sewer System

3.2.1 Introduction

The second major goal of the Sanitary Sewer Master Plan is to determine the locations, sizes, capacities of trunk sewers, treatment plants and pumping stations to serve future growth. This goal is achieved through the following series of steps: (1) identify Beecher's ultimate sanitary sewer service area (referred to in this report as the "Planning Area"); (2) determine potential land use within the area that is not currently served by sanitary sewers; (3) determine the location of future trunk sewers, pumping stations and treatment plants to serve these areas; (4) calculate the flow rates to the proposed trunk sewers, pumping stations and treatment plants; (5) determine sizes for the trunk sewers, pumping stations and treatment plants to serve future growth areas; and (6) determine the cost of the proposed trunk sewers based on sewer size and depth.

This report provides only the projected size and general location of new trunk sewers, pumping stations and treatment plants needed to serve future growth. The actual location of these regional facilities will depend on layouts of future streets and the type, size, and configuration of future developments.

3.2.2 Planning Area and Population

The Eastern Will County Wastewater Planning Study was used as the source of the water and wastewater flow demands. The P.E. of each sub-basin of the Planning Area is shown on Exhibit D.

A detailed breakdown of the projected land use, service area, and P.E. for each sub-basin in the planning area is included in Appendix A. This Appendix also includes a summary of the P.E. and flow rates for each sub basin. These values should be updated as areas develop to determine the actual impact on the proposed sanitary sewer system.

The total sanitary sewer loading of the areas containing sanitary sewers and areas the Village has committed to provide sanitary sewer service is approximately 9,100 P.E. Future growth areas represent potentially an additional 105,000 P.E. Thus, the total future growth within the Planning Area could increase the population served by sanitary sewers to a total of 114,100 P.E.

3.2.3 Sub-Basins

For the purpose of determining service areas for future trunk sewers and pumping stations, the Planning Area was divided into a number of regional basins and sub-basins. Regional basin boundaries were determined using natural drainage patterns and man-made boundaries. The basins delineate the service areas of existing and proposed trunk sewers and pumping stations. The regional basins were divided into smaller sub-basins to allow for planning of a more refined future trunk sewer network. Sub-basin boundaries were determined by a number of factors, including: natural drainage patterns, service area size, and potential development phasing. The regional basin and sub-basin boundaries and labels are shown on Exhibit D.

A labeling convention was established to identify each of the sub-basins. These labels will be used throughout this report to refer to specific areas of the Planning Area. A sub-basin label consists of a letter and a number (the letter indicates the regional basin).

3.2.4 Location of New Facilities

Exhibit C shows the location of the proposed trunk sewers, pumping stations, treatment plants and force mains needed to serve the entire Planning Area. The Village should preserve corridors to allow for the construction of these facilities as development occurs.

Although it is preferable to minimize the number of pumping stations, they are sometimes the most cost effective means of conveying flow to the WWTP. The alternative to pumping stations is deeper trunk sewers. The factors to consider when evaluating pumping stations and deep sewers are: (1) the cost of installing sewers increases significantly with depth, (2) safety for installation crews decreases dramatically with increased depth, and (3) maintenance on deep sewers can be expensive and dangerous with traditional open-cut methods. In order to mitigate these impacts, while still minimizing the number of pumping stations, trunk sewers were planned for a maximum depth of twenty-five feet.

The proposed trunk sewer sizes and elevations shown on Appendix B are sized to accommodate the ultimate anticipated wastewater flows for the entire Planning Area. The size and depth of each proposed trunk sewer was selected not only to accommodate the ultimate anticipated wastewater flows, but also to minimize the number of pumping stations. As areas tributary to these proposed trunk sewers begin to develop, temporary smaller sewers at shallower depths may be considered to be constructed to be more cost accommodating to developers.

3.2.5 Sizing of New Facilities

Using the calculated P.E. for each sub-basin, sizes were determined for the new trunk sewers and pumping stations. Exhibit C shows the location of the trunk sewers, pumping stations, treatment plants and force mains needed to serve the Planning Area.

Appendix B is a summary of the trunk sewers sizes and pumping station capacities required to serve the entire Planning Area. The tributary P.E. and peak flow rate for each trunk sewer section is also provided. Proposed trunk sewers were sized to flow at a maximum of 67 percent capacity for the projected peak flow rate for trunk sewers from 12-inch to 18-inch diameter and 80 to 90 percent capacity for trunk sewers over 21-inch diameter. The ultimate sewer system sizing and layout was determined based on the Eastern Will County Wastewater Planning Study and the available topographical data, and can be easily modified as these factors are updated.

Trunk sewers were profiled at depths ranging from ten feet to 25 feet. This provides adequate depth to service the entire sub-basin, while minimizing utility conflicts, cost and safety concerns. For the scope of the sewer master plan the trunk sewer design slope, elevation and size were based on the most recent topographical data from United States Geological Survey (USGS) maps. The final design slope, elevation, and size of the trunk sewers should be verified with a topographical survey of the ground along the proposed sewers.

In addition to sewer sizing, opinions of probable cost have been determined for the proposed trunk sewers and relief sewers. These costs incorporated a number of factors, including sewer diameter and depth, number of manholes, trench backfill and surface restoration requirements. Based on the percentages seen for recently constructed trunk sewers in the area, it was estimated that 70 percent of the length of a proposed trunk sewer will not require granular trench backfill and 30 percent will be located in areas requiring granular backfill. In addition, it was anticipated that 90 percent of the length of the trunk sewers will require grass or sod restoration and 10 percent will require pavement restoration. The final location, i.e. in parkway or under pavement or in an open area, will not be determined until final design plans are prepared. Appendix F includes opinions of probable cost for the proposed trunk sewers and a table summarizing the unit prices for a range of sewer diameters at various depth ranges and locations. Opinions of probable cost are not included for the proposed pumping stations and treatment plants because of the many unknown factors that can impact the cost.

3.2.6 Service Regions

West Region - This region includes the area generally bound by Ashland Avenue to the east, Crawford Avenue to the west, Offner Road to the north, and County Line Road to the south. A discussion of the proposed trunks sewers, pumping stations and sub-basins to be served are as follows:

1. **Exline Slough North Trunk Sewer –** (Basins S117A, B, C, D; S117F, H) This trunk sewer begins at the intersection of Kentucky Road and Exline Slough and continues north along Exline Slough to Eagle Lake Road.

2. **Church Road West Trunk Sewer –** (Basins 117E; S103A, B; S103C, D, E; S101B, C) This trunk sewer begins just north of Indiana Avenue tying into the Exline Slough North Trunk Sewer and continues northwest to Eagle Lake Road.

3. **Indiana Avenue West Trunk Sewer –** (Basin S117G) This trunk sewer begins at the Exline Slough North Trunk Sewer at Indiana Avenue and continues east along Indiana Avenue to just west of Ashland Avenue.

4. **Plum Creek Pump Station –** (Basins S100A & S81A) This 390 gpm pumping station will serve the northwestern most portion of the Planning Area. This station should be located in the northeast corner of basin S100A. This pumping station will discharge through a six-inch force main to the Exline Slough North Trunk Sewer South Plant Trunk Sewer.

5. **Exline Slough East Trunk Sewer –** (Basins S121A, B, C, & D) This trunk sewer starts at the Exline Slough pumping station and continues east along the east branch of Exline Slough ending about one-half mile east of Western Avenue, just north of Kentucky Road.

6. **Exline Slough South Trunk Sewer –** (Basin S156A) This trunk sewer begins at the Kentucky Road Pumping Station and continues south along Exline Slough ending near County Line Road.

7. **Kentucky Road Pumping Station** - (Basins S156A, Exline Slough East Trunk Sewer and Exline Slough North Trunk Sewer) This 4,100 gpm pumping station will serve the west portion of the Planning Area. This station should be located on the northeast corner of basins S156A. This pumping station will discharge through an eighteen inch force main along Kentucky Road to WWTP 2.

Central Region - This region includes the area tributary to Trim Creek watershed. This area is generally bound by County Line Road to the South, Offner Road to the north, Cottage Grove to the east and Ashland Ave to the west. A discussion of the proposed trunk sewers, pumping stations and sub-basins to be served are as follows:

1. **WWTP No. 2 Trunk Sewer** – (Basins S137B; S116A, B, C, D, E, F, & G) This trunk sewer begins at WWTP No. 2 and continues northeast along Trim Creek to Eagle Lake Road.

2. **Wastewater Treatment Plant No. 2** – (Basin 137A, Kentucky Road Pumping Station, County Line Road Pumping Station) This future treatment plant is approximately on-half mile east of Ashland Avenue on Kentucky Road at Trim Creek.

3. **Trim Creek West Trunk Sewer** – (Basins S111A, B, & C) This trunk sewer begins on the WWTP No. 2 Trunk Sewer along Trim Creek at Corning Road and continues northwest along the west leg of Trim Creek to approximately one-half mile north of Indiana Avenue.

4. **Trim Creek Northwest Trunk Sewer** – (Basins S91A, B, C, & D) This trunk sewer begins at the north end of the WWTP No. 2 Trunk Sewer located at approximately Cottage Grove and Eagle Lake Road and continues northwest along Trim Creek and ends at South Park Avenue.

5. **Trim Creek North East Trunk Sewer** – (Basins S106G, K, J, I, F, H, E, D, B, & C) This trunk sewer begins at the north end of the WWTP No. 2 Trunk Sewer located at approximately Cottage Grove and Eagle Lake Road and continues southeast along Trim Creek and ends near Church Road.

6. **WWTP 1 Bypass Sewer** – This trunk sewer begins at WWTP 1 and terminates at its connection to MH 3 of the WWTP 2 Trunk Sewer. This sewer follows Trim Creek and will be used to send all flow from WWTP 1 to WWTP 2.

East Region - This region includes the area that will be tributary to the existing Pike Creek watershed. A discussion of the proposed trunk sewers and sub-basins to be served are as follows:

1. **County Line Road Pumping Station** (Basins S139A, Pike Creek Trunk Sewer, County Line Road Trunk Sewer) **-** This 5,300 gpm pumping station will serve the eastern most portion of the Planning Area. This station should be located in the southwest corner of basin S141A on County Line Road adjacent to Pike Creek. This pumping station will discharge through an 18-inch force main to the WWTP No. 2.

2. **Pike Creek Trunk Sewer** – (Basins S141B, C, D, E, F, G, H, I, J, K, L, & M) This trunk sewer begins at the County Line Road Pumping Station and continues north along Pike Creek to approximately one-half mile east of Stoney Island and one-half mile north of Indiana Avenue.

3. **County Line Road Trunk Sewer** – (Basins S141B; S163A, B, C; S130A) This trunk sewer begins at the County Line Road Pumping Station and continues east along County Line Road ending at State Line Road.

4. **State Line Pumping Station** – (Basins S126C, D; S106A & B) This 950 gpm pumping station will serve the southeast most portion of the Planning Area. This station should be located on the northeast corner of basins S126C. This pumping station will discharge through an eight inch force main along State Line Road to the east end of the County Line Road Trunk Sewer.

3.2.7 Conclusions

Exhibit C shows the general location of the new trunk sewers and pumping stations required to serve the planning area. This exhibit will serve as a planning tool for the Village of Beecher to determine the sizes and locations of sanitary sewer facilities required to serve future developments. Trunk sewer design parameters are summarized in Appendix B, while opinions of probable costs are in Appendix D. The number of pumping stations to serve the planning area has been minimized. A consequence of the Village's approach to minimizing the number of new pumping stations is that small developments may not be able to connect to Beecher's sanitary sewer system until a larger development can provide the required infrastructure to serve an entire region unless a temporary pump station is approved by the Village. Table 8 summarizes the capacities for each of the proposed pumping stations.

TABLE 8

Proposed Pumping Stations – Sizes and Capacities

Pumping Station	Connected P.E.	Peak Hourly Flow Rate (gpm)	Force Main Size (inches)
Plum Creek	1,500	390	6-inch
State Line	4,100	950	8-inch
Kentucky Road	23,000	4,100	18-inch
County Line Road	30,500	5,300	18-inch

3.3 Impact of Future Growth on Existing Sanitary Sewer System

3.3.1 Introduction

This section evaluates the impact that future growth will have on the existing sanitary sewer system. The existing trunk sewers, pumping stations, and waste water treatment plant are discussed in terms of the capacity to handle additional flows from growth in sub-basins already served by existing sewers, as well as growth in areas not presently served by sewers.

The capacities of the existing trunk sewers were determined from the size and slope obtained from Village records. When existing records did not indicate the existing slope, minimum slopes were used to calculate the sewer's capacity. Trunk sewers are normally designed at 80 percent capacity, which allows an additional safety factor should growth in an area exceed current projections. The existing trunk sewers were evaluated at 100 percent capacity, but any relief sewers which may be needed were sized at approximately 80 percent capacity. Relief sewers are shown on Exhibit C. Appendix C provides a detailed summary of the projected flow rates to each section of the existing trunk sewer system, as well as the capacity and size of any relief sewers needed to supplement the existing sewer.

3.3.2 Hunter's Chase and Church Road Trunk Sewers

These trunk sewers extend east along Church Road from Dutch American Way to the Hunter's Chase subdivision at Fox Hound Trail and then north and east through the subdivision to its terminus at the north property line at Quail Hollow Drive. This 12-inch sewer currently provides service to sub-basins TC16 and TC20.

A detailed analysis of the capacity of this sewer is provided in Appendix E. It was estimated that this trunk sewer is not subject to significant amounts of I/I because this sewer and those tributary to it have been recently constructed using pipe materials and installation techniques that minimize I/I. In the future, this trunk sewer will receive flow from sub-basins TC21 and future developments in sub-basin TC20. There is sufficient capacity in these trunk sewers to handle the projected total of 1,722 P.E. (435 gpm peak). Development densities in the tributary service area should be monitored to determine if a relief sewer is required.

3.3.3 Dutch American Way Trunk Sewer

This trunk sewer extends north through the Trim Creek Industrial Park along Dutch American Way. This 12-inch sewer has a capacity of 875 gpm and provides service to sub-basin TC14.

This sewer has sufficient capacity to convey the existing flow rate. The industrial development in this sub-basin is only partially built out and will experience substantial growth in the future. However, despite the additional growth to the north, the trunk sewer has sufficient capacity to convey the ultimate flow rate of 130 gpm (470 P.E.).

3.3.4 Trim Creek Industrial Park Trunk Sewer

This trunk sewer commences just north of Catalpa Street and extends north along Trim Creek Tributary to Church Road. This 15-inch sewer was constructed in 1998 to serve the industrial park along Dutch American Way. Due to its recent construction, this sewer is probably not subject to significant amounts of I/I. The trunk sewer has a capacity of 1,120 gpm peak, which is sufficient to convey the existing flow demand. Sub-basins TC18 and TC 19 are also tributary to this sewer and are not yet developed. Upon build-out of sub-basins TC 18 and TC19 the total peak flow through this sewer will be 1,026 gpm, which will leave 94 gpm of available capacity. Generally speaking, once area TC18 and TC19 are developed, this sewer will be approximately at its full capacity.

3.3.5 Cardinal Creek Drive Trunk Sewer

The Cardinal Creek Drive Trunk Sewer commences at the Cardinal Creek Pumping Station located one block north of the intersection of Indiana Avenue and Cardinal Creek Drive and extends north along Cardinal Creek drive to Church Road. This 12-inch sewer was constructed in the 2000s. The trunk sewer has a capacity ranging from 1,011 gpm near Church Road and 2,981 gpm closer to the pumping station where the sewer upsizes to 18-inch. The capacity of each section of this sewer is identified in Appendix C. This sewer has capacity to handle the all of the flow from Sub-Basins H9, H8, H7, H6, H5, H4, H3, H2 in the build-out condition, however this sewer has no additional capacity to convey flows from any additional development without installation of a relief sewer.

3.3.6 Trim Creek Trunk Sewer (Catalpa to Excess Flow Pump Station)

The upstream section of the Trim Creek Trunk Sewer commences at the Excess Flow Pumping Station located at Beecher Grade School and extends north to Catalpa Street. Currently, this 15-inch sewer receives all of the flow from the north portion of the Village, including the Miller Street Pumping Station, less Sub-Basins H9 through H1.

The flow capacity analysis of this trunk sewer includes additional flow for the I/I measured in the 2003 flow monitoring project. The capacity of each section of this sewer is identified in Appendix C. Upon build-out of the upstream area, it is projected that approximately 7,402 P.E. will be tributary to this sewer. An 18-inch relief sewer has been installed from MH 166 (located at Country Lane and Catalpa St) south to the Excess Flow Pumping Station and together these sewers have capacity to convey the total calculated flow rates in the built-out condition.

3.3.7 Trim Creek Trunk Sewer (Excess Flow Pump Station to WWTP)

The downstream section of the Trim Creek Trunk Sewer commences at the Excess Flow Pumping Station located at Beecher Grade School and extends south to the WWTP. Currently, this 15-inch sewer receives all of the flow from the Village, except for the Excess Flow Pumping Station which pumps directly to the WWTP. The Excess Flow Pumping Station has a capacity of 2,500 gpm with a 12-inch force main installed along Trim Creek from the Pumping Station to WWTP 1.

The flow capacity analysis of this trunk sewer includes additional flow for the I/I measured in the 2003 flow monitoring project. The capacity of each section of this sewer is identified in Appendix C. Upon build-out of the upstream area, it is projected that approximately 9,816 P.E. will be tributary to this sewer. This 15-inch trunk sewer is undersized and relief sewers are required. The following discussion describes the recommended phasing of relief sewers to accommodate the expected flow from this area.

