Report for

Waunakee Utilities, Village of Waunakee, Wisconsin

Sanitary Sewer Comprehensive Plan

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EXECUTIVE SUMMARY

This report updates the Sanitary Sewer Comprehensive Plan completed in 2013. The goal of the Sanitary Sewer Comprehensive Plan is to review the existing system to identify problem areas in the system and evaluate the potential for future development outside of the currently developed Village of Waunakee (Village) limits.

The Village continues to grow through development of residential, commercial, and industrial areas. Residential growth is the primary driver for expansion of the sanitary sewer and water supply systems. The Water System Study (2018 Study) has been updated in parallel with this Sanitary Sewer Comprehensive Plan. Both reports should be referenced as the service area expands.

A. Changes Since 2013 Sanitary Sewer Comprehensive Plan (2013 Plan)

As a backdrop for discussing current and future expansion of the sanitary sewer system, the following summarizes the areas of growth and infrastructure improvements since 2013.

1. Residential Development

The Westbridge development (NW-10 as shown in Figure 3.04-1), located west of the Meadows of Sixmile Creek golf course and south of Kopp Road, is a 135-acre residential development with 283 lots. The area is served with water and sewer service along the Kopp Road corridor. A redundant water main connection was also installed from the southern edge of the development to an existing main along Highway 19. Wastewater flows to the Westbridge Pumping Station at Kopp Road and discharges to the Northwest interceptor sewer.

The Northridge subdivision was partially complete at the time of the 2013 Plan. In the past five years, the northeast portion of the development was completed. The first phases of the development were in the NE-2 subbasin. The last phase is in the NE-3 subbasin. Water service is provided by an extension of mains from the primary pressure zone and an extension of the boosted zone in the adjacent Waunakee Heights (SM-2) plat.

The Kilkenny Farms development is located west of the Southbridge neighborhood and east of CTH Q. The area is primarily residential with a small commercial area as discussed below. There are 383 residential lots platted with homebuilding ongoing. The development is in the SS-3 subbasin which flows to the Blue Ridge Pumping Station. A 15-inch interceptor sewer was installed through the development that will allow service to the west across CTH Q.

The Kilkenny Farms–West neighborhood is located at the southwest corner of CTH Q and Woodland Drive. The currently proposed development includes residential, mixed use, and commercial areas. Residential areas are expected to include 210 single family dwellings and 400 apartment units. Approximately 16.8 acres will be commercial area. The 2013 Plan showed drainage subbasins SS-6 and SS-7 in this area. Those subbasins were redrawn based on the layout and drainage shown in the urban service area amendment application. The revised subbasin boundaries can be seen in Figure 3.04-1.

The Woodland Crest development is located at the southeast corner of CTH Q and Woodland Drive. The proposed development includes 30 acres of a mix of commercial and residential development. Commercial development may include a grocery and convenience store, and residential development has been considered to be mostly multifamily, although no plan has yet been approved for the residential portion.

Arboretum Village is a 113-lot residential development located north of Arboretum Drive and west of Hogan Road. The area is included in the R-4 subbasin which drains to the Ravine interceptor sewer.

2 Commercial Development

Kilkenny Farms Commons is a 43-acre commercial/retail/mixed use development located in the northwest corner of the Kilkenny residential neighborhood in the CTH Q corridor. Proposed businesses here include medical and dental offices, retail shops, dining establishments, and child and elder care facilities. Development in this area is ongoing.

3. Redevelopment Downtown (SM-1 area)

In 2013, redevelopment at the northeast corner of Madison Street and Main Street produced a 50-unit apartment building with commercial space on the ground floor. It is called Madison/Main development.

In 2015, commercial redevelopment of the former Koltes Lumber property occurred, producing several commercial and restaurant spaces. This site will be referred to as the Lone Girl site, being the current anchor tenant.

Under construction in 2018 is the redevelopment of the north 200 block of East Main Street, which will consist of 105 apartment units and two restaurant/commercial spaces. It is called Lamphouse.

4. Industrial Development

Frank H. Street located in the Waunakee Industrial Park was extended by approximately 550 feet in 2017. The project extended water and sewer service to allow approximately 20 acres of industrial development in the IP-3 subbasin. Construction of a multi-unit small business building is underway in 2018. Further development in this subbasin is likely when the demand materializes.

5. Point Source Contributions

Appendix B was updated from the 2013 Plan to show several new locations that contribute wastewater in quantities significant enough to be broken out as separate point sources. They include the BrightStar Senior Living on Quinn Drive, Home Again Assisted Living in Kilkenny Farms, Waunakee Intermediate School on Woodland Drive, and Octopi Brewing on Uniek Drive in the industrial park.

Estimated wastewater flows from each of the new developments and point sources were used to update the flow and sewer capacity projections from the 2013 Plan.

6. Infrastructure Improvements

Ongoing development serves to extend interceptor sewers in those areas of the Village seeing growth. Examples include sewer extensions through the Westbridge and Kilkenny developments.

The Village also continues to make improvements within the existing service area. One such improvement was the new segment of 10-inch sanitary sewer along Centennial Drive in 2015. This connection allows greater capacity through the lower portion of the Endres subbasin.

The Village has also replaced clay sewers on multiple streets since 2013 as part of its annual public works improvement projects.

B. Methodology

The ultimate service area was broken down into subbasins tributary to their downstream interceptor extension, and flows for these areas were calculated based on existing development and future land use assumptions. These future flows were added in logical sequence to the existing system until critical capacity within the downstream system was reached. The boundary of this area is referred to as the Available Capacity Service Limit and is shown in Figure 4.07-1. Note that this area represents the approximate limits of service with minimal additional downstream improvements to the existing system.

Finally, existing and future flows were combined to determine the appropriate pipe sizes and limits for each existing interceptor and future extension. This information is shown in Figure 4.08-1.

C. Summary of Conclusions and Recommendations

Based on this analysis, the following conclusions and recommendations were made:

1. Conclusions

a. Flow metering indicates the system may experience significant I/I during peak events.

(1) Recommendation

(a) Additional flow metering should be completed to identify problem areas throughout the community and develop a plan for addressing the sources of infiltration and inflow (I/I).

b. The existing Northwest Interceptor Sewer from Kopp Road south through Fairbrook Drive presents critical capacity issues that limit additional development in the northern region of the ultimate development area.

(1) Recommendations

(a) Monitor flows annually to assess capacity.

(b) Upgrade existing sewers to accommodate ultimate service area flows when necessary.

(c) Construct a relief sewer along Century Avenue between the Northwest and Sixmile Interceptor Sewers to alleviate short-term capacity concerns.

c. The Division Street Interceptor Sewer is currently near critical capacity and limits future development north of STH 19.

(1) Recommendations

(a) Complete additional evaluation of existing industrial park flows to verify capacity concerns.

(b) Upgrade existing sewers to accommodate ultimate service area flows.

d. The Southside Interceptor Sewer and Blue Ridge Pumping Station will likely require upgrades for future development west of CTH Q.

(1) Recommendation

(a) Improvements to the existing pumping station and force mains should be evaluated as future development occurs. Gravity mains should be upgraded to accommodate ultimate service area flows.

e. Current service area accommodates approximately 3,000 acres (16,000 equivalent residents). The available capacity service area accommodates approximately 6,200 acres (38,400 residents) with improvements to the Southside Interceptor. The ultimate service area includes approximately 11,000 acres (68,000 residents), but will require improvements to much of the existing system to accommodate those residents.

(1) Recommendations

(a) 40 percent of the undeveloped lands tributary to sewers with available capacity are in the Eastern Region.

(b) Future capital improvement projects should consider upgrades to accommodate ultimate service area flows.

f. The Village has consistently experienced reasonable growth through new development.

(1) Recommendation

(a) Review and update the Sanitary Sewer Comprehensive Plan every five years.

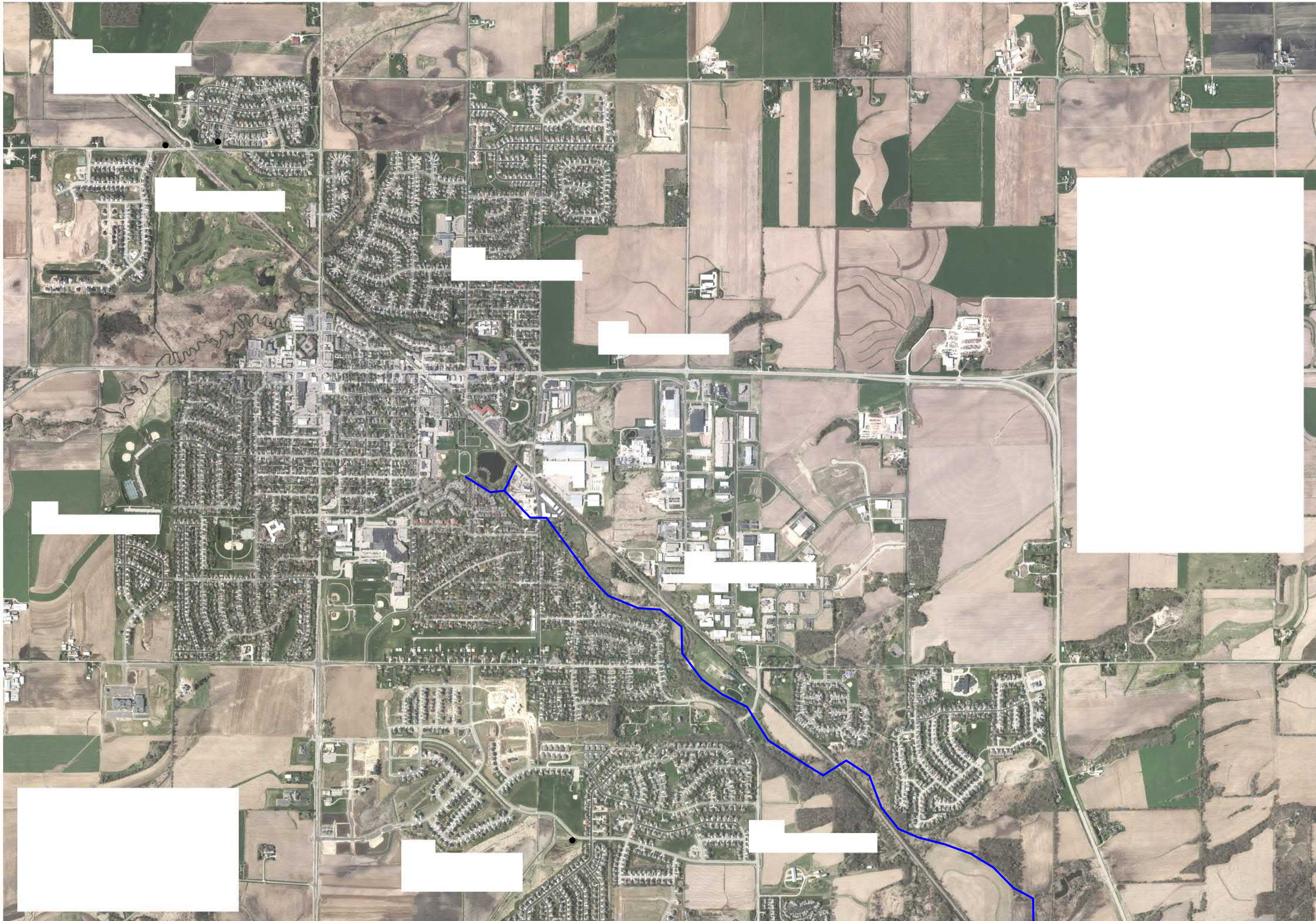
g. The Village desires to replace clay sewers as warranted, or with road reconstruction projects, to maintain flow, reduce backups, and limit infiltration.