By analyzing the reserve capacities in the 18-inch trunk sewer, it is apparent that the relief sewer could be constructed in two phases since the downstream portion of the sewer has less capacity than the upstream portion. Phase 1 would be to install a relief sewer from Manhole 19 (located midblock between Indiana Avenue and Pasadena Avenue) to the WWTP. This section of trunk sewer currently is undersized. Phase 2 would be to install a relief sewer to supplement the sewer from Manhole 19 north to Manhole 49 (located on Elm Street on the east side of Trim Creek). The sanitary sewer loading calculations show that the sewer between manholes 49 to 19 will be undersized by less than 300 gpm in the built-out condition, (which is less than the capacity of an 8-inch sewer at minimum slope) – because this calculated undersize is relatively so small, we do not recommend planning to build a relief sewer for this section of sewer and rather recommend that the flows in these sewers are monitored after development occurs and then decide if a relief sewer is required.

Phase 1 requires a 24-inch trunk sewer from the WWTP to manhole 19. Due to the routing difficulty, it would be more feasible to install the sewer west on Pasadena from manhole 18 to the existing 8-inch sewer on Maxwell Street. The Cardinal Creek development is required by annexation agreement to install a 12-inch sewer to supplement this section of trunk sewer for their development. This sewer should be increased to 24-inches by having the Village install the sewer and accepting cash-in-lieu from the developer or by reimbursing the developer for oversizing.

Phase 2 requires a 24-inch relief trunk sewer along Maxwell Street to manhole 72 located at Elm Street. As mentioned above, we recommend monitoring the flows in the sewers after the majority of the development has occurred in the existing planning area to determine if this Phase 2 relief sewer will be required.

3.3.8 Indiana Avenue Trunk Sewer

The Cardinal Creek Pumping Station discharges to an 8-inch sewer at Highlington Court. Between manhole 215 and manhole 25 this sewer has a slope of 0.4 percent and a capacity of 340 gpm, which is equivalent to approximately 1,315 P.E. After issuance of the 251st building permit, the developer of Cardinal Creek is required to increase the size of the sewer from manhole 215 to manhole 25 to a 12-inch pipe at 1.5% which has a capacity of 1,958 gpm (9,200 P.E.). This will provide sufficient capacity for the entire development west of manhole 25.

When additional development occurs that is tributary to the Cardinal Creek Pumping Station, a new trunk sewer will be required from manhole 215 at Highlington Court to the WWTP, as shown on Exhibit C.

3.3.9 Pump Stations

Excess Flow Pumping Station - This pumping station is located at the west side of the Beecher Grade School and east of Fireman's Park. The pump station consists of a portable pump with a capacity of 2,500 gpm. In order for the wet well to receive flow, the depth of flow in the adjacent sewer must rise above an overflow weir. Thus, this pumping station only operates during wet weather conditions.

The Village has indicated that this pumping station will be converted to a submersible pumping station in the future. Our estimate of probable cost to install 2 submersible pumps, wet well level management system, valve vault, control enclosure and controls, and SCADA upgrades is $450,000, including Engineering and Contingency.

Miller Street Pumping Station - This pumping station is located at Miller Street and Lange Avenue in Lange's Subdivision. This duplex submersible pumping station is equipped with two (2) 75 gpm pumps. The reliable capacity of this pumping station with the largest pump out of service is 75 gpm, which is equivalent to 265 P.E. This pumping station receives flow from a portion of sub-basin TC13 and pumps it through a 4-inch force main to an 8-inch sewer on Miller Street. The tributary area to this pumping station is built out and should not receive a significant increase in flow in the future

Abandoning this pumping station would require construction of a sewer across the golf course, which is an unlikely alternative at this time.

Fairway Drive Pumping Station - This pumping station is located on Fairway Drive in the Highlington Estates Subdivision. This duplex submersible pumping station is equipped with two (2) 80 gpm pumps. The reliable capacity of this pumping station with the largest pump out of service is 80 gpm, which is equivalent to 280 P.E. This pumping station receives flow from a portion of sub-basin TC17 and pumps it through a 4-inch force main to an 8-inch sewer on Miller Street. The tributary area to this pumping station is partially built out and will not receive flow from additional sub-basins in the future.

Similar to the Miller Street Pumping Station, abandoning this pump station would require a sewer across the golf course, which is considered unlikely at this time.

3.3.10 Waste Water Treatment Plants

The Village of Beecher currently operates one WWTP #1. The plant is located on the east bank of the west tributary of Trim Creek approximately one quarter of a mile south of Indiana Avenue on Ahrens Drive. It has a design capacity of 6,000 P.E. with a design average flow of 0.6 MGD and a design maximum flow of 1.5 MGD. The WWTP #1 property is large enough to expand the WWTP #1 to double its capacity, which would be a total design capacity of 12,000 P.E. with a design average flow of 1.2 MGD and a design maximum flow of 3.0 MGD.

Upon build-out of the areas the Village has currently committed to provide water and sewer service to (Areas TC and H, shown on Exhibit B) the calculated P.E. loading to WWTP #1 is calculated to be approximately 13,000 P.E. and therefore we recommend WWTP #1 is expanded as large as the WWTP #1 footprint will allow, which is approximately 12,000 P.E. The total calculated P.E. loading for the entire Planning Area is approximately 114,000 P.E. and therefore an additional future WWTP #2 will also need to be constructed in the future as development occurs. The location of the future WWTP #2 is recommended to be constructed adjacent to Trim Creek on Kentucky Road. Once the future WWTP #2 is constructed we recommend that all wastewater is conveyed to the future WWTP #2 via a gravity trunk sewer and that the existing WWTP #1 is used as an excess flow facility.

As previously mentioned, when the areas that the Village has currently committed to serve are built out (Areas TC and H, shown on Exhibit B) the flows from these areas will take up all the available capacity of existing WWTP #1 after it is expanded. Accordingly, any new developments outside of the existing Village limits will need to flow to the new WWTP #2. WWTP #2 will need to be constructed in phases as development occurs. After WWTP #2 is constructed, then at a time convenient to the Village a trunk sewer can be installed to connect WWTP #1 to WWTP #2 so that all Village waste water is treated at WWTP #2 and WWTP #1 will then only be used for excess flow storage.

3.3.11 Conclusions

Tributary P.E and Flow Rates to WWTPs - Establishing a plan to convey wastewater to the treatment plant is of immediate concern to the Village because of several factors including: projected development growth in the near future, existing trunk sewers nearing capacity, and planning of a WWTP expansion. Construction of the Third airport or the Illiana Expressway would also result in increased sanitary sewer loadings on the Village's sewer system from population growth and commercial/industrial development.

As previously mentioned, upon build-out of the areas of the Village has committed to provide sewer service to (Areas TC and H, shown on Exhibit B) the calculated P.E. loading to WWTP #1 is calculated to be approximately 13,000 P.E. and therefore we recommend WWTP #1 is expanded as large as the WWTP #1 footprint will allow, which is approximately 12,000 P.E. The wastewater from any additional developments will need to be conveyed to and treated at a new WWTP #2. Once the new WWTP #2 is constructed, all Village wastewater should be treated at WWTP #2 and WWTP #1 should be used as an excess flow facility. Wastewater from WWTP #1 will flow to WWTP #2 via a trunk sewer along Trim Creek.

Relief Sewers and Pumping Station Upgrades - Several of the existing trunk sewers do not have capacity to handle ultimate flow rates. Construction of relief sewers is one alternative to convey the ultimate flow rate to the treatment plant. It is possible to decrease the size of the relief sewers by minimizing I/I. Procedures to reduce I/I are discussed in the 2004 Flow Monitoring Report. The improvements necessary to accommodate future flow rates without I/I reduction are summarized below.

1. The existing 15-inch Trim Creek Trunk Sewer upstream of the Excess Flow Pumping Station does not have reserve capacity to handle existing wet weather and future flow rates. A parallel, 18-inch relief sewer will be required from manhole 167B to manhole 68 to supplement the capacity of this sewer.

2. The existing 15-inch and 18-inch Trim Creek Trunk Sewer downstream of the Excess Flow Pumping Station will not have reserve capacity to handle future flow rates. A 24-inch relief sewer will be required from manhole 68 to the WWTP supplement the capacity of this sewer. This relief sewer can be constructed in two phases.

3.3.12 Recommendations

The following is a summary of the recommended actions to be taken by the Village of Beecher.

1. Adopt this Sanitary Sewer Master Plan and utilize it for the planning and construction of sanitary trunk sewers and pumping stations to serve the future development of the Village.

2. Update this report periodically to reflect changes in key factors such as development philosophy, service area, and modifications to the existing collection system. This will keep the report current for use by the Village.

3. Complete design engineering and construction of the WWTP #1 expansion to 12,000 P.E. as soon as possible. Also establish a method to fund the treatment plant expansion. Two methods would be to secure an IEPA low-interest loan and also collect the pro-rated share from developers who wish to develop property tributary to WWTP #1.

4. Schedule a flow monitoring program every few years to meter flow rates at key locations in the trunk sewers and pumping stations for several month periods.

5. Establish a method to fund the construction of the relief sewers for the Trim Creek Trunk Sewer. One method would be to collect the pro-rated share from developers who wish to develop property tributary to these trunk sewers.

VILLAGE OF BEECHER, ILLINOIS
SANITARY SEWER MASTER PLAN

APPENDIX A

<u>Projected P.E. for each Sub-Basin</u>

The tables included in this appendix summarize the projected P.E from each sub-basin. These tables can be easily updated to reflect actual P.E. and flow rates as developments are constructed.

VILLAGE OF BEECHER, ILLINOIS
SANITARY SEWER MASTER PLAN

Regional and Sub-basin Population Equivalents and Flow Rate Summary

		Existing Conditions			Future Conditions		
			Flow Rate			Flow Rate	
Regional Basin	Sub-basin	PE	Average (GPM)	Peak Day (GPM)	PE	Average (GPM)	Peak Day (GPM)
S-80							
	S-80A	0	0.0	0.0	218	15.2	62.7
	Total	**0**	**0**	**0**	**218**	**15**	**63**
S-81							
	S-81A	0	0.0	0.0	213	14.8	61.2
	Total	**0**	**0**	**0**	**213**	**15**	**61**
S-85							
	S-85A	0	0.0	0.0	305	21.2	86.3
	S-85B	0	0.0	0.0	904	62.8	240.3
	Total	**0**	**0**	**0**	**1,209**	**84**	**327**
S-88							
	S-88A	0	0.0	0.0	933	64.8	247.4
	Total	**0**	**0**	**0**	**933**	**65**	**247**
S-91							
	S-91A	0	0.0	0.0	1,393	96.8	358.2
	S-91B	0	0.0	0.0	933	64.8	247.4
	S-91C	0	0.0	0.0	960	66.7	254.0
	S-91D	0	0.0	0.0	1,385	96.2	356.4
	Total	**0**	**0**	**0**	**4,671**	**324**	**1,216**
S-96							
	S-96A	0	0.0	0.0	1,401	97.3	360.1
	Total	**0**	**0**	**0**	**1,401**	**97**	**360**
S-99							
	S-99A	0	0.0	0.0	1,754	121.8	442.1
	Total	**0**	**0**	**0**	**1,754**	**122**	**442**
S-100							
	S-100A	0	0.0	0.0	1,195	83.0	311.2
	Total	**0**	**0.0**	**0.0**	**1,195**	**83.0**	**311.2**
S-101							
	S-101B	0	0.0	0.0	146	10.1	42.5
	S-101C	0	0.0	0.0	162	11.2	47.0
	Total	**0**	**0**	**0**	**308**	**21**	**90**

VILLAGE OF BEECHER, ILLINOIS
SANITARY SEWER MASTER PLAN

Regional and Sub-basin Population Equivalents and Flow Rate Summary

		Existing Conditions			Future Conditions		
			Flow Rate			Flow Rate	
Regional Basin	Sub-basin	PE	Average (GPM)	Peak Day (GPM)	PE	Average (GPM)	Peak Day (GPM)
S-103							
	S-103A	0	0.0	0.0	120	8.3	35.2
	S-103B	0	0.0	0.0	120	8.3	35.2
	S-103C	0	0.0	0.0	80	5.6	23.7
	S-103D	0	0.0	0.0	162	11.2	46.9
	Total	**0**	**0**	**0**	**482**	**33**	**141**
S-106							
	S-106A	0	0.0	0.0	1,042	72.3	274.0
	S-106B	0	0.0	0.0	1,051	73.0	276.4
	S-106C	0	0.0	0.0	1,045	72.6	274.8
	S-106D	0	0.0	0.0	1,556	108.0	396.3
	S-106E	0	0.0	0.0	1,048	72.8	275.6
	S-106F	0	0.0	0.0	524	36.4	144.2
	S-106G	0	0.0	0.0	1,022	71.0	269.3
	S-106H	0	0.0	0.0	1,035	71.9	272.5
	S-106I	0	0.0	0.0	775	53.8	208.1
	S-106J	0	0.0	0.0	775	53.8	208.1
	S-106K	0	0.0	0.0	1,575	109.4	400.9
	Total	**0**	**0**	**0**	**11,448**	**795**	**3,000**
S-107							
	S-107A	0	0.0	0.0	2,047	142.2	508.7
	Total	**0**	**0.0**	**0.0**	**2,047**	**142.2**	**508.7**
S-111							
	S-111A	0	0.0	0.0	807	56.0	216.2
	S-111B	0	0.0	0.0	2,136	148.3	528.5
	S-111C	0	0.0	0.0	1,770	122.9	445.7
	Total	**0**	**0**	**0**	**4,712**	**327**	**1,190**
S-116							
	S-116A	0	0.0	0.0	1,539	106.9	392.4
	S-116B	0	0.0	0.0	1,542	107.1	393.2
	S-116C	0	0.0	0.0	1,978	137.3	493.0
	S-116D	0	0.0	0.0	768	53.3	206.5
	S-116E	0	0.0	0.0	768	53.3	206.5
	S-116F	0	0.0	0.0	1,539	106.9	392.4
	S-116G	0	0.0	0.0	899	62.4	239.1
	S-116H	0	0.0	0.0	1,578	109.6	401.4
	S-116I	0	0.0	0.0	100	6.9	29.5
	S-116J	0	0.0	0.0	300	20.8	85.0
	S-116K	0	0.0	0.0	120	8.3	35.2
	Total	**0**	**0**	**0**	**11,131**	**737**	**2,724**

VILLAGE OF BEECHER, ILLINOIS
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Regional and Sub-basin Population Equivalents and Flow Rate Summary

Regional Basin	Sub-basin	Existing Conditions PE	Existing Conditions Flow Rate Average (GPM)	Existing Conditions Flow Rate Peak Day (GPM)	Future Conditions PE	Future Conditions Flow Rate Average (GPM)	Future Conditions Flow Rate Peak Day (GPM)
S-117							
	S-117A	0	0.0	0.0	1,372	95.3	353.2
	S-117B	0	0.0	0.0	690	47.9	186.8
	S-117C	0	0.0	0.0	1,374	95.4	353.7
	S-117D	0	0.0	0.0	2,062	143.2	511.9
	S-117E	0	0.0	0.0	1,035	71.9	272.5
	S-117F	0	0.0	0.0	1,039	72.2	273.5
	S-117G	0	0.0	0.0	1,031	71.6	271.4
	S-117H	0	0.0	0.0	1,387	96.3	356.7
	Total	**0**	**0**	**0**	**9,990**	**694**	**2,580**
S-121							
	S-121A	0	0.0	0.0	2,118	147.1	524.5
	S-121B	0	0.0	0.0	805	55.9	215.6
	S-121C	0	0.0	0.0	1,071	74.4	281.1
	S-121D	0	0.0	0.0	2,155	149.6	532.7
	Total	**0**	**0**	**0**	**6,148**	**427**	**1,554**
S-125							
	S-125A	0	0.0	0.0	262	18.2	74.8
	S-125B	0	0.0	0.0	476	33.0	131.6
	S-125D	0	0.0	0.0	274	19.0	77.9
	Total	**0**	**0**	**0**	**1,012**	**70**	**284**
S-126							
	S-126A	0	0.0	0.0	821	57.0	219.6
	S-126B	0	0.0	0.0	943	65.5	249.9
	S-126C	0	0.0	0.0	1,289	89.5	333.6
	S-126D	0	0.0	0.0	1,292	89.7	334.3
	Total	**0**	**0**	**0**	**4,345**	**302**	**1,137**
S-130							
	S-130A	0	0.0	0.0	801	55.6	214.7
	Total	**0**	**0**	**0**	**801**	**56**	**215**
S-137							
	S-137A	0	0.0	0.0	3,517	244.2	826.1
	S-137B	0	0.0	0.0	2,247	156.1	553.4
	S-137C	0	0.0	0.0	2,384	165.5	583.6
	S-137D	0	0.0	0.0	590	41.0	161.4
	Total	**0**	**0**	**0**	**8,738**	**607**	**2,124**

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Regional and Sub-basin Population Equivalents and Flow Rate Summary