(1) Recommendation

(a) Develop a program for televising and documenting sewers and laterals, starting in the oldest neighborhoods, to develop an inventory of replacements to be made.

(b) Televise all sewers in low lying areas, those paralleling drainage swales, and in areas of known high ground water, looking for sources of inflow and infiltration. Take corrective actions to eliminate inflow, and consider options to limit or eliminate infiltration.

h. Monitor key sewer interceptors and pumping stations for capacity as development tributary to those sewers occurs. See Figure ES-1 for a listing.



SECTION 1
INTRODUCTION

1.01 PURPOSE

The purpose of this report is to provide an update to the 2013 Sanitary Sewer Comprehensive Plan (2013 Plan). The 2013 Plan reviewed the Village of Waunakee's (Village) sanitary sewer or collection system capacity and developed a comprehensive plan to establish proper and logical growth of its sanitary sewer utility infrastructure. This plan update will allow continued system improvements to be implemented economically as areas develop and sewage flows increase.

1.02 SCOPE

The study area includes those portions of the Village currently supplied with municipal sanitary sewer as well as future areas that will require sanitary sewer service.

The scope of the report update includes the following elements:

1. Provide an executive summary of the findings of the 2018 Sanitary Sewer Comprehensive Plan Update (2018 Plan Update).
2. Provide an introduction section to identify objectives of the 2018 Sanitary Sewer Comprehensive Plan Update.
3. Provide a narrative summarizing the Village's existing sewage collection system and revise the figures and tables to reflect current infrastructure, land use, and estimated flows.
4. Provide a narrative summarizing the Village's ultimate service area boundary and update the associated figures and tables.
5. Provide a narrative summarizing conclusions and alternatives for maintaining, extending, and improving the Village's sewer system, including identifying potential deficiencies in the system in the next five to ten years.
6. Update figures and tables in the appendices to reflect current development.

1.03 DEFINITIONS

DU	dwelling unit
DU/ac	dwelling units per acre
FUDA	Future Urban Development Area
gcd	gallons per capita per day
GIS	Geographic Information System
gpd	gallons per day
gpd/ac	gallons per day per acre
gpm	gallons per minute
hp	horsepower
I/I	infiltration and inflow

MMSD	Madison Metropolitan Sewerage District
SPL	Scientific Protein Labs
Village	Village of Waunakee

SECTION 2
SANITARY SEWER FLOW MONITORING PROGRAM

This section is unchanged from the 2013 Study and is included here for reference.

2.01 FLOW MONITORING LOCATIONS

Five temporary flow meters were installed to develop an understanding of flow rates at key points within the Village. Locations were identified and reviewed with Village personnel. Following the site selections, the proposed locations were visited, and a determination was made regarding the suitability of each site for the collection of flow data. Two of the meters were placed on the Madison Metropolitan Sewerage District (MMSD) interceptor.

Figure 2.01-1 shows the existing collection system and the location of the flow monitors installed in the system. Table 2.01-1 presents a description of each site and lists the manhole where each meter is located.

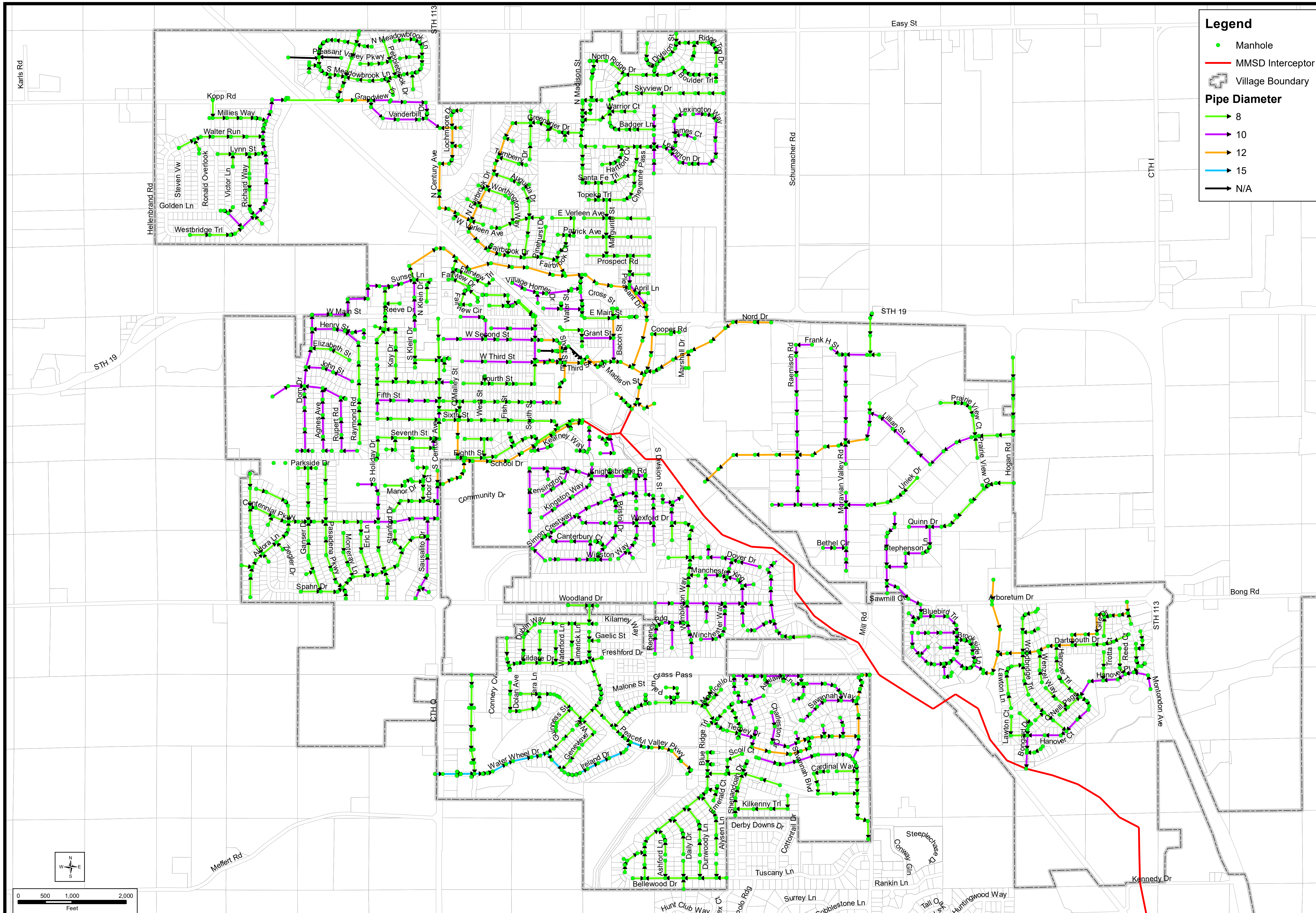
Meter Identification	Sanitary Sewer Size/Location	Comments
Century-12	12-inch sanitary sewer/located east of South Century Avenue in the driveway of Endres Manufacturing.	Meters flow from the area southwest of the intersection of South Century Avenue and 8th Street.
Division-24 (MH14-357 on MMSD Interceptor)	24-inch sanitary sewer/located east of the intersection of South Division Street and Knightsbridge Road adjacent to Sixmile Creek.	Meters flow on the main interceptor for approximately the northern half of the study area.
Fairbrook-12	12-inch sanitary sewer/located at the intersection of North Fairbrook Drive, Edgemere Court, and Sawgrass Court.	Meters flow from the northeast portion of the study area.
Kennedy-30 (MH14-326 on MMSD Interceptor)	30-inch sanitary sewer/located east of the intersection of Kennedy Drive and railroad tracks.	Meters flow on the main interceptor from the entire study area.
Muirfield-12	12-inch sanitary sewer/located at the intersection of West Verleen Avenue and Muirfield Court.	Meters flow from the northwest portion of the study area.

Table 2.01-1 Flow Monitor Locations

2.02 FLOW METER CALIBRATION

The flow meters were installed in the collection system on April 2, 2012. For each location, a manhole entry was made, and the equipment was placed into operation and calibrated. Calibration consisted of taking a manual level reading in the sanitary sewer and comparing it to the level reading of the flow monitor. Software provided with the flow meters allowed the user to enter the correct level reading, thereby calibrating the unit.

Later that day, each flow metering site was visited and another manhole entry was made. Levels were measured and compared to monitor readings. If necessary, the calibration of each monitor was adjusted. Typically, flow meters will require an adjustment to the calibration after the initial installation because the internal electronics of the flow meters require this period to adjust to



EXISTING SEWER COLLECTION SYSTEM
SANITARY SEWER COMPREHENSIVE PLAN

VILLAGE OF WAUNAKEE
DANE COUNTY, WISCONSIN

STRAND ASSOCIATES®

FIGURE 2.01-1
1602.125

Path: S:\MAD\1600--1699\1602\125\Drawings\GIS\Sanitary System 11x17.mxd User: danc Date: 9/25/2018 Time: 11:14:08 AM *in-situ* temperature and humidity conditions. Usually, after this first adjustment, the meters will stay calibrated. No additional manhole entries were made during the study to confirm proper calibration.

2.03 EQUIPMENT MAINTENANCE AND DATA COLLECTION

After the initial installation and subsequent calibration check, the flow meters were visited on a weekly basis by Village staff. Data was collected on each unit, and a visual check of data quality was made to confirm the meters were operating correctly. Debris, such as gravel, silt, and rags, was removed from the manhole on several occasions. However, even after cleaning the debris from the manholes, there were multiple periods where data could not be used from the meters because of the continued accumulation of debris.

The flow meters were removed on June 8, 2012.

2.04 DRY WEATHER FLOW MONITORING DATA SUMMARY

Dry weather flows were developed by plotting flows and rainfall and identifying the driest days during the metering period. The flows from these days, after removing the Village of Dane flows, were averaged to develop an average dry weather day. The period of May 1 through May 13, 2012, was identified as the driest period and was used to develop an average dry weather day. The flow monitor located near Kennedy Drive was the focus of developing average dry weather flows because it measures nearly all the flow from the Village. Table 2.04-1 presents a summary of the dry weather flow data for this site.

Site Name	Pipe Diameter (in)	Average Flow (gpm)	Maximum Flow (gpm)
Kennedy-30	30	992	1,101

Table 2.04-1 Kennedy-30 Dry Weather Flow Summary

2.05 INFILTRATION DETERMINATION

Infiltration to the collection system is characterized by high base flows following a rain event. Infiltration is most often caused by elevated groundwater levels that allow water to enter leaking pipe joints and other defects in the sanitary sewers and manholes. Infiltration is often difficult to cost-effectively remove from a collection system.

To determine existing infiltration, the average dry weather flow was plotted against the average daily flow, disregarding days of rain, from the Kennedy-30 flow monitor and is shown in Figure 2.05-1.

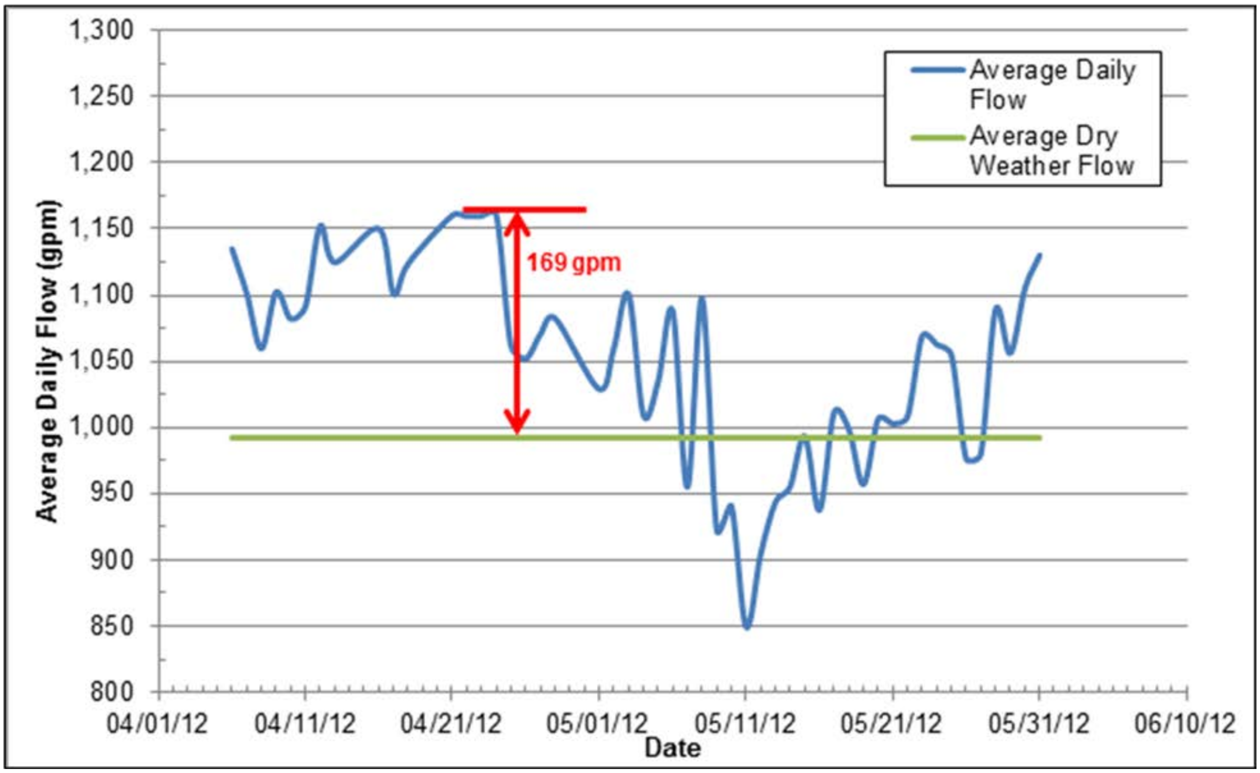


Figure 2.05-1 Kennedy-30 Infiltration Calculation

It is assumed that the difference between the average dry weather flow and the peak average nonrainfall daily flow is the infiltration to the collection system. The peak average daily flow occurred on April 20 and resulted in a difference of 169 gpm, or approximately 243,000 gallons per day (gpd). Applying this infiltration value over the entire Kennedy-30 service area or 2,313 acres results in an infiltration rate of approximately 105 gallons per day per acre (gpd/ac).

2.06 INFLOW DETERMINATION

Inflow to the collection system is characterized by a rapid rise in flow rate during and immediately following a rain event. Inflow sources are direct connections between the sanitary sewer system and surface water associated with the rainfall. Inflow sources may include downspouts and foundation drains connected to the sanitary sewer along with sump pumps and cross-connections to storm sewer.

To determine existing inflow, 15-minute flows that occurred during the rain event on Thursday, April 19, 2012, were plotted against the average 15-minute flows of the Thursday before and the three Thursdays after the rain event from the Kennedy-30 flow monitor and are shown in Figure 2.06-1. The same day of the week was chosen to maintain similar shapes of the diurnal patterns.

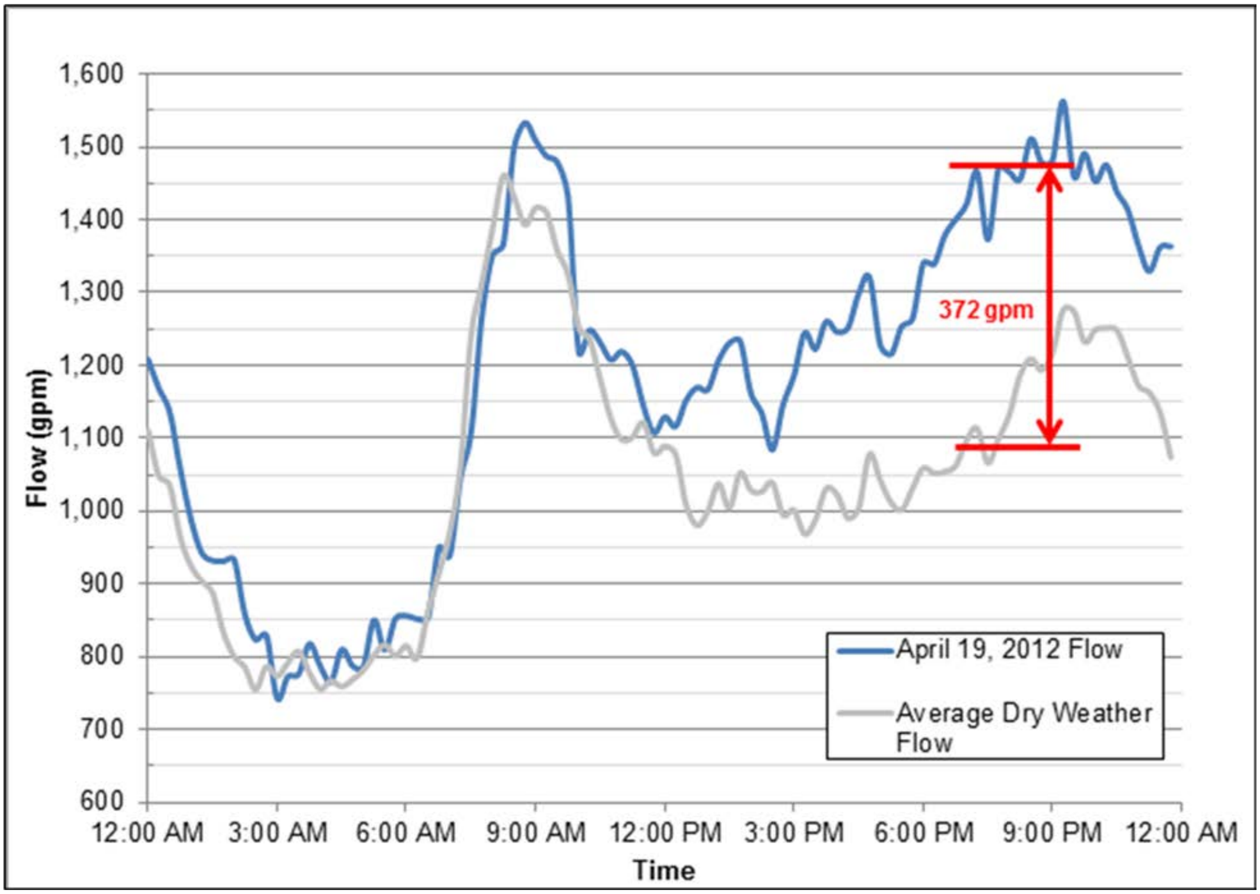


Figure 2.06-1 Kennedy-30 Inflow Calculation

It is assumed that the difference between the average adjacent Thursday weather flow and the peak 15-minute flow is the inflow to the collection system. The peak 15-minute flow occurred at 7:45 P.M. and resulted in a difference of 372 gpm, or approximately 536,000 gpd. Applying this inflow value over the entire Kennedy-30 service area or 2,313 acres results in an inflow rate of approximately 232 gpd/ac.