		Existing Conditions			Future Conditions		
			Flow Rate			Flow Rate	
Regional Basin	Sub-basin	PE	Average (GPM)	Peak Day (GPM)	PE	Average (GPM)	Peak Day (GPM)
S-139							
	S-139A	0	0.0	0.0	1,749	121.5	441.0
	Total	**0**	**0**	**0**	**1,749**	**121**	**441**
S-141							
	S-141A	0	0.0	0.0	608	42.2	166.0
	S-141B	0	0.0	0.0	1,967	136.6	490.7
	S-141C	0	0.0	0.0	1,240	86.1	321.8
	S-141D	0	0.0	0.0	2,006	139.3	499.3
	S-141E	0	0.0	0.0	1,675	116.3	423.9
	S-141F	0	0.0	0.0	1,548	107.5	394.4
	S-141G	0	0.0	0.0	1,232	85.6	320.0
	S-141H	0	0.0	0.0	1,236	85.8	320.9
	S-141I	0	0.0	0.0	1,244	86.4	322.7
	S-141J	0	0.0	0.0	1,236	85.8	320.9
	S-141K	0	0.0	0.0	620	43.0	168.9
	S-141L	0	0.0	0.0	1,228	85.3	319.0
	S-141M	0	0.0	0.0	3,704	257.2	865.0
	Total	**0**	**0**	**0**	**19,543**	**1,357**	**4,933**
S-142							
	S-142C	0	0.0	0.0	443	30.7	123.0
	S-142D	0	0.0	0.0	437	30.4	121.6
	Total	**0**	**0**	**0**	**880**	**61**	**245**
S-156							
	S-156A	0	0.0	0.0	3,060	212.5	730.0
	Total	**0**	**0**	**0**	**3,060**	**213**	**730**
S-163							
	S-163A	0	0.0	0.0	1,070	74.3	280.8
	S-163B	0	0.0	0.0	1,097	76.2	287.4
	S-163C	0	0.0	0.0	783	54.4	210.2
	Total	**0**	**0**	**0**	**2,949**	**205**	**778**

VILLAGE OF BEECHER, ILLINOIS
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Regional and Sub-basin Population Equivalents and Flow Rate Summary

		Existing Conditions			Future Conditions		
			Flow Rate			Flow Rate	
Regional Basin	Sub-basin	PE	Average (GPM)	Peak Day (GPM)	PE	Average (GPM)	Peak Day (GPM)
TC							
	TC 1	150	10.4	43.7	0	0.0	0.0
	TC 2	98	6.8	28.9	0	0.0	0.0
	TC 3	60	4.1	17.8	0	0.0	0.0
	TC 4	245	17.0	70.0	0	0.0	0.0
	TC 5	186	12.9	53.6	0	0.0	0.0
	TC 6	168	11.7	48.7	0	0.0	0.0
	TC 7	0	0.0	0.0	221	15.3	63.3
	TC 8	151	10.5	43.8	0	0.0	0.0
	TC 9	595	41.3	162.6	0	0.0	0.0
	TC 10	693	48.1	187.4	0	0.0	0.0
	TC 11	683	47.4	184.9	0	0.0	0.0
	TC 12	223	15.5	64.0	100	6.9	29.5
	TC 13	965	67.0	255.3	0	0.0	0.0
	TC 14	300	20.8	85.0	170	11.8	49.3
	TC 15	50	3.5	15.0	0	0.0	0.0
	TC 16	1,898	131.8	474.9	0	0.0	0.0
	TC 17	0	0.0	0.0	350	24.3	98.4
	TC 18	0	0.0	0.0	875	60.8	233.1
	TC 19	0	0.0	0.0	1,059	73.5	278.2
	TC 20	602	41.8	164.4	0	0.0	0.0
	TC 21	0	0.0	0.0	385	26.7	107.7
	Total	**7,065**	**491**	**1,900**	**2,774**	**193**	**752**
H							
	H 1	218	15.1	62.4	0	0.0	0.0
	H 2	336	23.3	94.7	70	4.9	20.8
	H 3	144	10.0	41.8	0	0.0	0.0
	H 4	319	22.1	90.0	0	0.0	0.0
	H 5	308	21.4	87.1	0	0.0	0.0
	H 6	151	10.5	43.8	0	0.0	0.0
	H 7	220	15.3	63.1	100	6.9	29.5
	H 8	0	0.0	0.0	875	60.8	233.1
	H 9	375	26.0	105.0	0	0.0	0.0
	Total	**2,069**	**144**	**588**	**1,045**	**73**	**283**
TOTAL		***9,133***	***634***	***2,488***	***104,949***	***7,270***	***26,867***

Sub-Basin S-80: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM Avg*	*GPM Peak*	*mgd Avg*	*mgd Peak*
S-80A	84.0	5.2	437	218	109	30	121	0.04	0.17
Subtotal	***84.0***		***437***	***218***	***109***				
TOTAL	***84.0***		***437***	***218***	***109***	***30***	***121***	***0.04***	***0.17***
	(Acres)		***(P.E.)***	***(P.E.)***	***(P.E.)***	***(GPM)***	***(GPM)***		

Sub-Basin S-81: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM Avg*	*GPM Peak*	*mgd Avg*	*mgd Peak*
S-81A	282.0	1.5	426	213	106	30	119	0.04	0.17
Subtotal	***282.0***		***426***	***213***	***106***				
TOTAL	***282.0 (Acres)***		***426 (P.E.)***	***213 (P.E.)***	***106 (P.E.)***	***30 (GPM)***	***119 (GPM)***	***0.04***	***0.17***

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-85: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM*		*mgd*	
						Avg	*Peak*	*Avg*	*Peak*
S-85A	84.0	7.3	610	305	152	42	166	0.06	0.24
Subtotal	***84.0***		***610***	***305***	***152***				
S-85B	249.0	7.3	1,808	904	452	126	454	0.18	0.65
Subtotal	***249.0***		***1,808***	***904***	***452***				
TOTAL	***333.0***		***2,418***	***1,209***	***604***	***168***	***591***	***0.24***	***0.85***
	(Acres)		***(P.E.)***	***(P.E.)***	***(P.E.)***	***(GPM)***	***(GPM)***		

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-88: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM*		*mgd*	
						Avg	*Peak*	*Avg*	*Peak*
S-88A	232.0	8.0	1,865	933	466	130	468	0.19	0.67
Subtotal	***232.0***		***1,865***	***933***	***466***				
TOTAL	***232.0***		***1,865***	***933***	***466***	***130***	***468***	***0.19***	***0.67***
	(Acres)		***(P.E.)***	***(P.E.)***	***(P.E.)***	***(GPM)***	***(GPM)***		

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-91: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM*		*MGD*	
						Avg	*Peak*	*Avg*	*Peak*
S-91A	360.0	7.7	2,786	1,393	697	194	671	0.28	0.97
Subtotal	***360.0***		***2,786***	***1,393***	***697***				
A-91B	241.0	7.7	1,865	933	466	130	468	0.19	0.67
Subtotal	***241.0***		***1,865***	***933***	***466***				
S-91C	248.0	7.7	1,920	960	480	133	480	0.19	0.69
Subtotal	***248.0***		***1,920***	***960***	***480***				
S-91D	358.0	7.7	2,771	1,385	693	192	668	0.28	0.96
Subtotal	***358.0***		***2,771***	***1,385***	***693***				
TOTAL	***1,207.0***		***9,342***	***4,671***	***2,336***	***649***	***1,936***	***0.93***	***2.79***
	(Acres)		***(P.E.)***	***(P.E.)***	***(P.E.)***	***(GPM)***	***(GPM)***		

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-96: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM*		*mgd*	
						Avg	*Peak*	*Avg*	*Peak*
S-96A	925.0	3.0	2,803	1,401	701	195	675	0.28	0.97
Subtotal	***925.0***		***2,803***	***1,401***	***701***				
TOTAL	***925.0***		***2,803***	***1,401***	***701***	***195***	***675***	***0.28***	***0.97***
	(Acres)		***(P.E.)***	***(P.E.)***	***(P.E.)***	***(GPM)***	***(GPM)***		

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-99: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM Avg*	*GPM Peak*	*mgd Avg*	*mgd Peak*
S-99A	432.0	8.1	3,508	1,754	877	244	824	0.35	1.19
Subtotal	***432.0***		***3,508***	***1,754***	***877***				
TOTAL	***432.0***		***3,508***	***1,754***	***877***	***244***	***824***	***0.35***	***1.19***
	(Acres)		***(P.E.)***	***(P.E.)***	***(P.E.)***	***(GPM)***	***(GPM)***		

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-100: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*		*GPM*		*mgd*	
							Avg	*Peak*	*Avg*	*Peak*
S-100A	1,390.0	1.7	2,391	1,195	598		166	585	0.24	0.84
Subtotal	***1,390.0***		***2,391***	***1,195***	***598***					
TOTAL	***1,390.0***		***2,391***	***1,195***	***598***		***166***	***585***	***0.24***	***0.84***
	(Acres)		***(P.E.)***	***(P.E.)***	***(P.E.)***		***(GPM)***	***(GPM)***		

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-101: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM*		*MGD*	
						Avg	*Peak*	*Avg*	*Peak*
A-101B	321.0	0.9	292	146	73	20	83	0.03	0.12
Subtotal	***321.0***		***292***	***146***	***73***				
S-101C	356.0	0.9	324	162	81	22	91	0.03	0.13
Subtotal	***356.0***		***324***	***162***	***81***				
TOTAL	***677.0***		***616***	***308***	***154***	***43***	***168***	***0.06***	***0.24***
	(Acres)		***(P.E.)***	***(P.E.)***	***(P.E.)***	***(GPM)***	***(GPM)***		

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-103: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM*		*MGD*	
						Avg	*Peak*	*Avg*	*Peak*
S-103A	240.0	1.0	240	120	60	17	69	0.02	0.10
Subtotal	***240.0***		***240***	***120***	***60***				
A-103B	240.0	1.0	240	120	60	17	69	0.02	0.10
Subtotal	***240.0***		***240***	***120***	***60***				
S-103C	160.0	1.0	160	80	40	11	46	0.02	0.07
Subtotal	***160.0***		***160***	***80***	***40***				
S-103D	323.0	1.0	323	162	81	22	91	0.03	0.13
Subtotal	***323.0***		***323***	***162***	***81***				
S-103E	242.0	1.0	242	121	61	17	69	0.02	0.10
Subtotal	***242.0***		***242***	***121***	***61***				
TOTAL	***1,205.0*** ***(Acres)***		***1,205*** ***(P.E.)***	***603*** ***(P.E.)***	***301*** ***(P.E.)***	***84*** ***(GPM)***	***313*** ***(GPM)***	***0.12***	***0.45***

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-106: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM*		*MGD*	
						Avg	*Peak*	*Avg*	*Peak*
S-106A	320.0	6.5	2,083	1,042	521	145	517	0.21	0.74
Subtotal	***320.0***		***2,083***	***1,042***	***521***				
A-106B	323.0	6.5	2,103	1,051	526	146	521	0.21	0.75
Subtotal	***323.0***		***2,103***	***1,051***	***526***				
S-106C	321.0	6.5	2,090	1,045	522	145	518	0.21	0.75
Subtotal	***321.0***		***2,090***	***1,045***	***522***				
S-106D	478.0	6.5	3,112	1,556	778	216	741	0.31	1.07
Subtotal	***478.0***		***3,112***	***1,556***	***778***				
S-106E	322.0	6.5	2,096	1,048	524	146	520	0.21	0.75
Subtotal	***322.0***		***2,096***	***1,048***	***524***				
S-106F	161.0	6.5	1,048	524	262	73	276	0.10	0.40
Subtotal	***161.0***		***1,048***	***524***	***262***				
A-106G	314.0	6.5	2,044	1,022	511	142	508	0.20	0.73
Subtotal	***314.0***		***2,044***	***1,022***	***511***				
S-106H	318.0	6.5	2,070	1,035	518	144	514	0.21	0.74
Subtotal	***318.0***		***2,070***	***1,035***	***518***				
S-106I	238.0	6.5	1,549	775	387	108	395	0.15	0.57
Subtotal	***238.0***		***1,549***	***775***	***387***				
S-106J	238.0	6.5	1,549	775	387	108	395	0.15	0.57
Subtotal	***238.0***		***1,549***	***775***	***387***				
S-106K	484.0	6.5	3,151	1,575	788	219	749	0.32	1.08
Subtotal	***484.0***		***3,151***	***1,575***	***788***				
TOTAL	***3,517.0 (Acres)***		***22,895.7 (P.E.)***	***11,447.8 (P.E.)***	***5,723.9 (P.E.)***	***1,590 (GPM)***	***4,124 (GPM)***	***2.29***	***5.94***

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-107: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM*		*mgd*	
						Avg	*Peak*	*Avg*	*Peak*
S-107A	414.0	9.9	4,094	2,047	1,024	284	945	0.41	1.36
Subtotal	***414.0***		***4,094***	***2,047***	***1,024***				
TOTAL	***414.0***		***4,094***	***2,047***	***1,024***	***284***	***945***	***0.41***	***1.36***
	(Acres)		***(P.E.)***	***(P.E.)***	***(P.E.)***	***(GPM)***	***(GPM)***		

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-111: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM*		*MGD*	
						Avg	*Peak*	*Avg*	*Peak*
S-111A	238.0	6.8	1,614	807	403	112	410	0.16	0.59
Subtotal	***238.0***		***1,614***	***807***	***403***				
S-111B	630.0	6.8	4,271	2,136	1,068	297	981	0.43	1.41
Subtotal	***630.0***		***4,271***	***2,136***	***1,068***				
S-111C	522.0	6.8	3,539	1,770	885	246	831	0.35	1.20
Subtotal	***522.0***		***3,539***	***1,770***	***885***				
TOTAL	***1,390.0 (Acres)***		***9,424 (P.E.)***	***4,712 (P.E.)***	***2,356 (P.E.)***	***654 (GPM)***	***1,950 (GPM)***	***0.94***	***2.81***

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-116: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM Avg*	*GPM Peak*	*MGD Avg*	*MGD Peak*
S-116A	481.0	6.4	3,078	1,539	770	214	734	0.31	1.06
Subtotal	***481.0***		***3,078***	***1,539***	***770***				
A-116B	482.0	6.4	3,085	1,542	771	214	735	0.31	1.06
Subtotal	***482.0***		***3,085***	***1,542***	***771***				
S-116C	618.0	6.4	3,955	1,978	989	275	917	0.40	1.32
Subtotal	***618.0***		***3,955***	***1,978***	***989***				
S-116D	240.0	6.4	1,536	768	384	107	392	0.15	0.56
Subtotal	***240.0***		***1,536***	***768***	***384***				
S-116E	240.0	6.4	1,536	768	384	107	392	0.15	0.56
Subtotal	***240.0***		***1,536***	***768***	***384***				
S-116F	481.0	6.4	3,078	1,539	770	214	734	0.31	1.06
Subtotal	***481.0***		***3,078***	***1,539***	***770***				
S-116G	281.0	6.4	1,798	899	450	125	452	0.18	0.65
Subtotal	***281.0***		***1,798***	***899***	***450***				
S-116H	493.0	6.4	3,155	1,578	789	219	750	0.32	1.08
Subtotal	***493.0***		***3,155***	***1,578***	***789***				
S-116I	10.0	20.0	200	100	50	14	58	0.02	0.08
Subtotal	***10.0***		***200***	***100***	***50***				
S-116J	30.0	20.0	600	300	150	42	164	0.06	0.24
Subtotal	***30.0***		***600***	***300***	***150***				
S-116K	12.0	20.0	240	120	60	17	69	0.02	0.10
Subtotal	***12.0***		***240***	***120***	***60***				
TOTAL	***3,316.0 (Acres)***		***22,262 (P.E.)***	***10,611 (P.E.)***	***5,306 (P.E.)***	***1,546 (GPM)***	***4,029 (GPM)***	***2.23***	***5.80***

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-117: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM Avg*	*GPM Peak*	*MGD Avg*	*MGD Peak*
S-117A	644.0	4.3	2,743	1,372	686	191	662	0.27	0.95
Subtotal	***644.0***		***2,743***	***1,372***	***686***				
A-117B	324.0	4.3	1,380	690	345	96	355	0.14	0.51
Subtotal	***324.0***		***1,380***	***690***	***345***				
S-117C	645.0	4.3	2,748	1,374	687	191	663	0.27	0.95
Subtotal	***645.0***		***2,748***	***1,374***	***687***				
S-117D	968.0	4.3	4,124	2,062	1,031	286	951	0.41	1.37
Subtotal	***968.0***		***4,124***	***2,062***	***1,031***				
S-117E	486.0	4.3	2,070	1,035	518	144	514	0.21	0.74
Subtotal	***486.0***		***2,070***	***1,035***	***518***				
S-117F	488.0	4.3	2,079	1,039	520	144	516	0.21	0.74
Subtotal	***488.0***		***2,079***	***1,039***	***520***				
S-117G	484.0	4.3	2,062	1,031	515	143	512	0.21	0.74
Subtotal	***484.0***		***2,062***	***1,031***	***515***				
S-117H	651.0	4.3	2,773	1,387	693	193	669	0.28	0.96
Subtotal	***651.0***		***2,773***	***1,387***	***693***				
TOTAL	***4,690.0 (Acres)***		***19,979 (P.E.)***	***9,990 (P.E.)***	***4,995 (P.E.)***	***1,387 (GPM)***	***3,681 (GPM)***	***2.00***	***5.30***