2.07 FLOW CONTRIBUTION CALCULATIONS

Flow meter data was also used to determine appropriate values for expected flow contributions from residential, commercial, and industrial areas of the Village. Typical residential contribution was calculated by finding an average daily flow for each metered area throughout the observation period, removing data obtained during wet weather. Table 2.07-1 lists the flow average daily flows for each area:

Meter Location	Contributing Basin(s)	Approximate Population	Average Daily Flow (gpd)	Average Residential Flow (gcd)
Fairbrook	NE-2	740	53,000	70
Century	E-2	1,140	37,500	30
Muirfield	NW-1	700	50,200	70
Division	North Village	9,490	1,086,150	110
Kennedy	All Village	12,100	1,220,420	100
			Average	**80**

*Note that the 2013 metered flows for the Division Street and Kennedy Street meter have been adjusted to remove the 2013 flows from the Village of Dane Interceptor (51,700 gpd) and Scientific Protein Labs (300,000 gpd)

Table 2.07-1 Average Residential Flow Contribution

As shown in Table 2.07-1, the average daily flows from each of the meters were compared to the approximate population contributing to that sewer line. The approximate population for each area was determined by multiplying the number of homes by the 2010 census population density of 2.69 residents per home. In addition, approximate resident totals from long-term health care facilities such as Waunakee Manor and Cannery Row Senior Center were included, as well as approximate student population totals from area schools. The average daily flow from each basin was then divided by the approximate population served to establish measured average daily flow of gallons per capita per day (gcd) for each basin. As shown above, these values ranged from 30 to 110 gcd depending on the area served. This variability can be contributed in part to the variety of land uses served within each basin. For instance, the Fairbrook, Century, and Muirfield meters were all placed in lines that generally serve residential areas of the Village. The Division and Kennedy meters were placed in the MMSD interceptor sewer, which includes commercial and industrial flows as well. As noted, the flows to these lines were adjusted to account for the flows contributed by the Village of Dane (51,700 gpd) and Scientific Protein Laboratories (300,000 gpd). However, the remaining commercial and industrial point sources were not investigated and, therefore, were not separated out from the raw flow data. An average value for all of the metered areas was calculated to be 80 gcd. This number is consistent with values commonly used for projecting residential sewer flows as well as with previous studies completed for the Village.

SECTION 3
EXISTING COLLECTION SYSTEM AND CAPACITY SUMMARY

Updates to this section include revised figures and tables to reflect areas of development and collection system improvements made since the 2013 Plan.

3.01 EXISTING COLLECTION SYSTEM SUMMARY

As shown in Figure 2.01-1, the Village's collection system consists of underground gravity sewer ranging in size from 8 inches to 24 inches and three pumping stations and associated force main. These collectors drain to the MMSD's 24- and 30-inch interceptor. The Village of Dane also discharges to MMSD by using the Village's collection system through an intermunicipal agreement. Through that agreement, the Village of Dane is allotted up to 1.075 cubic feet per second (482 gpm) of flow. Discharge from the Villages of Dane and Waunakee eventually end up at the MMSD Nine Springs wastewater treatment plant.

3.02 PUMPING STATIONS

The Village currently owns and operates three pumping stations to serve developments that are too low in elevation to utilize gravity sewer. The Meadowbrook Pumping Station, located at 900 Countryside Crossing in the northwest portion of the Village, was constructed in 2000 and discharges to a manhole on Kopp Road. The Blue Ridge pumping station, located on Peaceful Valley Parkway in the southern portion of the Village, was constructed in 2005 and discharges to a manhole on Shenandoah Drive. The Westbridge Pumping Station, located on Kopp Road in the northwest portion of the Village, was constructed in 2012 and discharges to the same manhole on Kopp Road that the Meadowbrook Pumping Station discharges to. All three pumping stations contain submersible pumps in a precast wet well, a separate valve vault, and natural gas powered standby generators.

3.03 EXISTING CAPACITY

The existing collection system was broken into separate sewersheds based on the general flow path through each area into the MMSD interceptor sewer. For the purpose of this study, a route through each sewershed was designated as the local interceptor for that area regardless of pipe size. Each local interceptor route was chosen as the main collector of the existing basin as well as the logical route to serve future development. Figure 3.03-1 shows the route of each interceptor, along with the boundaries of each tributary sewershed. The interceptors designations for each region are:

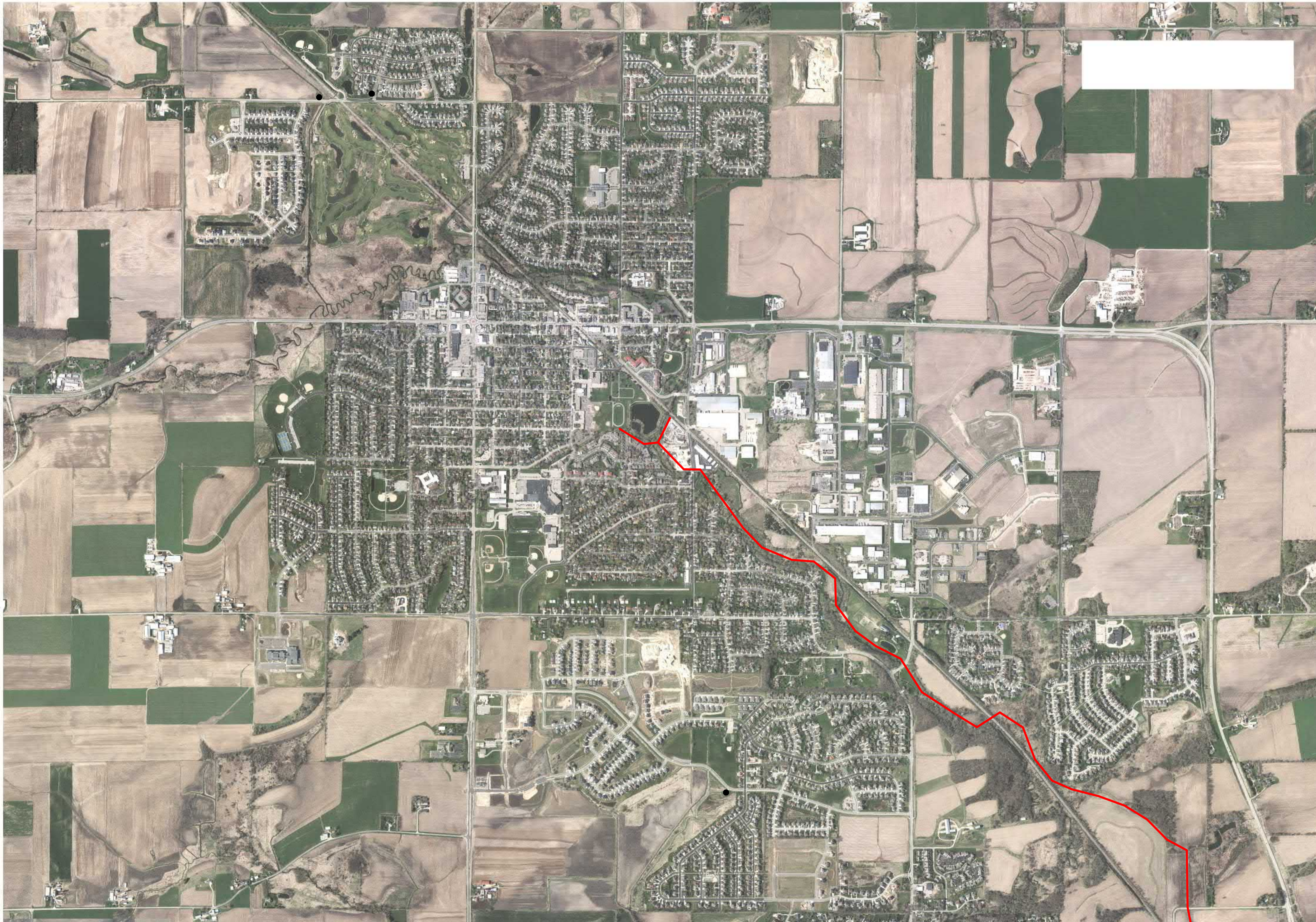
Eastern Region
- Bongard Drive
- Ravine
- Industrial Park

Southwestern Region
- Southside
- Endres

Northern Region
- Sixmile
- Division Street
- Northwest
- Northeast

Note that a central component of the Village bounded by Knightsbridge Road on the north and Dover Drive to the south drains directly to the MMSD interceptor, so this area was used to verify existing flow contributions only, and was not analyzed for capacity or future service capabilities.