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-121: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM*		*MGD*	
						Avg	*Peak*	*Avg*	*Peak*
S-121A	637.0	6.7	4,236	2,118	1,059	294	974	0.42	1.40
Subtotal	***637.0***		***4,236***	***2,118***	***1,059***				
A-121B	242.0	6.7	1,609	805	402	112	409	0.16	0.59
Subtotal	***242.0***		***1,609***	***805***	***402***				
S-121C	322.0	6.7	2,141	1,071	535	149	530	0.21	0.76
Subtotal	***322.0***		***2,141***	***1,071***	***535***				
S-121D	648.0	6.7	4,309	2,155	1,077	299	989	0.43	1.42
Subtotal	***648.0***		***4,309***	***2,155***	***1,077***				
TOTAL	***1,849.0***		***12,296***	***6,148***	***3,074***	***854***	***2,446***	***1.23***	***3.52***
	(Acres)		***(P.E.)***	***(P.E.)***	***(P.E.)***	***(GPM)***	***(GPM)***		

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-125: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM*		*MGD*	
						Avg	*Peak*	*Avg*	*Peak*
S-125A	160.0	3.3	525	262	131	36	144	0.05	0.21
Subtotal	***160.0***		***525***	***262***	***131***				
A-125B	290.0	3.3	951	476	238	66	252	0.10	0.36
Subtotal	***290.0***		***951***	***476***	***238***				
S-125D	167.0	3.3	548	274	137	38	150	0.05	0.22
Subtotal	***167.0***		***548***	***274***	***137***				
TOTAL	***617.0 (Acres)***		***2,024 (P.E.)***	***1,012 (P.E.)***	***506 (P.E.)***	***141 (GPM)***	***503 (GPM)***	***0.20***	***0.72***

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-126: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM*		*MGD*	
						Avg	*Peak*	*Avg*	*Peak*
S-126A	289.0	5.7	1,642	821	410	114	416	0.16	0.60
Subtotal	***289.0***		***1,642***	***821***	***410***				
A-126B	332.0	5.7	1,886	943	471	131	472	0.19	0.68
Subtotal	***332.0***		***1,886***	***943***	***471***				
S-126C	454.0	5.7	2,579	1,289	645	179	626	0.26	0.90
Subtotal	***454.0***		***2,579***	***1,289***	***645***				
S-126D	455.0	5.7	2,584	1,292	646	179	628	0.26	0.90
Subtotal	***455.0***		***2,584***	***1,292***	***646***				
TOTAL	***1,530.0***		***8,690***	***4,345***	***2,173***	***604***	***1,820***	***0.87***	***2.62***
	(Acres)		***(P.E.)***	***(P.E.)***	***(P.E.)***	***(GPM)***	***(GPM)***		

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-130: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM*		*mgd*	
						Avg	*Peak*	*Avg*	*Peak*
S-130A	282.0	5.7	1,602	801	400	111	407	0.16	0.59
Subtotal	***282.0***		***1,602***	***801***	***400***				
TOTAL	***282.0***		***1,602***	***801***	***400***	***111***	***407***	***0.16***	***0.59***
	(Acres)		***(P.E.)***	***(P.E.)***	***(P.E.)***	***(GPM)***	***(GPM)***		

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-137: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM*		*MGD*	
						Avg	*Peak*	*Avg*	*Peak*
S-137A	953.0	7.4	7,033	3,517	1,758	488	1,516	0.70	2.18
Subtotal	***953.0***		***7,033***	***3,517***	***1,758***				
A-137B	609.0	7.4	4,494	2,247	1,124	312	1,026	0.45	1.48
Subtotal	***609.0***		***4,494***	***2,247***	***1,124***				
S-137C	646.0	7.4	4,767	2,384	1,192	331	1,081	0.48	1.56
Subtotal	***646.0***		***4,767***	***2,384***	***1,192***				
S-137D	160.0	7.4	1,181	590	295	82	308	0.12	0.44
Subtotal	***160.0***		***1,181***	***590***	***295***				
TOTAL	***2,368.0***		***17,476***	***8,738***	***4,369***	***1,214***	***3,291***	***1.75***	***4.74***
	(Acres)		***(P.E.)***	***(P.E.)***	***(P.E.)***	***(GPM)***	***(GPM)***		

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-139: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*		*GPM*		*mgd*	
							Avg	*Peak*	*Avg*	*Peak*
S-139A	551.0	6.4	3,499	1,749	875		243	822	0.35	1.18
Subtotal	***551.0***		***3,499***	***1,749***	***875***					
TOTAL	***551.0***		***3,499***	***1,749***	***875***		***243***	***822***	***0.35***	***1.18***
	(Acres)		***(P.E.)***	***(P.E.)***	***(P.E.)***		***(GPM)***	***(GPM)***		

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-141: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM*		*MGD*	
						Avg	*Peak*	*Avg*	*Peak*
S-141A	158.0	7.7	1,217	608	304	84	316	0.12	0.46
Subtotal	***158.0***		***1,217***	***608***	***304***				
S-141B	511.0	7.7	3,935	1,967	984	273	913	0.39	1.31
Subtotal	***511.0***		***3,935***	***1,967***	***984***				
S-141C	322.0	7.7	2,479	1,240	620	172	605	0.25	0.87
Subtotal	***322.0***		***2,479***	***1,240***	***620***				
S-141D	521.0	7.7	4,012	2,006	1,003	279	928	0.40	1.34
Subtotal	***521.0***		***4,012***	***2,006***	***1,003***				
S-141E	435.0	7.7	3,350	1,675	837	233	791	0.33	1.14
Subtotal	***435.0***		***3,350***	***1,675***	***837***				
S-141F	402.0	7.7	3,095	1,548	774	215	737	0.31	1.06
Subtotal	***402.0***		***3,095***	***1,548***	***774***				
S-141G	320.0	7.7	2,464	1,232	616	171	601	0.25	0.87
Subtotal	***320.0***		***2,464***	***1,232***	***616***				
S-141H	321.0	7.7	2,472	1,236	618	172	603	0.25	0.87
Subtotal	***321.0***		***2,472***	***1,236***	***618***				
S-141I	323.0	7.7	2,487	1,244	622	173	606	0.25	0.87
Subtotal	***323.0***		***2,487***	***1,244***	***622***				
S-141J	321.0	7.7	2,472	1,236	618	172	603	0.25	0.87
Subtotal	***321.0***		***2,472***	***1,236***	***618***				
S-141K	161.0	7.7	1,240	620	310	86	322	0.12	0.46
Subtotal	***161.0***		***1,240***	***620***	***310***				
S-141L	319.0	7.7	2,456	1,228	614	171	600	0.25	0.86
Subtotal	***319.0***		***2,456***	***1,228***	***614***				
S-141M	962.0	7.7	7,407	3,704	1,852	514	1,586	0.74	2.28
Subtotal	***962.0***		***7,407***	***3,704***	***1,852***				
TOTAL	***5,076.0 (Acres)***		***39,085 (P.E.)***	***19,543 (P.E.)***	***9,771 (P.E.)***	***2,714 (GPM)***	***6,421 (GPM)***	***3.91***	***9.25***

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-142: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM*		*MGD*	
						Avg	*Peak*	*Avg*	*Peak*
S-142C	161.0	5.5	886	443	221	61	236	0.09	0.34
Subtotal	***161.0***		***886***	***443***	***221***				
S-142D	159.0	5.5	875	437	219	61	233	0.09	0.34
Subtotal	***159.0***		***875***	***437***	***219***				
TOTAL	***320.0***		***1,760***	***880***	***440***	***122***	***443***	***0.18***	***0.64***
	(Acres)		***(P.E.)***	***(P.E.)***	***(P.E.)***	***(GPM)***	***(GPM)***		

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-156: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM*		*mgd*	
						Avg	*Peak*	*Avg*	*Peak*
S-156A	715.0	8.6	6,120	3,060	1,530	425	1,344	0.61	1.94
Subtotal	***715.0***		***6,120***	***3,060***	***1,530***				
TOTAL	***715.0***		***6,120***	***3,060***	***1,530***	***425***	***1,344***	***0.61***	***1.94***
	(Acres)		***(P.E.)***	***(P.E.)***	***(P.E.)***	***(GPM)***	***(GPM)***		

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin S-163: Tributary Wastewater Flow Calculations

Subbasin	*Area (acres)*	*P.E./acre*	*EWCCPS P.E.**	*P.E. * 50%*	*P.E. * 25%*	*GPM Avg*	*GPM Peak*	*mgd Avg*	*mgd Peak*
S-163A	276.0	7.8	2,139	1,070	535	149	529	0.21	0.76
Subtotal	***276.0***		***2,139***	***1,070***	***535***				
S-163B	283.0	7.8	2,193	1,097	548	152	541	0.22	0.78
Subtotal	***283.0***		***2,193***	***1,097***	***548***				
S-163C	202.0	7.8	1,566	783	391	109	399	0.16	0.57
Subtotal	***202.0***		***1,566***	***783***	***391***				
TOTAL	***761.0***		***5,898***	***2,949***	***1,474***	***410***	***1,302***	***0.59***	***1.87***
	(Acres)		***(P.E.)***	***(P.E.)***	***(P.E.)***	***(GPM)***	***(GPM)***		

EWCWPS P.E. denotes the total build-out P.E. per the Eastern Will County Wastewater Planning Study. This master plan study is based on 50% of the EWCWPS P.E.

Sub-Basin TC: Tributary Wastewater Flow Calculations

Subbasin	*Land Use*	*Area or Units (acres)*	*Future Acres or Units*	*P.E./(acre or unit)*	*P.E*	*GPM*		*MGD*	
						Avg	*Peak*	*Avg*	*Peak*
TC1	# SFR	25.0		3.5	88	6	26	0.01	0.04
	# TH	25.0		2.5	63	4	19	0.01	0.03
Subtotal					***150***				
TC2	# SFR	28.0		3.5	98	7	29	0.01	0.04
Subtotal					***98***				
TC3	# SFR	17.0		3.5	60	4	18	0.01	0.03
Subtotal					***60***				
TC4	# SFR	70.0		3.5	245	17	70	0.02	0.10
Subtotal					***245***				
TC5	# SFR	53.0		3.5	186	13	54	0.02	0.08
Subtotal					***186***				
TC6	# SFR	48.0		3.5	168	12	49	0.02	0.07
Subtotal					***168***				
TC7	# SFR		63	3.5	221	15	63	0.02	0.09
Subtotal					***221***				
TC8	# SFR	43.0		3.5	151	10	44	0.02	0.06
Subtotal					***151***				
TC9	# SFR	170.0		3.5	595	41	163	0.06	0.23
Subtotal					***595***				
TC10	# SFR	55.0		3.5	193	13	56	0.02	0.08
	C	40.0	10	10.0	500	35	138	0.05	0.20
Subtotal					***693***				
TC11	# SFR	195.0		3.5	683	47	185	0.07	0.27
Subtotal					***683***				
TC12	# SFR	58.0		3.5	203	14	58	0.02	0.08
	# TH	48.0		2.5	120	8	35	0.01	0.05
Subtotal					***323***				
TC13	# SFR	180.0		3.5	630	44	172	0.06	0.25
	# TH	39.0	7	2.5	115	8	34	0.01	0.05
	C	22.0		10.0	220	15	63	0.02	0.09
Subtotal					***965***				
TC14	C	30.0	17	10.0	470	33	130	0.05	0.19
Subtotal					***470***				
TC15	C	5.0		10.0	50	3	15	0.01	0.02
Subtotal					***50***				
TC16	# SFR	438.0		3.5	1,533	106	391	0.15	0.56
	# TH	106.0		2.5	265	18	75	0.03	0.11
	C	10.0		10.0	100	7	29	0.01	0.04
Subtotal					***1,898***				
TC17	# SFR	100.0		3.5	350	24	98	0.04	0.14
Subtotal					***350***				
TC18	# SFR		250	3.5	875	61	233	0.09	0.34
Subtotal					***875***				
TC19	R3		121	8.8	1,059	74	278	0.11	0.40
Subtotal					***1,059***				
TC20	# SFR	62.0		3.5	217	15	62	0.02	0.09
	# TH	18.0		2.5	45	3	14	0.00	0.02
	C	34.0		10.0	340	24	96	0.03	0.14
Subtotal					***602***				
TC21	R3		44	8.8	385	27	108	0.04	0.16
Subtotal					***385***				
TOTAL					***10,224 (P.E.)***	***710 (GPM)***	***2,091 (GPM)***	***1.02***	***3.01***

Sub-Basin H: Tributary Wastewater Flow Calculations

Subbasin	*Land Use*	*Area or Units (acres)*	*Future Acres or Units*	*P.E./(acre or unit)*	*P.E*		*GPM Avg*	*GPM Peak*
H1	# SFR	45.0		3.5	158		11	46
	C	6.0		10.0	60		4	18
Subtotal					***218***			
H2	# SFR	76.0		3.5	266		18	76
	C	7.0	7	10.0	140		10	41
Subtotal					***406***			
H3	# SFR	41.0		3.5	144		10	42
Subtotal					***144***			
H4	# SFR	91.0		3.5	319		22	90
Subtotal					***319***			
H5	# SFR	88.0		3.5	308		21	87
Subtotal					***308***			
H6	# SFR	43.0		3.5	151		10	44
Subtotal					***151***			
H7	C	15.0	10	10.0	250		17	71
	# SFR	20.0		3.5	70			
Subtotal					***320***			
H8	# SFR		250	3.5	875		61	233
							0	0
Subtotal					***875***			
H9	# SFR	107.0		3.5	375		26	105
Subtotal					***375***			
TOTAL		***0.0*** ***(Acres)***			***3,114*** ***(P.E.)***		***216*** ***(GPM)***	***741*** ***(GPM)***

VILLAGE OF BEECHER, ILLINOIS
SANITARY SEWER MASTER PLAN

APPENDIX B

Proposed Trunk Sewer Capacity Analysis

The tables included in this appendix summarize the trunk sewers sizes and pumping station capacities required to serve the entire Planning Area.

VILLAGE OF BEECHER, ILLINOIS
SANITARY SEWER MASTER PLAN

Trunk Sewer & Pump Station Analysis

TRUNK SEWER (TS)/ PUMP STATION (PS)	*Tributary TS / PS*	*Sub-basin / Trunk Section*	*Upstream Point*	*Downstream Point*	*Sub-Basin P.E.*	*Cumulative P.E.*	*Peak Flow (gpm)*	*Required Capacity (gpm)*	*Min sewer slope in section (%)*	*Required TS Size (inches)*
East Basins										
Pike Creek TS										
		(1/3)S141B, S141A	1	0	1,264	18,231	3,409		0.058	30
		S141D, S141C	2	1	3,246	16,967	3,210		0.058	30
		S141E	3	2	1,675	13,721	2,684		0.067	27
		S141F	5	3	1,548	12,047	2,404		0.080	24
		S141G	6	5	1,232	10,499	2,139		0.080	24
		S141H	7	6	1,236	9,267	1,923		0.080	24
		S141J, S141I	9	7	2,479	8,031	1,700		0.100	21
		S141K	10	9	620	5,552	1,235		0.120	18
		S141L	11	10	1,228	4,932	1,113		0.120	18
		S141M	15	11	3,704	3,704	865		0.150	15
County Line Rd. TS										
		(2/3)S141B	2	0	1,312	10,448	2,130		0.10	21
		S163A, B, C	6	2	2,949	9,137	1,899		0.10	21
		S130A, S126A, (1/2)S126B	9	6	2,093	6,188	1,357		0.12	18
		State Line PS			4,095	4,095				
County Line Road PS	Pike Creek TS, County Line TS, S139A				30,429	30,429	5,222			
Stateline Road PS		*S-126D, C, (1/2)B & S106A*			4,095	4,095	945			

VILLAGE OF BEECHER, ILLINOIS
SANITARY SEWER MASTER PLAN

Trunk Sewer & Pump Station Analysis

TRUNK SEWER (TS)/ PUMP STATION (PS)	Tributary TS / PS	Sub-basin / Trunk Section	Upstream Point	Downstream Point	Sub-Basin P.E.	Cumulative P.E.	Peak Flow (gpm)	Required Capacity (gpm)	Min sewer slope in section (%)	Required TS Size (inches)
Central Basins										
WWTP #2 Trunk Sewer										
		S137A, B	0	0	5,764	38,087	6,285		0.036	42
		S137C	1	0	2,384	32,323	5,489		0.046	36
		S137D, (1/2)S111A	3	1	994	29,940	5,152		0.046	36
	Trim Creek West TS		3	3	4,309	28,946	5,010		0.046	36
		S116A	5	3	1,539	24,637	4,383		0.046	36
		S116B	7	5	1,542	23,098	4,154		0.046	36
		S116C, D	8	7	2,746	21,556	3,922		0.046	36
		S116E	12	8	768	18,810	3,500		0.058	30
		S116G	13	12	899	18,042	3,380		0.058	30
		S116F	14	13	1,539	17,143	3,238		0.058	30
		S116H	15	14	1,578	15,604	2,992		0.058	30
	Trim Creek NW TS		15	15	4,671	14,026	2,735		0.058	30
	Trim Creek NE TS		15	15	9,355	9,355	1,938		0.08	24
Trim Creek NW TS		S91B	2	15	933	4,671	1,061		0.12	18
		S91C	3	2	960	3,738	872		0.15	15
		S91A, D	4	3	2,779	2,779	670		0.15	15
Trim Creek NE TS		(1/2)S106G	1	15	511	9,355	1,938		0.08	24
		S106K	2	1	1,575	8,844	1,847		0.08	24
		S106J, (1/2)S106G	3	2	1,286	7,268	1,560		0.08	24
		S106F, I	4	3	1,299	5,983	1,318		0.10	21
		(1/2)S106H, (1/2)S106E	5	4	1,042	4,684	1,064		0.12	18
		(1/2)S106H, (1/2)S106E	6	5	1,042	3,642	852		0.12	18
		S106D	7	6	1,556	2,601	631		0.12	18
		S106C	8	7	1,045	1,045	275		0.15	15
Trim Creek West TS		(1/2)S111A	1	3	403	4,309	989		0.15	15
		S111B	3	1	2,136	3,905	907		0.15	15
		(1/2)S111C	4	3	885	1,770	446		0.22	12
		(1/2)S111C	5	4	885	885	236		0.40	8
WWTP 1 Bypass Sewer	*WWTP 1*						5,915		0.046	36
WWTP No. 2	*Kentucky Rd. PS*				22,756	91,272	12,885			
	County Line Road PS				30,429	68,516				
	WWTP 2 TS				38,087	38,087				