Each of the interceptors was analyzed to determine the existing theoretical capacity. Invert elevation, pipe size, and pipe length were determined from record drawings of the existing lines. In addition,

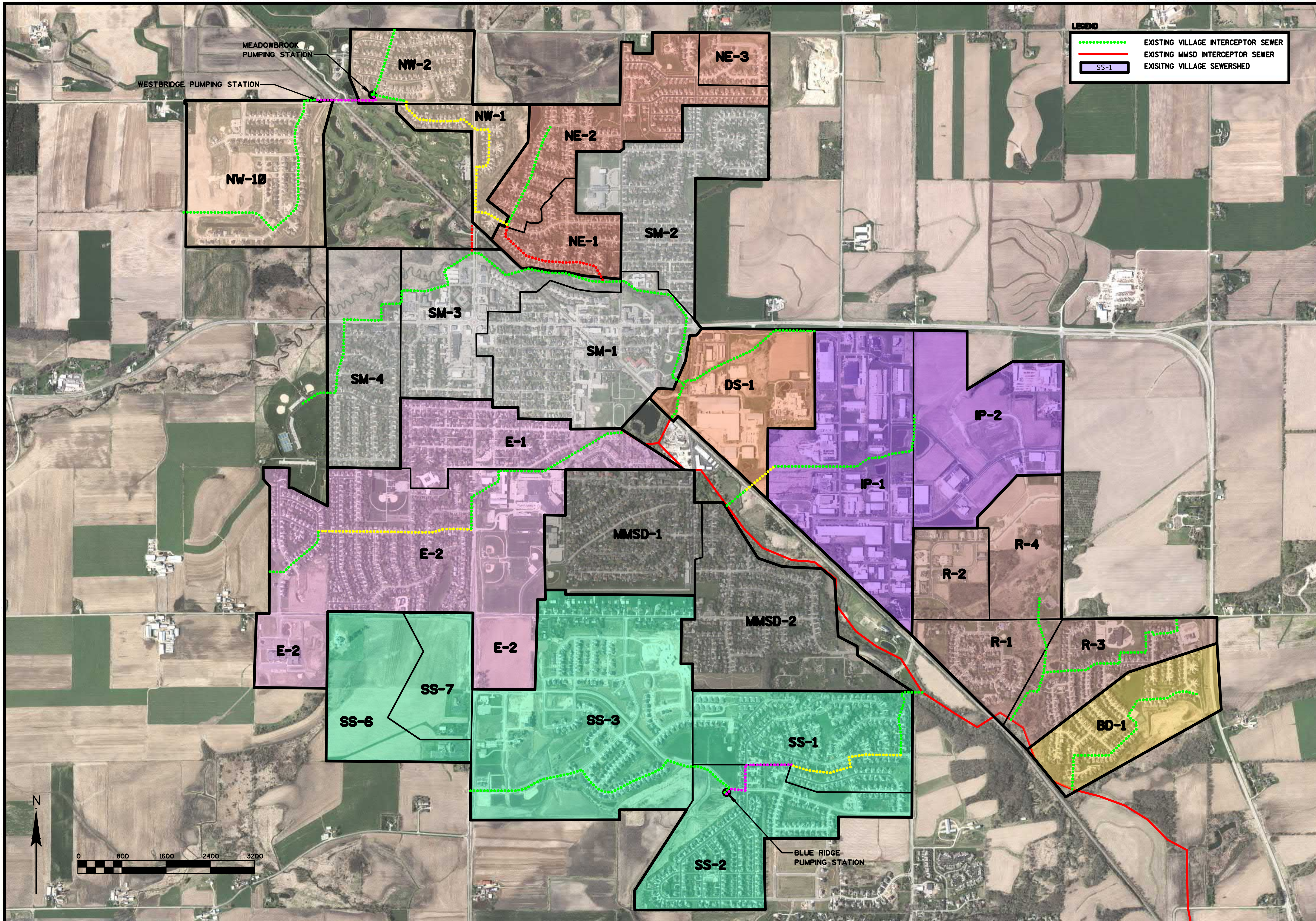


Strand Associates, Inc.® completed a topographic survey in 2012 to supplement some of the missing or incomplete information. It should be noted that vertical datums may have varied between respective plan sets and may not match with current survey information. As such, the pipe slope calculated for segments of sewer transitioning from record data information to survey information should be considered approximate, and further investigation may be prudent where these lines account for minimum capacity of the interceptor itself.

Theoretical capacity was calculated using Manning's equation, assuming an *n*-value of 0.013 and pipes flowing full. A summary of the existing capacities can be found in Appendix A.

3.04 EXISTING AREA FLOW CONTRIBUTIONS

Each sewershed was further broken down into subbasins in order to determine the expected flows contributed to each interceptor sewer. The subbasins are shown in Figure 3.04-1. For each basin, a single-family residence [dwelling unit (DU)] count was taken from aerial photography and the total number of residences was multiplied by the 2010 census population density of 2.69 persons per DU to calculate an equivalent population for that subbasin. This population equivalent was then multiplied by 80 gcd (the average per capita daily wastewater contribution discussed in Section 2.07) to calculate a base residential flow for that subbasin. Note that for the purposes of this study, small-scale multifamily residences were assumed to have the equivalent number of DUs. For example, a four-unit apartment complex was counted as four DUs. A summary of those calculations is shown in Table 3.04-1.



EXISTING SERVICE AREA SUBBASIN OVERVIEW

SANITARY SEWER COMPREHENSIVE PLAN

VILLAGE OF WAUNAKEE
DANE COUNTY, WISCONSIN

NO.	REVISIONS	DATE:

JOB NO.
1602.100

PROJECT MGR.
GSS

STRAND ASSOCIATES®

FIGURE
3.04-1

$FILE$ $USER$ $DATE$ $TIME$

Basin ID	Residential			
	Area (acres)	DUs	Equivalent Population	Base Flow (gpd)
BD-1	101.0	192	517	41,360
R-1	55.0	110	296	23,680
R-2	2.0	4	11	880
R-3	84.0	132	356	28,480
R-4	80	113	304	24,320
SS-1	146.0	281	756	60,480
SS-2	153.0	234	630	50,400
SS-3	270	383	1,030	82,422
SS-6	69.6	116	313	25,040
SS-7	47.1	493	1,327	106,160
IP-1	0.0	0	0	0
IP-2	0.0	0	0	0
IP-3	0.0	0	0	0
E-1	111.0	325	875	70,000
E-2	205.0	422	1,636	130,880
DS-1	3.0	16	44	3,520
SM-1	130.0	250.0	1,023	81,840
SM-2	152.0	288.0	775	62,000
SM-3	120.0	212.0	571	45,680
SM-4	120.0	237.0	638	51,040
NE-1	55.9	144.0	388	31,040
NE-2	130.0	275.0	740	59,200
NE-3	44.6	94	253	20,240
NW-1	52.0	168.0	452	36,160
NW-2	70.0	90.0	243	19,440
NW-10	135	283	762	60,960
MMSD-1	126.0	424.0	1,141	91,280
MMSD-2	132.0	272.0	732	58,560
		Totals	15,813	1,265,062

Table 3.04-1 Residential Wastewater Contributions

Within each sewershed, large-scale contributors such as schools, apartment complexes, senior care centers, and major industrial operations were identified as point-source contributions to the system. Flows for each institution were calculated based on industry standards and the results are shown in Table 3.04-2.

Basin ID	Facility Name	Total	Unit	Base Flow (gpd)
R-2	Arboretum Elementary School	511.0	Students	12,775.0
E-2	Waunakee Manor	104.0	Beds	5,200.0
E-2	Waunakee High School	1,101.0	Students	27,525.0
E-2	Woodland Schools	700	Students	17,500
MMSD-1	Waunakee Middle School	564.0	Students	14,100.0
DS-1	Scientific Protein Labs	-	-	216,000.0
SM-1	Cannery Row Senior Living Center	131.0	Beds	6,550.0
SM-1	St. John the Baptist School	100.0	Students	2,500.0
SM-1	Heritage Elementary School	311.0	Students	7,775.0
SM-1	Intermediate School	560.0	Students	14,000.0
SM-2	Prairie Elementary School	511.0	Students	12,775.0
R-2	BrightStar Senior Living	36	Beds	1,800
SS-3	At Home Again Assisted Living	70	Beds	3,500
IP-2	Octopi Brewing	-	-	9,300

Notes:

Flows for Scientific Protein Labs are based on the current discharge of 288,000 gpd and the plan to reduce discharge by 144,000 gpd in 2013 and then add 72,000 gpd by 2018 (according to Scientific Protein Labs Representatives)

The Octopi Brewing base flow is based on the recent average annual flow. Monthly high flows in June and July 2018 averaged 11,160 gpd.

Table 3.04-2 Point Source Wastewater Contributions

The residential and point-source contributions were then used to calculate an assumed value for commercial and industrial contributions. As mentioned in Section 2.07, a full investigation into the commercial and industrial property sewer contributions was not included in the scope of this study. Instead, existing land use mapping was used to measure the areas of the commercial and industrial properties within each basin, and the area was multiplied by an assumed wastewater contribution per acre of land to calculate an expected flow for each subbasin. The contribution factors were calculated by subtracting the total residential and point-source contribution from the measured average total daily flow, and then dividing that total by the measured area of commercial and industrial properties. Based on that calculation, wastewater contribution was assumed to be 1,000 gpd/ac for commercial properties and 1,500 gpd/ac for industrial properties. These factors were then multiplied by the area of each property with each subbasin to calculate the expected flows for that basin. A summary of those calculations is found in Table 3.04-3.