VILLAGE OF BEECHER, ILLINOIS
SANITARY SEWER MASTER PLAN

Trunk Sewer & Pump Station Analysis

TRUNK SEWER (TS)/ PUMP STATION (PS)	Tributary TS / PS	Sub-basin / Trunk Section	Upstream Point	Downstream Point	Sub-Basin P.E.	Cumulative P.E.	Peak Flow (gpm)	Required Capacity (gpm)	Min sewer slope in section (%)	Required TS Size (inches)
West Basins										
Exline Slough North Trunk Sewer										
		S117A	1	0	1,372	13,548	2,656		0.067	27
		S117B	2	1	690	12,177	2,426		0.067	27
		S117C	4	2	1,374	11,486	2,309		0.067	27
		(2/3)S117D	6	4	1,374	10,113	2,072		0.08	24
		(1/3)S117D	7	6	687	8,738	1,828		0.08	24
	Indiana Ave. TS		8	8	1,031	8,051				
	Church Road West TS	S117E,F	9	8	2,824	7,020	1,514		0.10	21
		S117H	13	9	1,387	4,196	966		0.12	18
		(1/2)S100A, (1/2)S96A	16	13	1,298	2,810	676		0.15	15
	Plum Creek PS		16	16	1,511	1,511	386		0.28	10
Plum Creek PS		(1/2)S100A, (1/2)S96A, S81A			1,511	1,511	386			
Kentucky Road PS	*Exline Slough North TS*				13,548	22,756	4,103			
	Exline Slough East TS				6,148	9,208				
	Kentucky Road South TS				3,060	3,060				
Indiana Ave. West TS		S117G	3	8	1,031	1,031	271		0.40	8
Exline Slough East TS		S121B	1	0	805	6,148	1,349		0.12	18
		S121C	2	1	1,071	5,343	1,194		0.12	18
		(1/2)S121A	3	2	1,059	4,273	981		0.12	18
		(1/2)S121A	4	3	1,059	3,214	763		0.15	15
		(1/2)S121D	6	4	1,077	2,155	533		0.22	12
		(1/2)S121D	7	6	1,077	1,077	283		0.40	8
Exline Slough South TS		S156A	3	0	3,060	3,060	730		0.15	15
Church Road West TS		S117E	1	9	1,035	1,784	449		0.22	12
		S103B, S103A	2	1	240	749	202		0.40	8
		S103C	3	2	80	509	140		0.40	8
		S101B	4	3	146	429	119		0.40	8
		S103E, S101C	5	4	283	283	80		0.40	8

VILLAGE OF BEECHER, ILLINOIS
SANITARY SEWER MASTER PLAN

APPENDIX C

<u>Existing Trunk Sewer Capacity Analysis</u>

The tables included in this appendix summarize the P.E. loading tributary to each existing trunk sewer and also summarizes the available reserve capacity.

VILLAGE OF BEECHER, ILLINOIS
SANITARY SEWER MASTER PLAN

Trunk Sewer Capacity Analysis

TRUNK SEWER (TS)/ PUMP STATION (PS)	***Tributary TS / PS***	***Sub-basin / Trunk Section***	***Upstream Point***	***Downstream Point***	***Sub-Basin P.E.***	***Cumulative P.E.***	***Peak Flow (gpm)***	***Existing Sewer Length (ft)***	***Existing Sewer Size(in)***	***Existing Sewer Slope (%)***	***Existing Sewer Capacity (gpm)***	***Existing Reserve Capacity (gpm)***
Capacity Analysis:												
Cardinal Creek TS		H1	0	PS	218	5,853	1,293	20	21	0.50	5,028	3,735
		H2	1	0	406	5,635	1,251	1,260	18	0.40	2,981	1,730
		H3	2	1	144	5,229	1,172	1,260	18	0.40	2,981	1,809
		H4	3	2	319	5,086	1,144	1,430	12	0.50	1,131	-13
		H5	4	3	308	4,767	1,081	580	12	0.50	1,131	50
		H6, H8	5	4	1,026	4,459	1,019	1,785	12	0.50	1,131	112
		H7	6	5	320	3,434	809	950	12	0.40	1,011	202
		H9	7	6	3,114	3,114	741	550	12	0.40	1,011	270
Cardinal Creek PS*	*Cardinal Creek TS*					5,853	1,293				1240.00	-52.82
	**Cardinal Creek Pumping Station existing pumps have a capacity of 500 gpm and per the annexation agreement, the developer has to upsize these pumps to 1240 gpm pumps upon issuance of the 251st building permit for the subdivision.*											
Indiana Ave. TS												
			3	2		6,203	1,360		8*	1.50	664	-696
	Cardinal Creek TS	TC17	4	3		6,203	1,360	90	12	0.40	1,011	-349
	**per annexation agreement, the developer must upsize this sewer to a 12-inch at 1.5% upon issuance of the 251st building permit. A 12-inch sewer at 1.5% has capacity of 1,958 gpm*											
Church Road TS												
		TC20	1	14	602	2,600	631	*	12	0.22	750.00	119
		TC16, S116I	2	1	1,998	1,998	498	*	12	0.22	750.00	252
	**lengths and as-built slopes were not verified because 12" sewer at IEPA minimum allowable slope of 0.22% shows there is adequate acapacity in this sewer.*											
Dutch American TS		TC14	1	14	470	470	130		12	0.50	750.00	875
Fairway Drive TS					126	126	37		8	0.40	340.00	303
Fairway Drive PS	*Fairway Drive TS*		n/a	n/a	126	126	37	n/a	n/a	n/a	80.00	43

VILLAGE OF BEECHER, ILLINOIS
SANITARY SEWER MASTER PLAN

APPENDIX D

Proposed Trunk Sewer Cost Estimates

The tables included in this appendix summarize the estimated costs to construct the proposed sewers required to serve the Planning Area.

VILLAGE OF BEECHER, ILLINOIS
SANITARY SEWER MASTER PLAN

Opinion of Probable Cost with Lawn and Pavement Restoration (including engineering & contingency)

Trunk Sewer	Point	Ground Elevation	Invert Elevation	Pipe Slope min. (%)	Depth (feet)	Length (feet)	Ground Slope (%)	Avg. Depth (ft)	Size (in)	Unit Price ($/ft)	Cost ($)
East Basins											
Pike Creek TS											
	0	680.0	663.0	0.058	17.0	2,000	0.00	16.4	30	$312	$624,400
	1	680.0	664.2	0.058	15.8	2,200	0.00	15.4	30	$265	$582,120
	2	684.0	669.0	0.067	15.0	2,200	-0.01	16.5	27	$312	$686,840
	3	695.0	677.0	0.08	18.0	1,300	0.00	15.8	24	$218	$283,920
	4	695.0	681.5	0.08	13.5	1,200	-0.01	16.8	24	$263	$315,840
	5	710.0	690.0	0.08	20.0	1,000	0.00	19.6	24	$263	$263,200
	6	710.0	690.8	0.08	19.2	1,800	0.00	19.0	24	$263	$473,760
	7	711.0	692.2	0.10	18.8	2,000	0.00	18.4	21	$237	$473,200
	8	713.0	695.0	0.10	18.0	2,000	0.00	20.0	21	$237	$473,200
	9	720.0	698.0	0.12	22.0	1,300	0.00	21.2	18	$237	$307,580
	10	720.0	699.6	0.12	20.4	1,200	0.00	19.7	18	$218	$262,080
	11	720.0	701.0	0.15	19.0	2,000	0.00	17.5	15	$189	$378,000
	12	720.0	704.0	0.15	16.0	2,500	0.00	15.1	15	$144	$360,500
	13	722.0	707.8								$5,484,640
County Line Rd TS											
	0	680.0	663.0	0.10	17.0	2,000	-0.01	18.5	21	$237	$473,200
	1	700.0	680.0	0.10	20.0	2,000	-0.01	22.0	21	$237	$473,200
	2	710.0	686.0	0.10	24.0	2,000	0.00	23.0	21	$237	$473,200
	3	710.0	688.0	0.10	22.0	2,000	-0.01	21.0	21	$237	$473,200
	4	720.0	700.0	0.10	20.0	2,000	0.00	16.5	21	$192	$383,600
	5	715.0	702.0	0.10	13.0	2,000	0.00	13.5	21	$192	$383,600
	6	718.0	704.0	0.12	14.0	2,000	-0.01	18.8	18	$218	$436,800
	7	730.0	706.4	0.12	23.6	2,000	0.01	17.4	18	$218	$436,800
	8	720.0	708.8	0.12	11.2	2,700	0.00	14.6	18	$175	$472,500
	9	730.0	712.0								$4,006,100

VILLAGE OF BEECHER, ILLINOIS
SANITARY SEWER MASTER PLAN

Opinion of Probable Cost with Lawn and Pavement Restoration (including engineering & contingency)

Trunk Sewer	Point	Ground Elevation	Invert Elevation	Pipe Slope min. (%)	Depth (feet)	Length (feet)	Ground Slope (%)	Avg. Depth (ft)	Size (in)	Unit Price ($/ft)	Cost ($)
Central Basins											
WWTP No. 2 TS											
	0	682.0	665.0	0.036	17.0	2,000	0.00	16.6	42	$332	$664,400
	1	682.0	665.7	0.046	16.3	1,000	0.00	17.6	36	$322	$322,200
	2	685.0	666.2	0.046	18.8	2,700	0.00	19.4	36	$322	$869,940
	3	690.0	670.0	0.046	20.0	1,500	0.00	19.7	36	$322	$483,300
	4	690.0	670.7	0.046	19.3	1,000	0.00	19.1	36	$322	$322,200
	5	690.0	671.2	0.046	18.8	2,000	0.00	19.4	36	$322	$644,400
	6	692.0	672.1	0.046	19.9	1,500	0.00	19.6	36	$322	$483,300
	7	692.0	672.8	0.046	19.2	2,400	0.00	18.7	36	$322	$773,280
	8	692.0	673.9	0.058	18.1	1,800	0.00	17.6	30	$312	$561,960
	9	692.0	674.9	0.058	17.1	1,200	0.00	16.7	30	$312	$374,640
	10	692.0	675.6	0.058	16.4	2,500	0.00	16.2	30	$312	$780,500
	11	693.0	677.1	0.058	15.9	1,300	0.00	15.6	30	$265	$343,980
	12	693.0	677.8	0.058	15.2	2,000	0.00	15.1	30	$265	$529,200
	13	694.0	679.0	0.058	15.0	1,500	0.00	15.1	30	$265	$396,900
	14	695.0	679.8	0.058	15.2	2,500	0.00	14.4	30	$265	$661,500
	15	695.0	681.3								$8,211,700
Trim Creek NW TS	15	695.0	681.3	0.12	13.7	1,400	-0.01	16.4	18	$175	$245,000
	1	707.0	688.0	0.12	19.0	1,600	0.00	18.5	18	$218	$349,440
	2	708.0	689.9	0.15	18.1	2,500	0.00	17.2	15	$189	$472,500
	3	710.0	693.7	0.15	16.3	1,600	0.00	17.6	15	$189	$302,400
	4	715.0	696.1								$1,369,340
Trim Creek NE TS	15	695.0	681.3	0.08	13.7	1,700	0.00	15.5	24	$218	$371,280
	1	700.0	682.6	0.08	17.4	1,700	0.00	19.2	24	$263	$447,440
	2	705.0	684.0	0.08	21.0	2,500	-0.01	25.0	24	$313	$783,000
	3	722.0	693.0	0.10	29.0	2,500	0.00	27.8	21	$287	$716,500
	4	722.0	695.5	0.12	26.5	2,500	0.00	25.0	18	$268	$671,000
	5	722.0	698.5	0.12	23.5	1,900	0.00	18.9	18	$218	$414,960
	6	715.0	700.8	0.12	14.2	3,000	0.00	12.4	18	$175	$525,000
	7	715.0	704.4	0.15	10.6	2,200	0.00	11.5	15	$111	$243,320
	8	720.0	707.7								$4,172,500
Trim Creek West TS	3	695.0	678.0	0.15	17.0	1,900	-0.01	18.0	15	$189	$359,100
	1	708.0	689.0	0.15	19.0	3,000	0.00	16.8	15	$144	$432,600
	2	708.0	693.5	0.15	14.5	1,500	-0.01	14.8	15	$144	$216,300
	3	718.0	703.0	0.22	15.0	2,300	-0.01	15.0	12	$133	$305,900
	4	740.0	725.0	0.40	15.0	2,000	-0.01	15.0	8	$118	$235,200
	5	750.0	735.0	0.40	15.0						$1,549,100

VILLAGE OF BEECHER, ILLINOIS
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Opinion of Probable Cost with Lawn and Pavement Restoration (including engineering & contingency)

Trunk Sewer	Point	Ground Elevation	Invert Elevation	Pipe Slope min. (%)	Depth (feet)	Length (feet)	Ground Slope (%)	Avg. Depth (ft)	Size (in)	Unit Price ($/ft)	Cost ($)
West Basins											
Exline Slough North TS											
	0	690.0	676.6	0.067	13.4	2,800	0.00	15.0	27	$230	$644,000
	1	695.0	678.5	0.067	16.5	2,000	0.00	15.9	27	$230	$460,000
	2	695.0	679.8	0.067	15.2	2,000	0.00	14.5	27	$230	$460,000
	3	695.0	681.2	0.067	13.8	2,800	0.00	12.9	27	$230	$644,000
	4	695.0	683.0	0.08	12.0	1,000	0.00	11.6	24	$181	$180,600
	5	695.0	683.8	0.08	11.2	1,000	0.00	10.7	24	$181	$180,600
	6	695.0	684.8	0.08	10.2	2,000	0.00	12.6	24	$181	$361,200
	7	702.0	687.0	0.08	15.0	1,000	0.00	14.6	24	$218	$218,400
	8	702.0	687.8	0.10	14.2	2,000	-0.01	14.6	21	$192	$383,600
	9	712.0	697.0	0.12	15.0	2,000	0.00	15.5	18	$175	$350,000
	10	716.0	700.0	0.12	16.0	1,800	-0.01	17.5	18	$218	$393,120
	11	725.0	706.0	0.12	19.0	1,200	0.00	20.0	18	$218	$262,080
	12	730.0	709.0	0.12	21.0	1,100	0.00	21.3	18	$218	$240,240
	13	732.0	710.3	0.15	21.7	2,900	0.00	16.0	15	$144	$418,180
	14	725.0	714.7	0.15	10.3	1,800	0.00	10.5	15	$111	$199,080
	15	728.0	717.4	0.15	10.6	2,000	0.00	10.1	15	$111	$221,200
	16	730.0	720.4								$5,616,300
Indiana Ave. West TS	8	702.0	690.0	0.40	12.0	2,000	-0.01	12.0	8	$84	$168,000
	1	712.0	700.0	0.40	12.0	1,500	-0.01	12.0	8	$84	$126,000
	2	720.0	708.0	0.40	12.0	2,000	0.00	8.0	8	$84	$168,000
	3	720.0	716.0								$462,000
Exline Slough East TS	0	690.0	676.6	0.12	13.4	2,000	-0.01	17.2	18	$175	$350,000
	1	700.0	679.0	0.12	21.0	1,000	0.00	20.4	18	$218	$218,400
	2	700.0	680.2	0.12	19.8	3,300	0.00	17.8	18	$218	$720,720
	3	700.0	684.2	0.15	15.8	3,000	0.00	13.6	15	$144	$432,600
	4	700.0	688.7	0.22	11.3	1,000	0.00	10.2	12	$101	$100,800
	5	700.0	690.9	0.22	9.1	2,000	-0.01	10.1	12	$101	$201,600
	6	715.0	704.0	0.40	11.0	2,000	-0.01	12.0	8	$84	$168,000
	7	725.0	712.0								$2,192,120
Kentucky Road South TS											
	0	690.0	676.6	0.15	13.4	1,800	0.00	12.1	15	$111	$199,080
	1	690.0	679.3	0.15	10.7	1,000	0.00	9.9	15	$111	$110,600
	2	690.0	680.8	0.15	9.2	1,000	0.00	8.4	15	$111	$110,600
	3	690.0	682.3								$420,280

VILLAGE OF BEECHER, ILLINOIS
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Opinion of Probable Cost with Lawn and Pavement Restoration (including engineering & contingency)

Trunk Sewer	Point	Ground Elevation	Invert Elevation	Pipe Slope min. (%)	Depth (feet)	Length (feet)	Ground Slope (%)	Avg. Depth (ft)	Size (in)	Unit Price ($/ft)	Cost ($)
Church Road West TS											
	9	712.0	697.0	0.22	15.0	3,600	-0.01	20.0	12	$175	$630,000
	1	730.0	704.9	0.22	25.1	3,300	0.00	17.5	12	$175	$577,500
	2	722.0	712.2	0.28	9.8	2,000	0.00	11.0	10	$91	$182,000
	3	730.0	717.8	0.28	12.2	2,200	0.00	14.1	10	$125	$274,120
	4	740.0	723.9	0.28	16.1	2,500	0.00	13.6	10	$125	$311,500
	5	742.0	730.9		11.1						$1,975,120

VILLAGE OF BEECHER, ILLINOIS
SANITARY SEWER MASTER PLAN

APPENDIX E

<u>Church Road and Trim Creek Trunk Sewers Capacity Analysis</u>

The table included in this appendix summarizes the P.E. loading tributary to the Church Road Trunk Sewer and the Trim Creek Trunk Sewers. This appendix also lists the reserve capacity available in each trunk sewer.