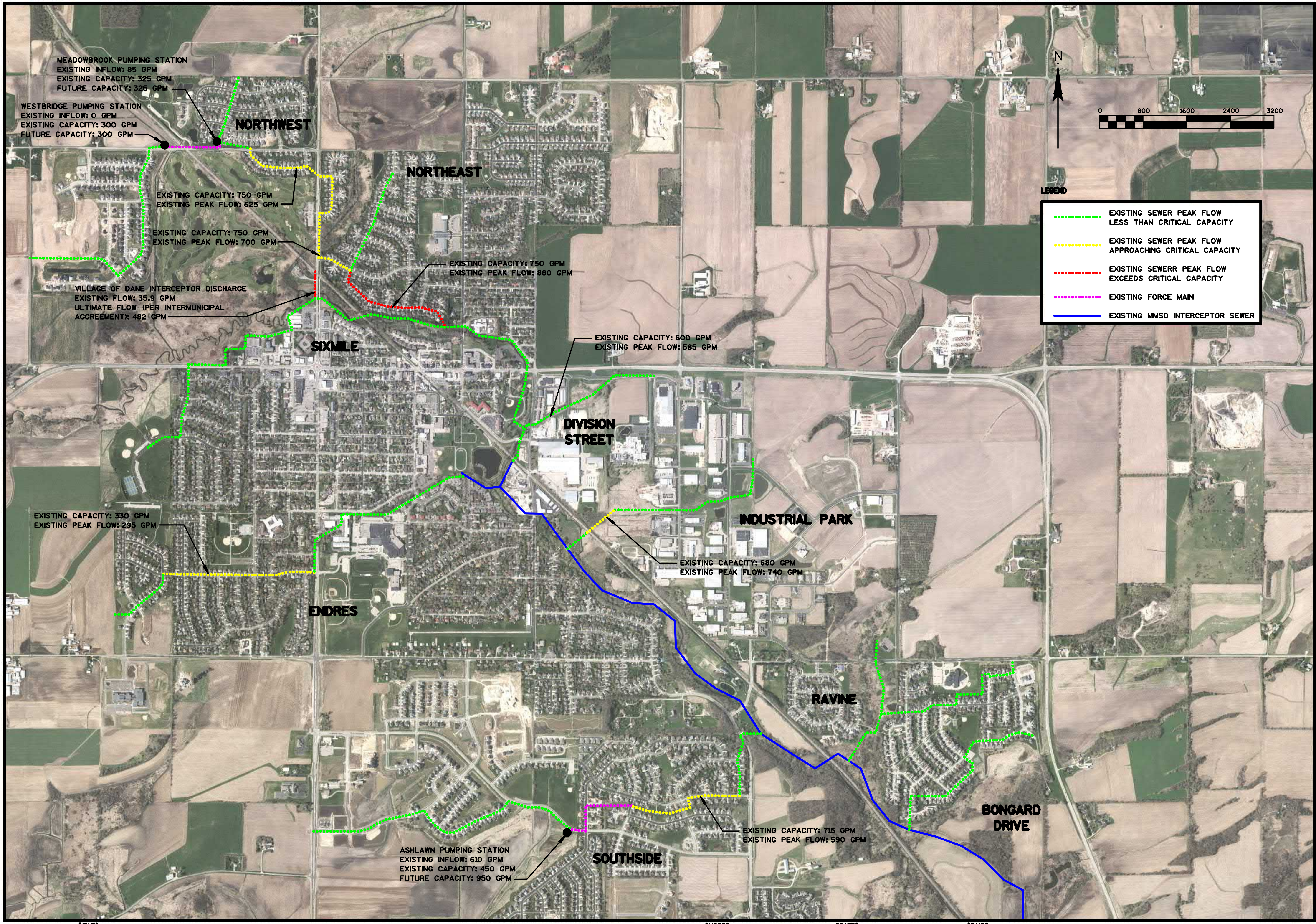
Basin ID	Commercial		Industrial	
	Area (acres)	Base Flow (gpd)	Area (acres)	Base Flow (gpd)
BD-1	0.0	0	0.0	0
R-1	0.0	0	0.0	0
R-2	15.0	1,590	0.0	0
R-3	18.0	4,320	0.0	0
SS-1	0.0	0	0.0	0
SS-2	0.0	0	0.0	0
SS-3	43.2	43,200	0.0	0
IP-1	3.5	280	200.0	300,000
IP-2	0.0	0	50	75,000
IP-3	0.0	0	0	0
E-1	3.0	6,000	0.0	0
E-2	30.4	36,480	10.0	15,000
DS-1	2.0	0	70.0	105,000
SM-1	37.0	0	0.0	0
SM-2	37.8	45,300	0.0	0
SM-3	32.0	800	0.0	0
SM-4	0.0	0	0.0	0
NE-1	0.0	0	0.0	0
NE-2	0.0	0	0.0	0
NW-1	0.0	0	0.0	0
NW-2	0.0	0	0.0	0
MMSD-1	0.0	0	1.0	1,500
MMSD-2	0.0	0	0.0	0

Table 3.04-3 Commercial and Industrial Wastewater Contributions

The base flow contributions from the residential, commercial, industrial, and point sources were then totaled for each basin, and a peaking factor of 2.5 was applied. Peak infiltration and inflow was calculated for each basin using the peak flow densities discussed in Section 2. These totals were added to the peak flow contributions to determine the total peak flow rate for each basin. A summary of the existing flow contributions can be found in Appendix B.

3.05 CRITICAL CAPACITY SEWERS

The existing flow contributions were then compared to the theoretical capacities of the existing interceptors to identify portions of the existing sewer system that may be vulnerable to surcharging during peak flow events. Figure 3.05-1 highlights these lines and lists the theoretical capacities and peak flow rates.



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NO.

EXISTING SEWER CAPACITY LIMITATIONS

SANITARY SEWER COMPREHENSIVE PLAN

VILLAGE OF WAUNAKEE
DANE COUNTY, WISCONSIN

JOB NO.
1602.100

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FIGURE
3.05-1

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3.06 VILLAGE OF DANE FLOW CONTRIBUTION

In addition to the local flow contributions, the Village also conveys flows from the Village of Dane via a connection of an existing force main to the Sixmile Interceptor Sewer near the North Century Avenue Bridge. As discussed in Section 2, in 2018, the Village of Dane contributed an average of 56,335 gpd (39 gpm) to the system, and this flow rate was used to determine the current local flow contributions. However, the allotted capacity of 1.075 cubic feet per second (482 gpm) listed in the Waunakee/Dane Municipal Service Agreement was used to calculate the remaining capacity of the system by subtracting this flow from the downstream sewers.

SECTION 4
ULTIMATE SERVICE AREA

This section has been updated to reflect areas of recent development and revisions to future service areas. The methodology is unchanged from the 2013 Study.

4.01 ULTIMATE SERVICE AREA DELINEATION

Each interceptor sewer was evaluated for extension into the lands surrounding current development limits of the Village. Topographic contours were used to determine the most efficient route to extend each sewer and determine the approximate limits available for service via gravity sewer. This Ultimate Service boundary, along with the current developed limits and municipal boundary, is shown on Figure 4.01-1. The ultimate service area was broken into three regions, and these regions were broken into subbasins corresponding to the receiving interceptor in the same fashion as the existing sewersheds discussed in Section 3. These regions and subbasins are shown in Figures 4.01-2, 4.01-3, and 4.01-4.

4.02 ULTIMATE SERVICE AREA SUBBASIN COMPOSITION

With limited information regarding land use planning outside of the current Village development limits, each subbasin within the three service regions was given an assumed composition for future development. This composition was broken down by percentage for residential, commercial, and industrial development based on its proximity to transportation facilities and classification of adjacent existing development. In general, industrial properties were designated in and around the existing industrial park, and commercial development was assumed to extend along the existing highways and around the existing business park. The majority of future development was assumed to be residential. Figure 4.02-1 shows the breakdown of each subbasin, and a summary of this information can be found in Appendix C.

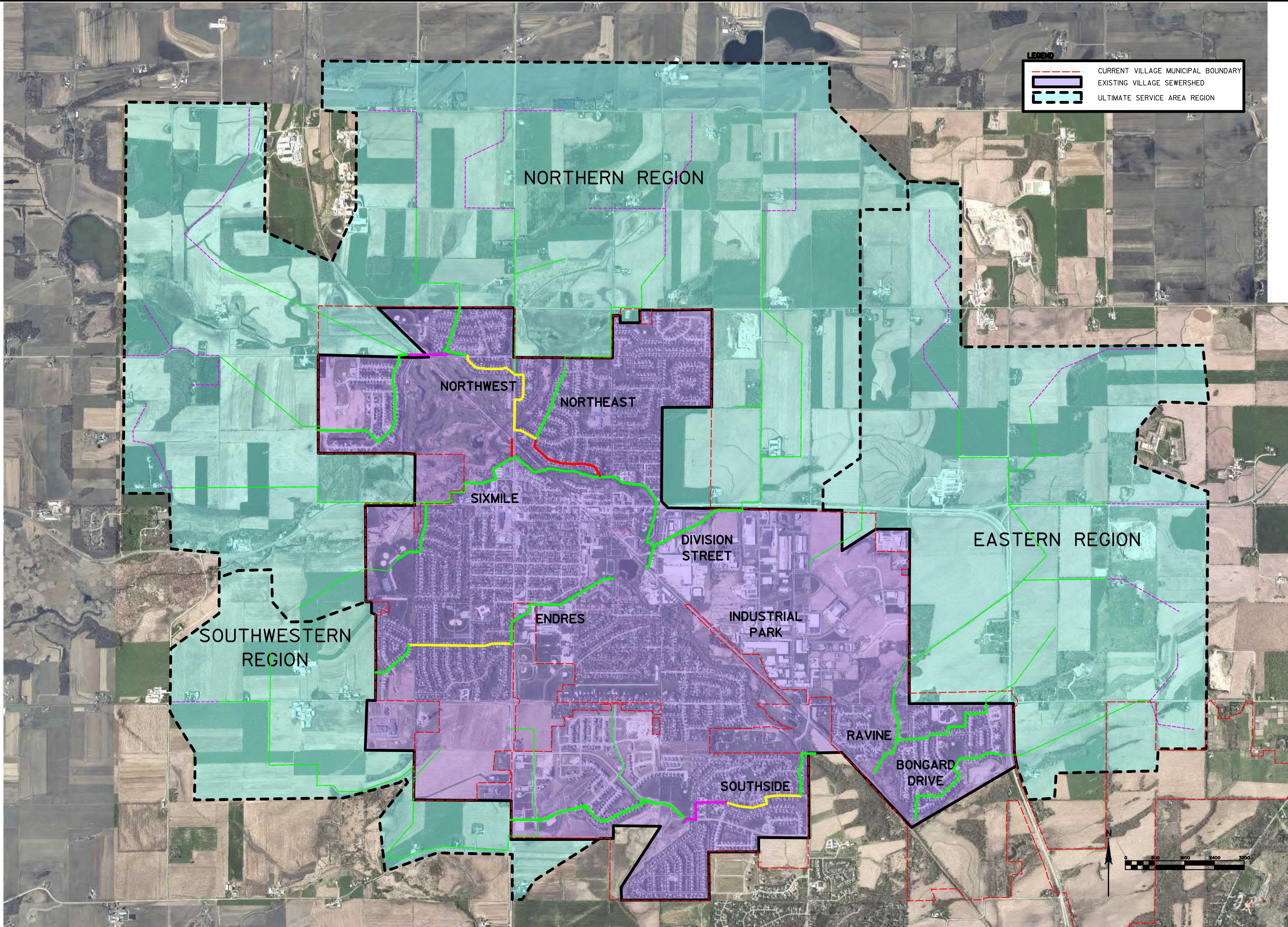
4.03 FUTURE RESIDENTIAL FLOW CONTRIBUTIONS

Flow contribution from areas of future development was calculated as a factor of the area and the assumed development classification. As discussed in Section 4.02, each subbasin was given a designated development breakdown by percentage of area. These classifications included residential (R-1, R-2, R-3, and R-5) as well as commercial (C-1) and industrial (I-1).