VILLAGE OF BEECHER, ILLINOIS
SANITARY SEWER MASTER PLAN

Trim Creek Trunk Sewer Capacity Analysis

							EXISTING FLOW CALCULATIONS									***FUTURE FLOW CALCULATIONS***						
	From Manhole	***To Manhole***	***Size (in)***	***Length (ft)***	***Slope (%)***	***Full Capacity***	***P.E.***	***Ex. Area added***	***Average DWF***	***Peak DWF***	***10-yr I/I Tributary***	***Total Peak Flow***	***Reserve Capacity***	***Reserve Capacity (with E.F Pumps & Trim Creek Relief Sewer)***	***Fut. P.E.***	***Fut. Area added***	***Average DWF***	***Peak DWF***	***Total Peak Flow***	***Reserve Capacity***	***Reserve Capacity (with E.F Pumps & Trim Creek Relief Sewer)***
Excess Flow Pump South to WWTP	3	1	18	12	1.000	4,710	8032	TC1	558	1,339	4,254	5,593	-883	1,617	1934		134	322	5,915	-1,205	1,295
	4	3	18	200	0.240	2,310	7882	TC2	547	1,314	4,254	5,568	-3,258	-758	1934		134	322	5,890	-3,580	-1,080
	17	4	18	175	0.300	2,580	7784	TC3	541	1,297	4,254	5,551	-2,971	-471	1934		134	322	5,874	-3,294	-794
	18	17	18	230	0.240	2,310	**7725**		**536**	**1,287**	**4,123**	5,410	-3,100	-600	1934		134	322	5,733	-3,423	-923
	19	18	18	236	0.220	2,300	7725	TC4,5	536	1,287	4,123	5,410	-3,110	-610	1934		134	322	5,733	-3,433	-933
	36	19	18	356	0.228	2,390	7294	TC6	507	1,216	3,623	4,839	-2,449	51	1934		134	322	5,161	-2,771	-271
	49	36	18	140	0.243	2,320	7126	TC7	495	1,188	3,448	4,636	-2,316	184	1934		134	322	4,958	-2,638	-138
	50	49	18	190	0.220	2,300	6906	TC8	480	1,151	3,198	4,349	-2,049	451	1934		134	322	4,671	-2,371	129
	51	50	15	400	0.500	2,050	6755		469	1,126	2,998	4,124	-2,074	426	1934		134	322	4,446	-2,396	104
	68	51	15	455	0.500	2,050	**6755**	TC9, 10	**469**	**1,126**	**2,998**	4,124	-2,074	426	1934		134	322	4,446	-2,396	104
Catalpa to Excess Flow PS	129	68	15	700	0.300	1,590	5468		380	911	1,893	2,804	-1,214	2,412	1,934		134	483	3,287	-1,697	1,929
	130	129	15	35	0.810	2,610	**5415**		**376**	903	**1,856**	2,759	-149	3,547	1,934		134	483	3,242	-632	3,063
	131	130	15	264	0.810	2,610	5415	(1/5) TC11, TC12	376	903	1,856	2,759	-149	3,547	1,934		134	483	3,242	-632	3,063
	154	131	15	350	0.360	1,740	4956	(1/5) TC11	344	826	1,429	2,255	-515	2,234	1,934		134	483	2,738	-998	1,751
	165	154	15	174	0.150	1,120	4819	(1/5) TC11	335	803	1,336	2,139	-1,019	1,730	1,934		134	483	2,622	-1,502	1,247
	166	165	15	340	0.150	1,120	4683	(1/5) TC11	325	780	1,206	1,986	-866	1,883	1,934		134	483	2,469	-1,349	1,400
	167	166	15	136	0.150	1,120	4546	TC15,13, (2/5)11	316	758	1,076	1,834	-714		1,934		134	483	1,559	-439	
Trim Creek Industrial Trunk Sewer	167A	167	15	25	0.150	1,120	3258		226	543	0	543	577		1,934		134	483	1,026	94	
	167B	167A	15	131	0.150	1,120	3258		226	543	0	543	577		1,934		134	483	1,026	94	
	167C	167B	15	400	0.150	1,120	3258		226	543	0	543	577		1,934		134	483	1,026	94	
	167D	167C	15	376	0.400	1,830	3258		226	543	0	543	1,287		1,934		134	483	1,026	804	
	167E	167D	15	400	0.400	1,830	3258		226	543	0	543	1,287		1,934	1/2 TC18	134	483	1,026	804	
	167F	167E	15	400	0.400	1,830	3258		226	543	0	543	1,287		1,496		104	382	925	905	
	167G	167F	15	341	0.400	1,830	3258		226	543	0	543	1,287		1,496		104	382	925	905	
	168A	167G	15	281	1.000	2,900	3258		226	543	0	543	2,357		1,496	1/2 TC18	104	382	925	1,975	
	168B	168A	15	25	0.400	1,830	3258		226	543	0	543	1,287		1,059	TC19	74	278	821	1,009	
Church Road	199A	168B	12	350	1.340	1,850	3258		226	543	0	543	1,307		0		0	0	543	1,307	
	199B	199A	12	350	1.340	1,850	2885	TC16	200	481	0	481	1,369		0		0	0	481	1,369	
	199C	199B	12	340	0.220	750	2,885	TC20	200	481	0	481	269		0		0	0	481	269	
	200A	199C	12	2250	0.220	750	2,283	TC16, TC21	159	381	0	381	370		0		0	0	381	370	
	168H	168B	12	1300	0.500	875	373	TC14	26	62	0	62	813		0		0	0	62	813	

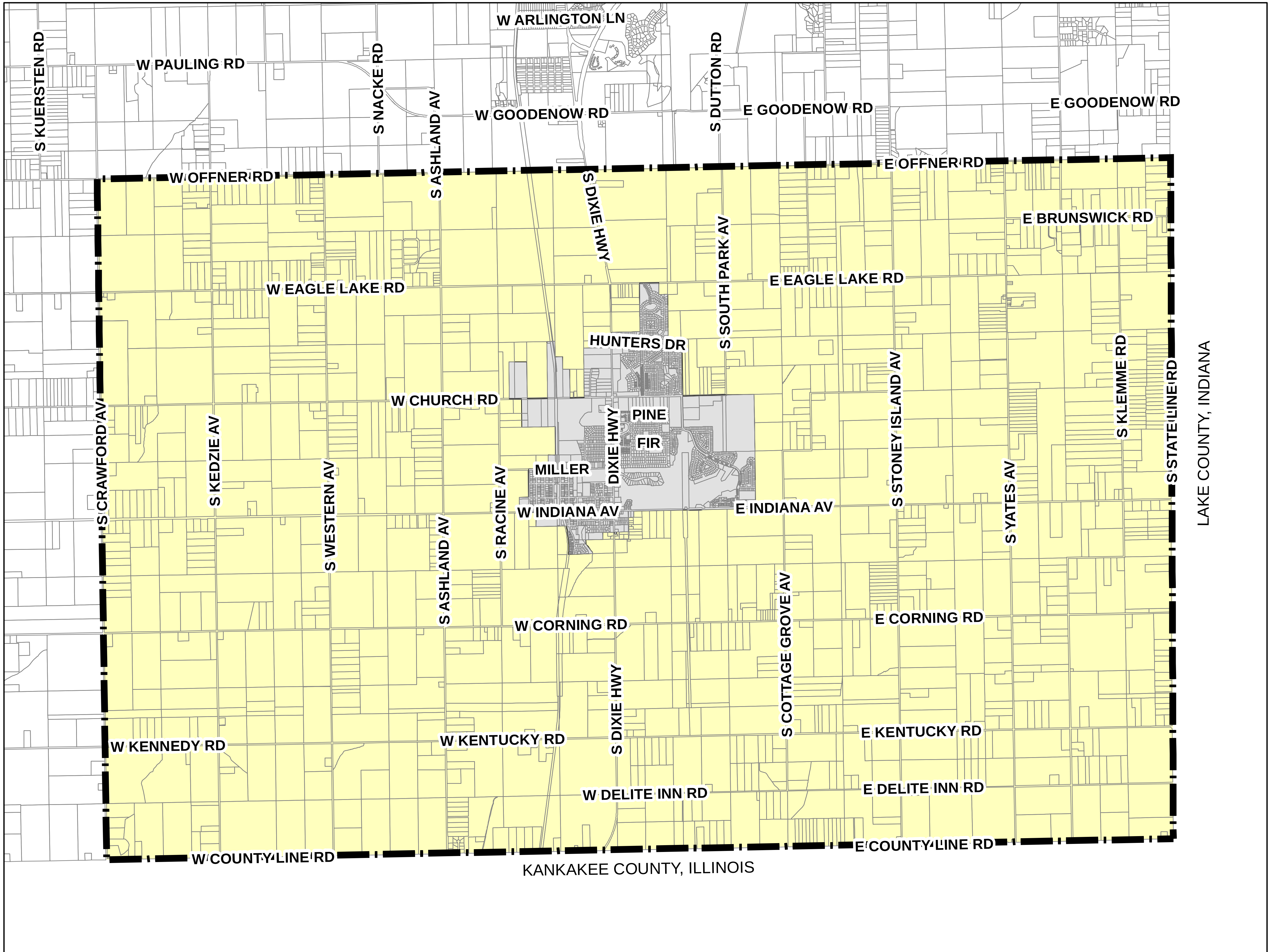
*** ALL FLOW CALCULATIONS ARE IN GPM*

Notes:

EXHIBIT A

VILLAGE OF BEECHER, ILLINOIS

PLANNING AREA MAP



Legend

EXISTING SERVICE AREA

PLANNING BOUNDARY

N

1 inch = 4,500 feet

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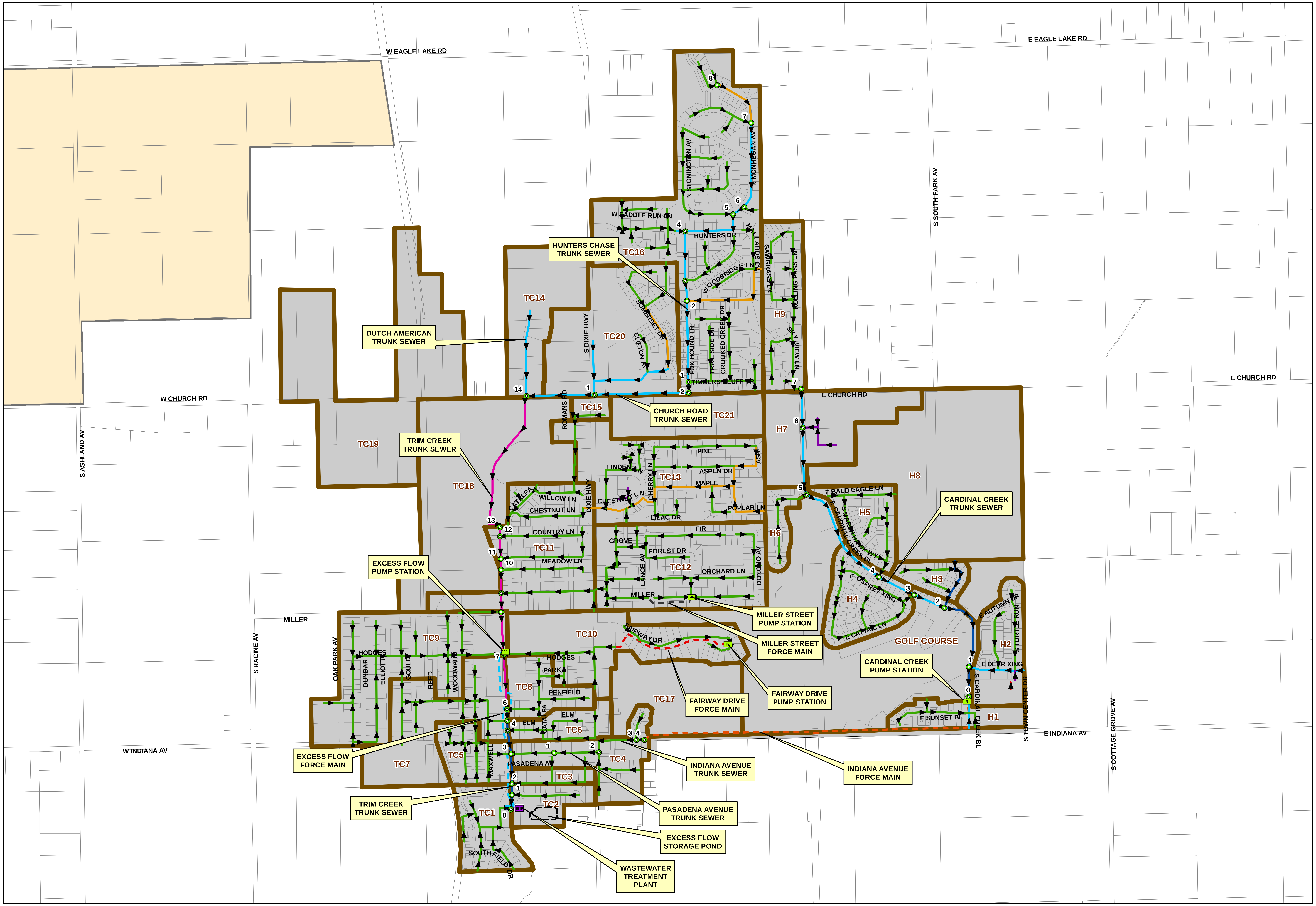
I:\Mokena\BEECHER\GENERAL\gis\MapDocuments\121011-water & sewer master plan\Exhibit A.mxd
364cak - 12/12/2013



Legend

4" SEWER	12" SEWER	3" FM	PUMPING STATION
6" SEWER	15" SEWER	4" FM	WWTP
8" SEWER	18" SEWER	10" FM	EXISTING VILLAGE LIMITS
10" SEWER	21" SEWER	12" FM	EXISTING SUBBASINS
			AIRPORT - INAUGURAL

VILLAGE OF BEECHER, ILLINOIS

EXISTING SUBBASIN BOUNDARY AND SANITARY SEWER SYSTEM MAP

N

0 400 800 1,200 1,600

Feet

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Consulting Engineers

Note: All sewers, Pumping Stations, WWTP, and server area shown are existing as of August 2013

EXHIBIT B

I:\Mokena\BEECH\GENERAL\gis\Map Documents\121011-water & sewer master plan\Exhibit B.mxd
364ctk - 12/23/2013

VILLAGE OF BEECHER, ILLINOIS

SUB-BASIN BOUNDARY MAP AND TRUNK SEWER MATER PLAN

N

0 1,000 2,000 3,000 4,000

Feet

Legend

- PLANNING BOUNDARY
- MAJOR DRAINAGE AREAS
- PLANNING SUB-BASINS
- AIRPORT - INAUGURAL
- PROPOSED ILLIANA EXPRESSWAY CORRIDOR
- PROPOSED PUMP STATIONS
- PROPOSED SEWERS
- PROPOSED FORCE MAIN
- EXISTING PUMPING STATION
- WWTP
- EXISTING SANITARY SEWER
- EXISTING FORCE MAIN

ILLIANA EXPRESSWAY CORRIDOR SOURCE: illianacorridor.org Tier 2 Alternatives Map

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Consulting Engineers

EXHIBIT C

N

0 1,000 2,000 3,000 4,000
Feet

VILLAGE OF BEECHER, ILLINOIS

SUB-BASIN BOUNDARY AND SANITARY SEWER LOADING MAP

Legend

- PLANNING BOUNDARY
- MAJOR DRAINAGE AREAS
- PLANNING SUB-BASINS
- EXISTING SUB-BASINS
- AIRPORT - INAUGURAL
- PROPOSED ILLIANA EXPRESSWAY CORRIDOR
- EXISTING VILLAGE LIMTS

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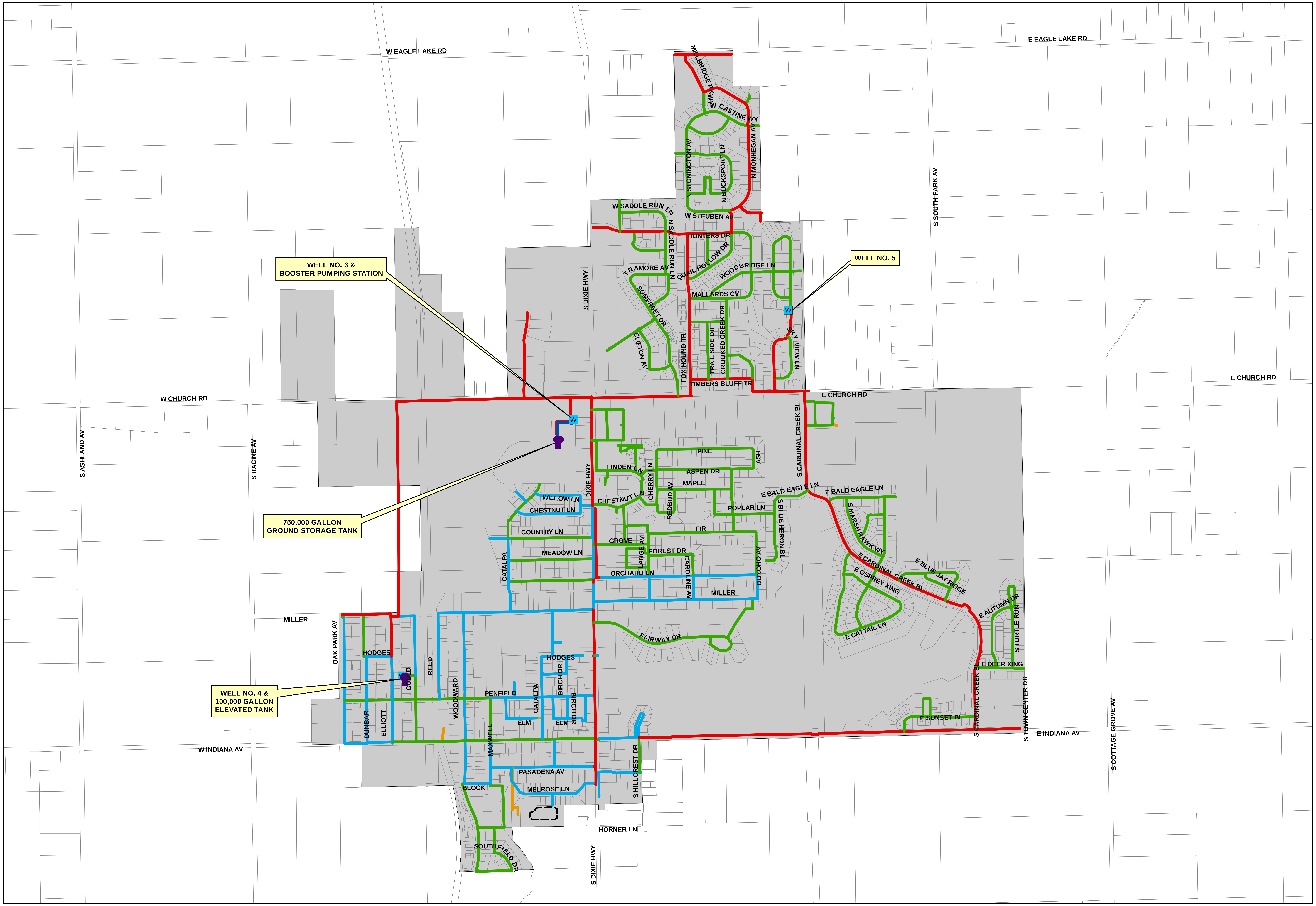
Consulting Engineers

ILLIANA EXPRESSWAY CORRIDOR SOURCE: illianacorridor.org Tier 2 Alternatives Map

EXHIBIT D



VILLAGE OF BEECHER, ILLINOIS

EXISTING SUBBASIN BOUNDARY AND WATER SYSTEM MAP

N

0 400 800 1,200 1,600

Feet

Legend

- 4" WATER
- 6" WATER
- 8" WATER
- 10" WATER
- 16" WATER
- TANKS
- Wells
- EXISTING VILLAGE LIMITS

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Note: All facilities shown are existing as of August 2013

EXHIBIT E

N

0 1,000 2,000 3,000 4,000

Feet

VILLAGE OF BEECHER, ILLINOIS

SUB-BASIN BOUNDARY MAP AND WATER SYSTEM CONCEPT

* Location and number of proposed wells and proposed tanks are shown for reference only.
Actual number and location will depend on development and well capacities.

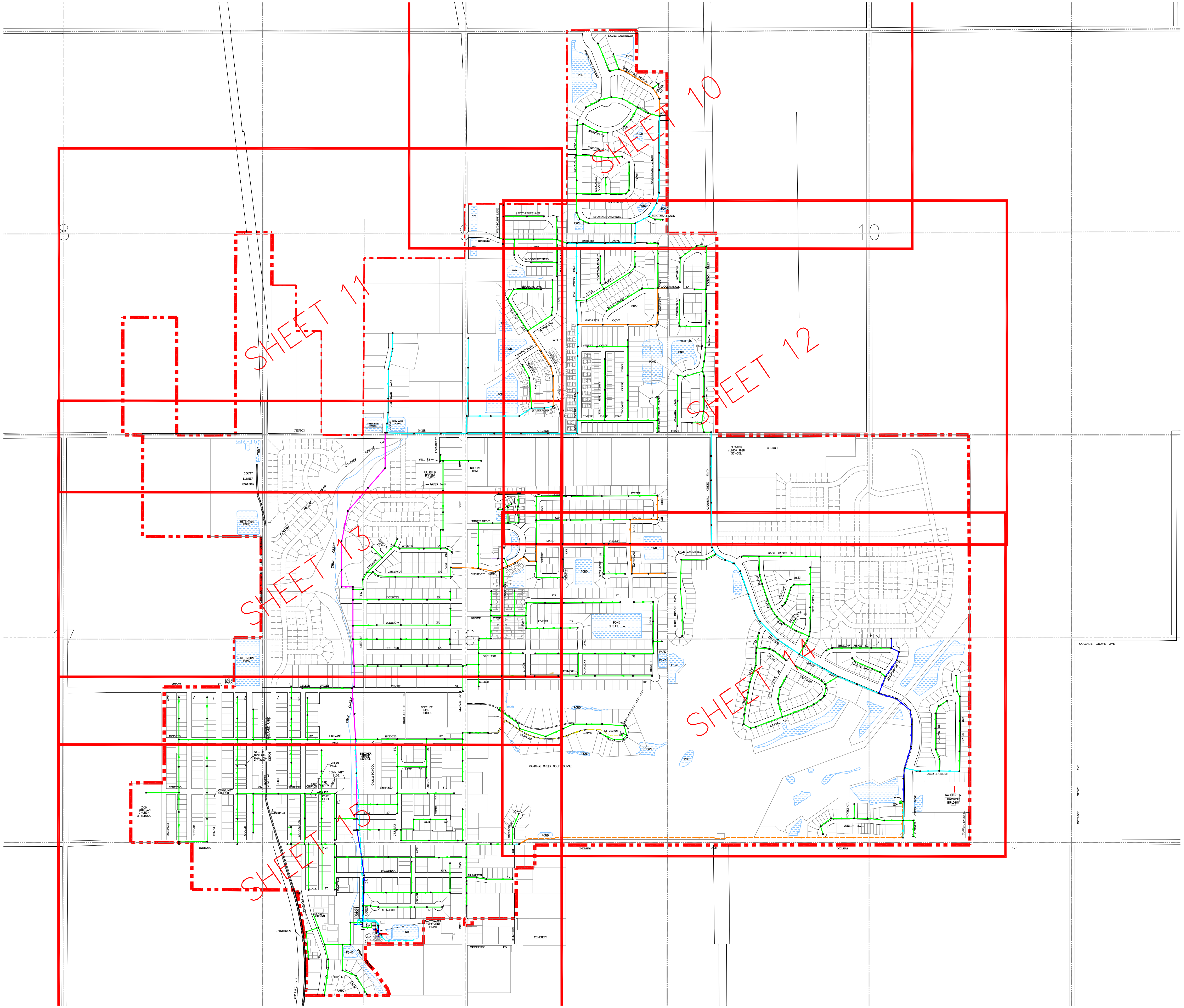
ILLIANA EXPRESSWAY CORRIDOR SOURCE: illianacorridor.org Tier 2 Alternatives Map

Legend

- PLANNING BOUNDARY
- AIRPORT - INAUGURAL
- PROPOSED ILLIANA EXPRESSWAY CORRIDOR
- PROPOSED BOOSTER PUMP STATION
- PROPOSED TANKS
- PROPOSED WELLS
- PROPOSED WATERMAINS
- PROPOSED UPPER PRESSURE ZONE
- EXISTING TANKS
- EXISTING WELLS
- EXISTING WATERMAINS

BAXTER & WOODMAN Consulting Engineers

EXHIBIT F



VILLAGE OF BEECHER, ILLINOIS

SANITARY SEWER ATLAS LEGEND

- SEWER SERVICE AND DISTANCE
- SEWER DIRECTION
- SEWER MANHOLE AND NUMBER
- SEWER STUB
- 3" SANITARY SEWER FORCE MAIN
- 4" SANITARY SEWER FORCE MAIN
- 4" SANITARY SEWER
- 6" SANITARY SEWER
- 8" SANITARY SEWER
- 10" SANITARY SEWER
- 12" SANITARY SEWER
- 12" SANITARY SEWER FORCE MAIN
- 15" SANITARY SEWER
- 18" SANITARY SEWER
- 21 SANITARY SEWER
- VILLAGE BOUNDARY

EXHIBIT G

N

0 300 600 900 1200

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Consulting Engineers

REVISED: MARCH 2014

SHEET 9 OF 22 – SANITARY SEWER

I:\MOKENA\BEECH\GENERAL\DWG\SANITARY\DWGS\SANMAP.DWG SAN-OVERALL-SHT9-36X24
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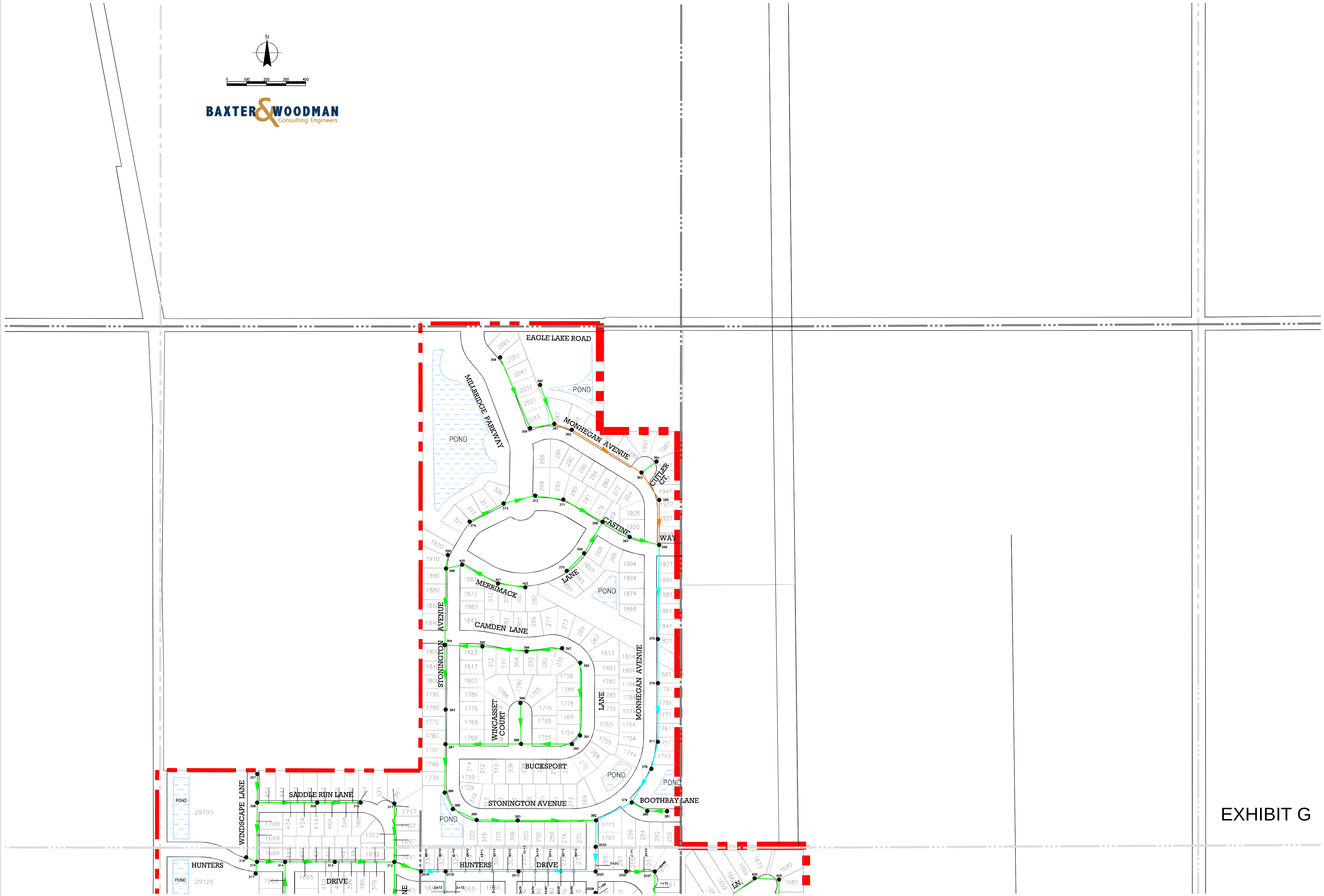
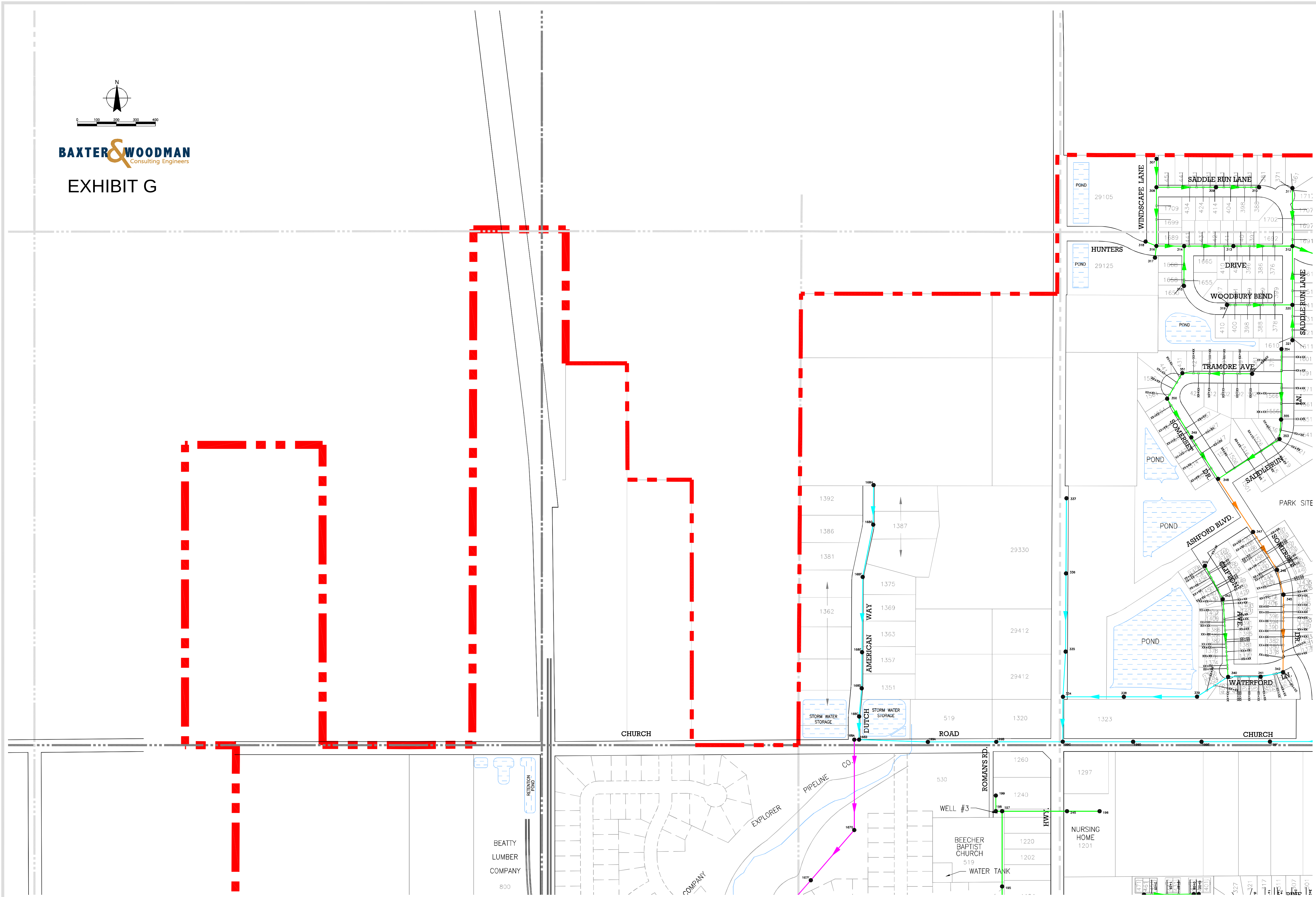

N
0 100 200 300 400
BAXTER & WOODMAN
Consulting Engineers
EAGLE LAKE ROAD
MILLBRIDGE PARKWAY
MONHEGAN AVENUE
CUTLER CT.
CASTINE WAY
MERRIMACK LANE
CAMDEN LANE
STONINGTON AVENUE
WINGCASSET COURT
BUCKSPORT LANE
BOOTHBAY LANE
SADDLE RUN LANE
WINDSCAPE LANE
HUNTERS DRIVE
POND