The first step in calculating residential flow contributions is to determine the population equivalent per gross acre of land. Using the Village Ordinances on minimum lot size as a reference, each classification was broken down into minimum DUs per acre (DUs/Ac) and then multiplied by the 2010 census value of 2.69 residents per DU. For each lot, an additional 100 percent of free space was included in the calculation to account for lawn area, green spaces, and roadways. For example, the equivalent population for an R-1 zone is:

R-1 Dwelling Units Per Acre:
43,560 SF/Ac /[9,500 SF/Lot (Min. Lot Size) + 9,500 SF/Lot (free space per lot)] = 2.30 DU/Ac

R-1 Equivalent Population per Acre:
2.30 DU/Ac x 2.69 residents/DU (per 2010 census) = 6.19 residents/ac



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ULTIMATE SERVICE AREA OVERVIEW

SANITARY SEWER COMPREHENSIVE PLAN

VILLAGE OF WAUNAKEE
DANE COUNTY, WISCONSIN

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FIGURE
4.01-1

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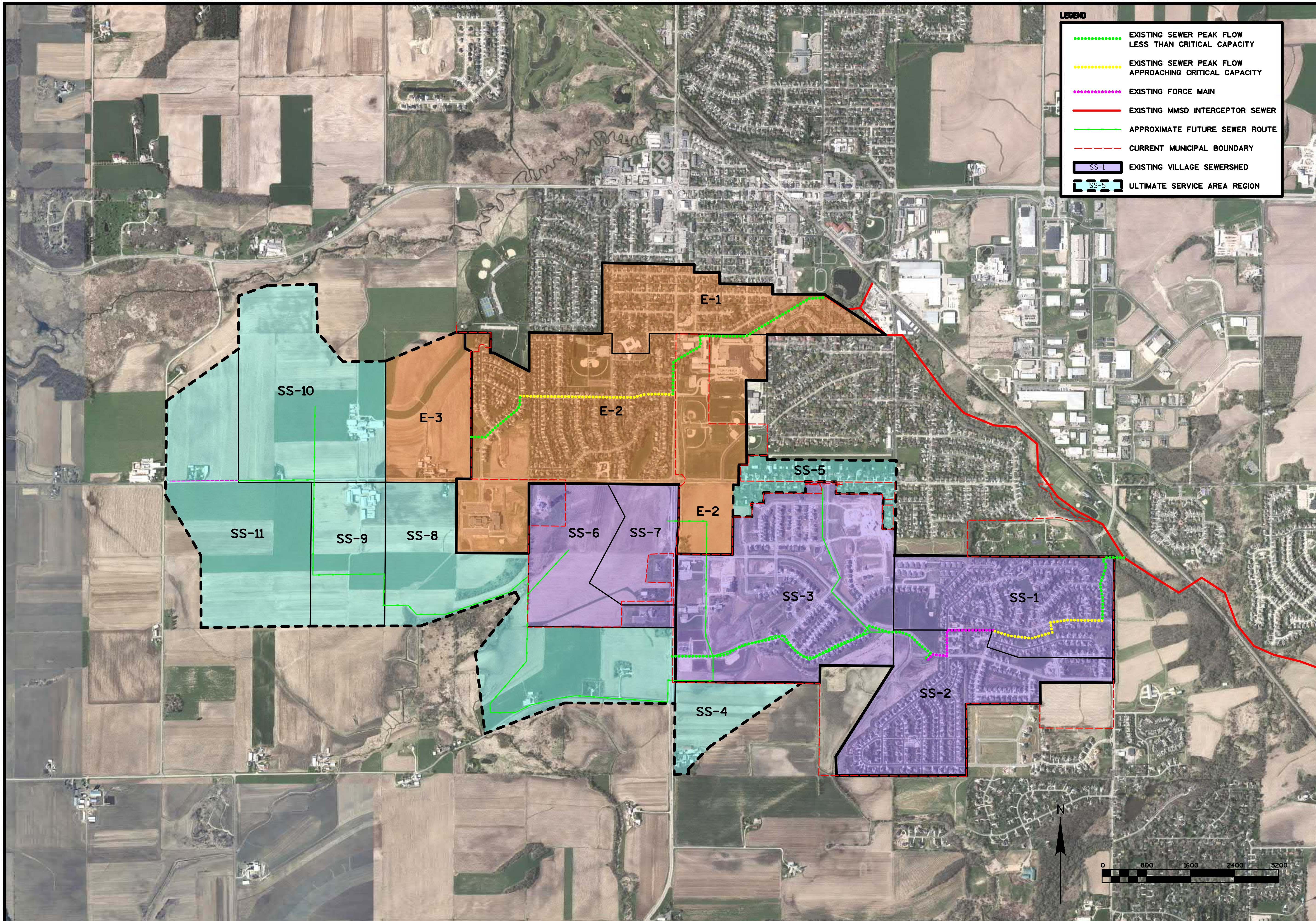

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ULTIMATE SERVICE AREA
SOUTHWESTERN REGION

SANITARY SYSTEM COMPREHENSIVE PLAN

VILLAGE OF WAUNAKEE
DANE COUNTY, WISCONSIN

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FIGURE
4.01-2

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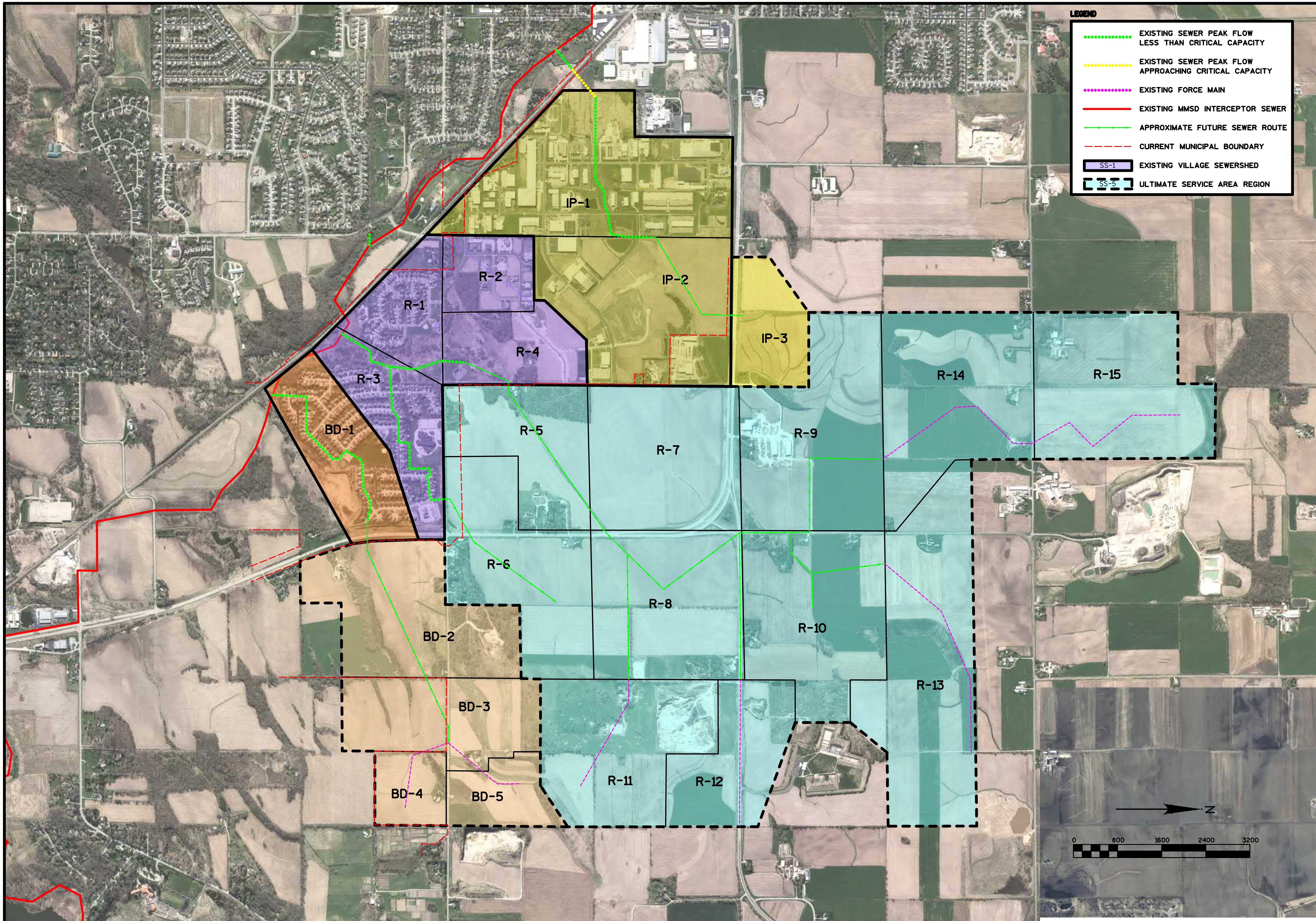