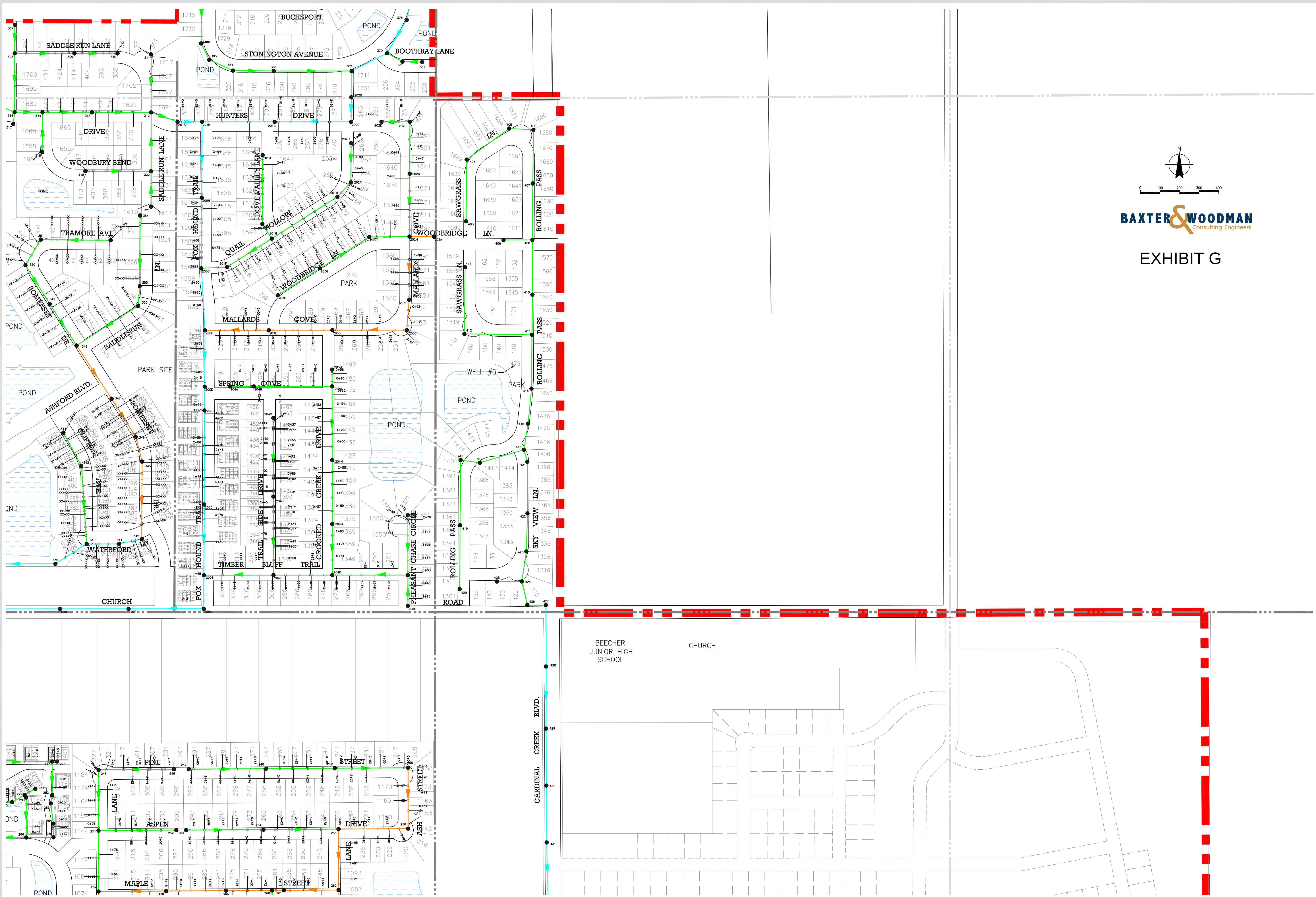
EXHIBIT G

EXHIBIT G



N
0 100 200 300 400
BAXTER & WOODMAN
Consulting Engineers
SADDLE RUN LANE
WINDSCAPE LANE
HUNTERS
DRIVE
WOODBURY BEND
SADDLE RUN LANE
TRAMORE AVE
SOMERSET DR.
SADDLE RUN
POND
PARK SITE
ASHFORD BLVD.
CLIFTON
WATERFORD
29105
29125
29330
29412
29412
1392
1386
1381
1362
1387
1375
1369
1363
1357
1351
519
1320
1323
STORM WATER STORAGE
STORM WATER STORAGE
DUTCH
AMERICAN
WAY
CHURCH
ROAD
CHURCH
ROMAN'S RD.
HWY
PIPELINE
CO.
EXPLORER
COMPANY
WELL #3
BEECHER BAPTIST CHURCH
519
WATER TANK
NURSING HOME
1201
1260
1240
1220
1202
1297
530
BEATTY
LUMBER
COMPANY
800
RETENTION POND
SHEET 11 OF 22 – SANITARY SEWER
I:\MOKENA\BEECH\GENERAL\DWG\SANITARY\DWGS\SANMAP.DWG SAN–SHT11
Plotted: 3/14/2014 7:50 AM By: 364CAK

EXHIBIT G



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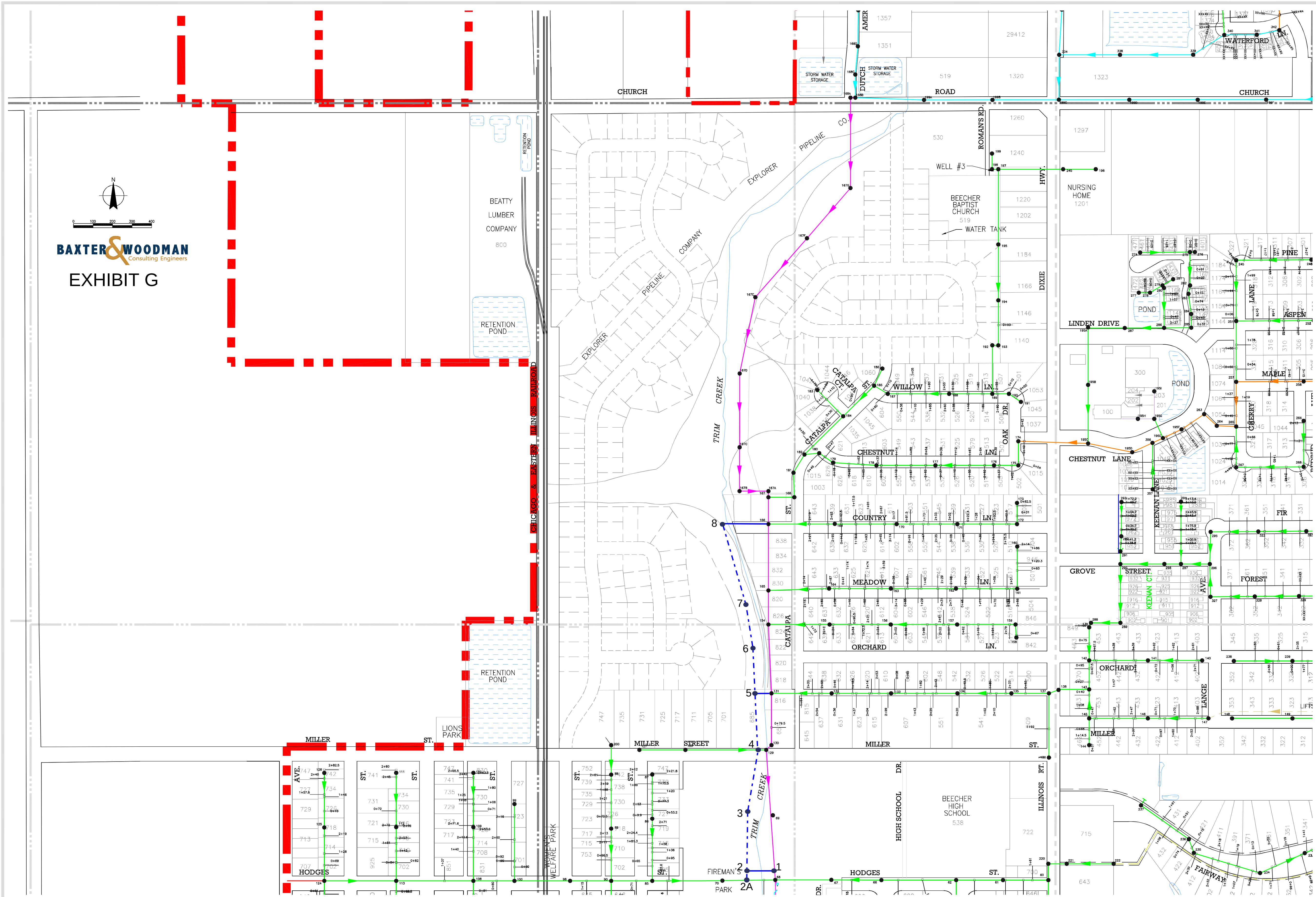
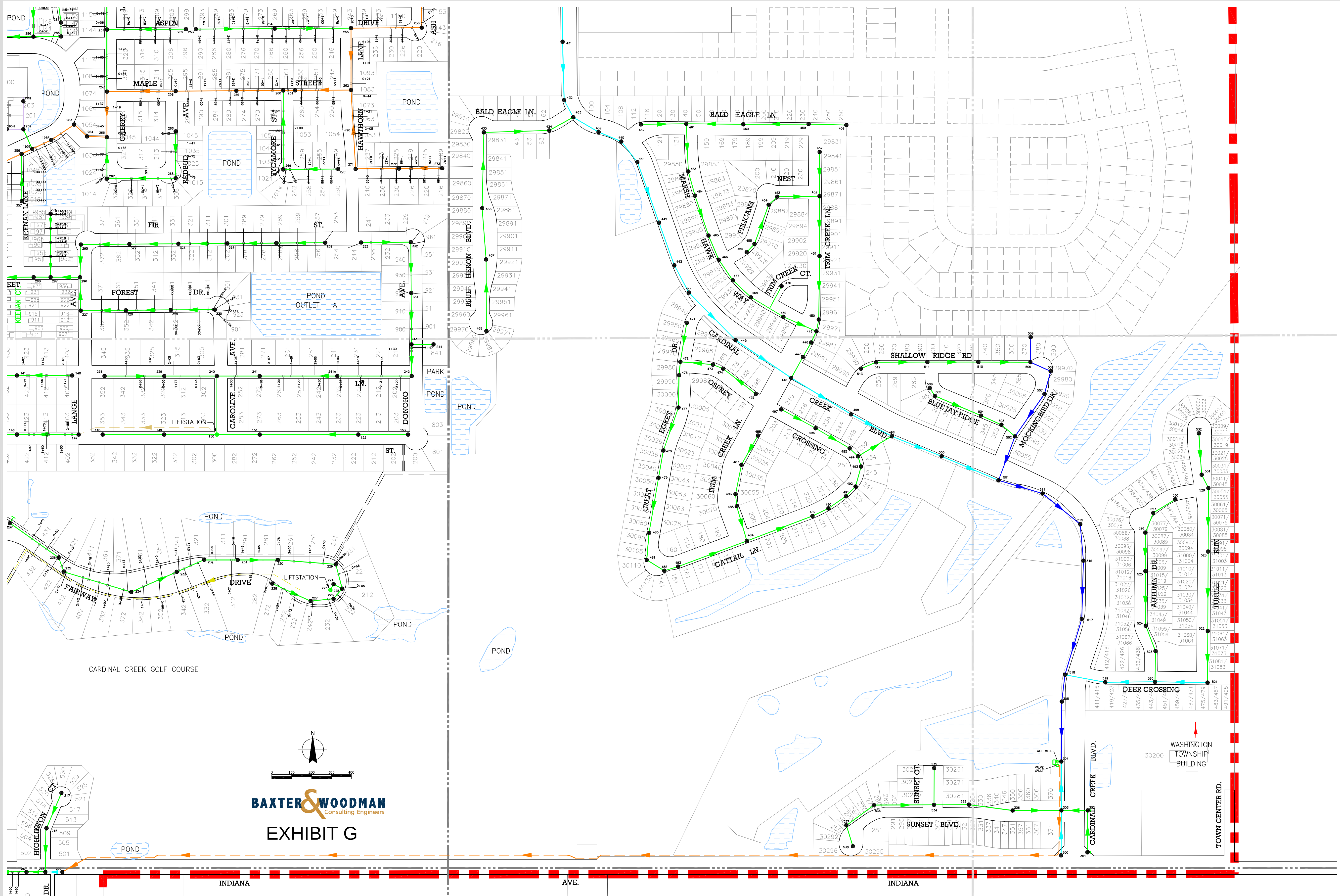


BAXTER & WOODMAN
Consulting Engineers
EXHIBIT G
N
CHURCH
ROAD
CHURCH
BEATTY
LUMBER
COMPANY
RETENTION POND
RETENTION POND
LIONS PARK
MILLER
ST.
MILLER
STREET
MILLER
ST.
HODGES
HODGES
ST.
CHICAGO & EASTERN ILLINOIS RAILROAD
EXPLORER
PIPELINE
COMPANY
EXPLORER
PIPELINE
CO.
TRIM
CREEK
TRIM
CREEK
STORM WATER STORAGE
STORM WATER STORAGE
DUTCH
AMER
ROMANS RD
WELL #3
BEECHER
BAPTIST
CHURCH
WATER TANK
DIXIE
HWY.
NURSING
HOME
WATERFORD
LN.
PINE
LANE
ASPEN
MAPLE
CHERRY
LINDEN DRIVE
POND
POND
CHESTNUT LANE
CATALPA
CT.
CATALPA
ST.
WILLOW
LN.
CHESTNUT
OAK
DR.
LN.
COUNTRY
LN.
MEADOW
LN.
ORCHARD
LN.
CATALPA
ST.
KEENAN
GROVE
STREET
AVE.
FOREST
FIR
ORCHARD
LANGE
MILLER
FAIRWAY
HIGH SCHOOL
DR.
BEECHER
HIGH
SCHOOL
ILLINOIS
RT.
FIREMAN'S
PARK
WOMEN'S
WELFARE
PARK
AVE.
8
7
6
5
4
3
2
2A
1
SHEET 13 OF 22 – SANITARY SEWER



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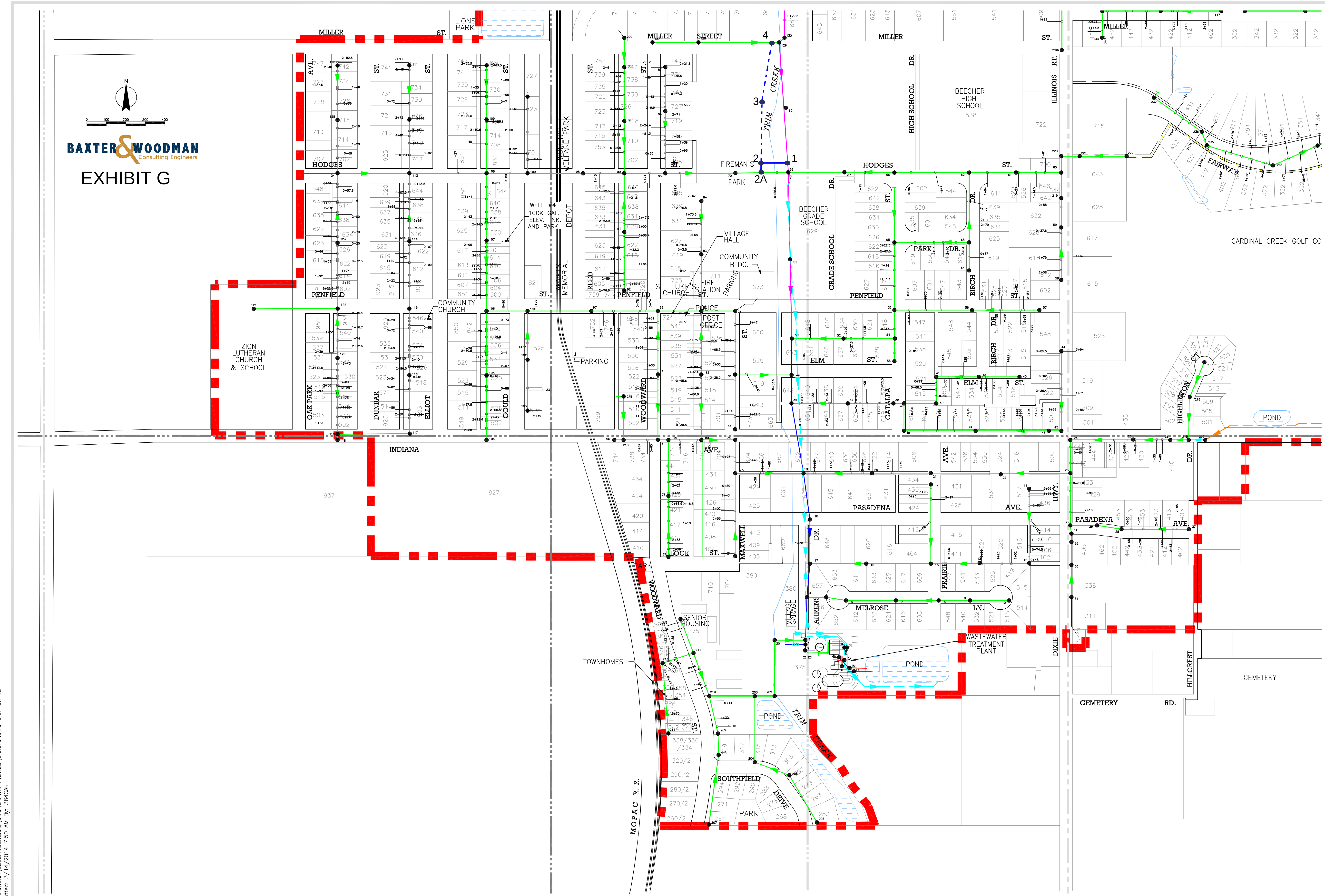

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