ULTIMATE SERVICE AREA
EASTERN REGION
SANITARY SYSTEM COMPREHENSIVE PLAN

VILLAGE OF WAUNAKEE
DANE COUNTY, WISCONSIN

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FIGURE
4.01-3

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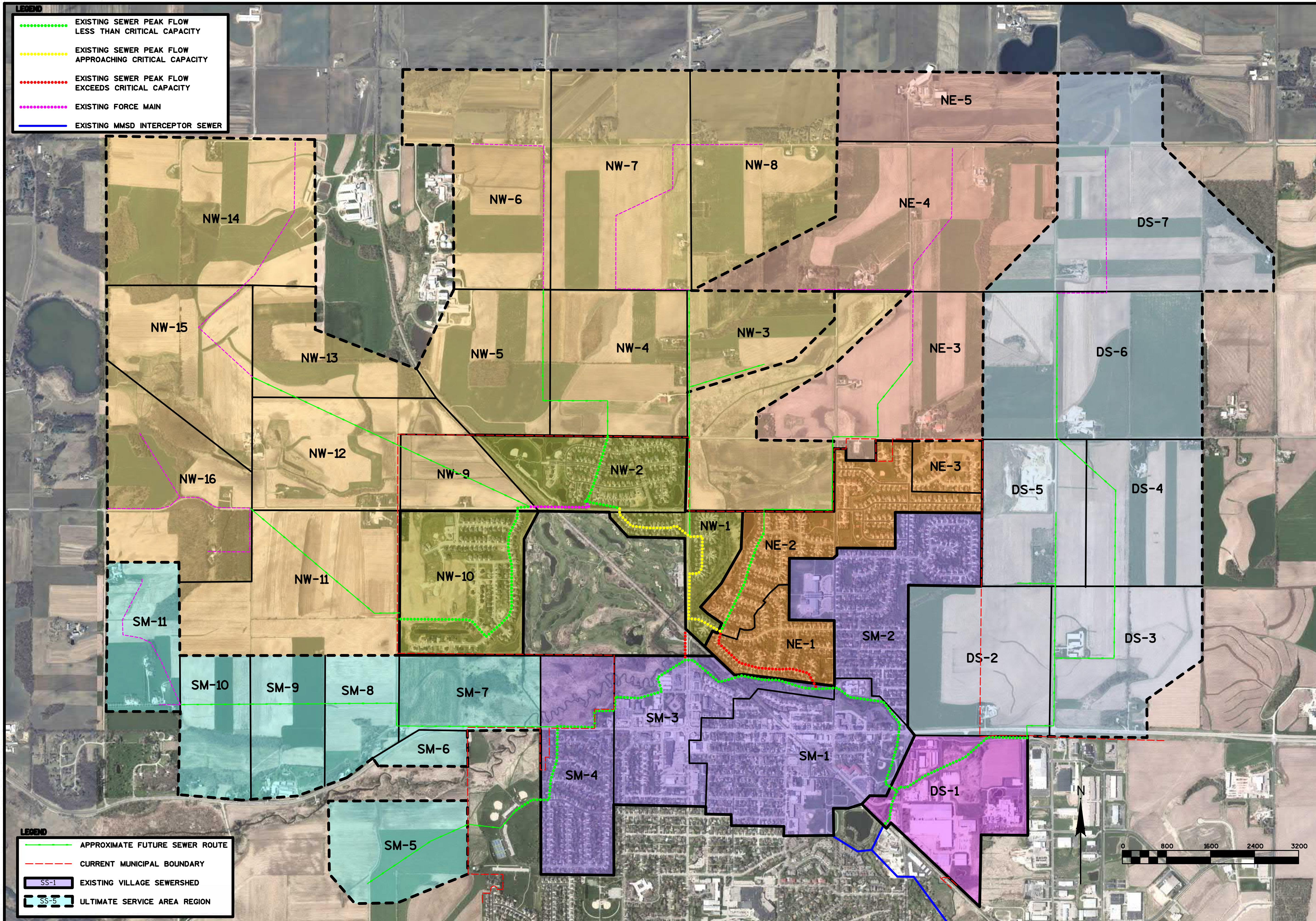

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ULTIMATE SERVICE AREA NORTHERN REGION

SANITARY SEWER COMPREHENSIVE PLAN

VILLAGE OF WAUNAKEE
DANE COUNTY, WISCONSIN

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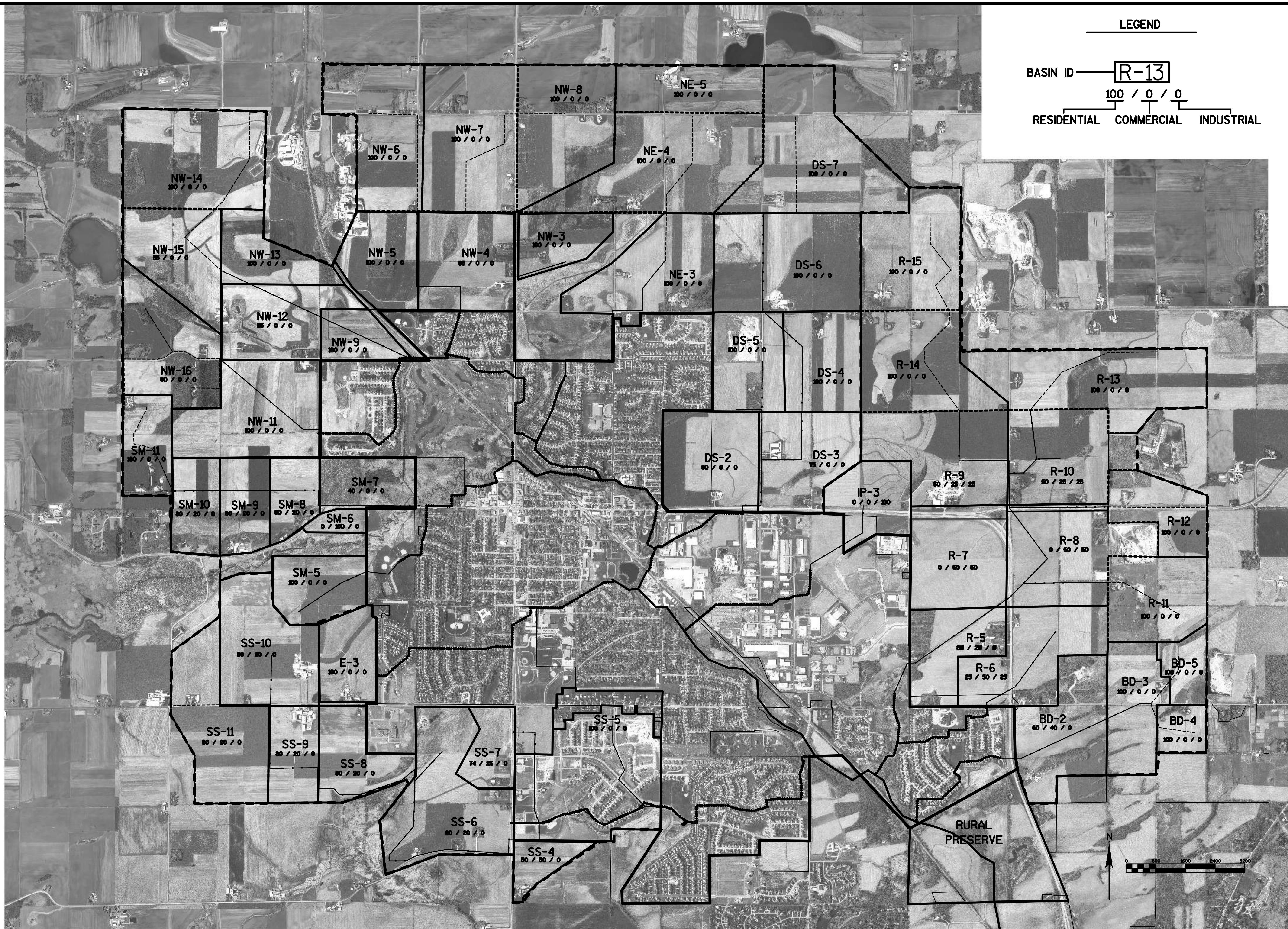
FIGURE
4.01-4



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SERVICE AREA COMPOSITION

SANITARY SEWER COMPREHENSIVE PLAN

VILLAGE OF WAUNAKEE
DANE COUNTY, WISCONSIN

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1602.100

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FIGURE
4.02-1

S:\MAD\1600--1699\1602\125\Drawings\CAD\Plan\4.02-1 Service Area Composition_New.dgn user: dimitriosh 10/5/2018 10:30:50 AM

Existing R-1 developments within the Village were then analyzed to calibrate the theoretical values. The following results indicate a strong correlation between the theoretical values and existing conditions:

1. Westview Meadows–1.63 DU/ac
2. Dormal Heights–2.65 DU/ac
3. Westridge–2.02 DU/ac

Equivalent population density was then multiplied by the per capita flow contribution of 80 gcd discussed in Section 2.07 to determine a theoretical dry weather base flow, and peaking factors were applied to calculate a theoretical peak dry-weather wastewater contribution per gross acre of land. Infiltration was then calculated using the maximum allowable rate of 200 gpd/In-Mile-Ac according to the Wisconsin Administrative Code. This figure was then added to the dry weather peak flow to calculate the total peak wastewater contribution per acre. A summary of these calculations can be found in Appendix C.

4.04 FUTURE COMMERCIAL AND INDUSTRIAL FLOW CONTRIBUTIONS

Assumed wastewater contribution factors were used for commercial (C-1) and industrial (I-1) properties. Engineering standards for these commercial and industrial properties are 2,000 gpd/ac and 2,500 gpd/ac, respectively. However, the values used in this study were adjusted to reflect the type of commercial and industrial development expected for the Village. The Kilkenny Commons mixed-use commercial development was used as a reference for this calculation. The development plan for that project was used to itemize the types of businesses within the development, and industry standards were used to develop typical flows for each business. The total flow expected for this development was approximately 54,800 gpd, or 1,000 gpd/ac. Based on this figure, flow contributions for future commercial and industrial development were adjusted to be 1,200 and 2,000 gpd/ac, respectively.

4.05 PEAKING FACTORS

It should be noted that for residential properties, the appropriate peaking factor depends on the contributing basin size (or equivalent population). When analyzing an individual basin, the appropriate peaking factors are:

1. Basin Size < 250 Ac = Peaking Factor 4.0
2. Basin Size 250 Ac-500 Ac = Peaking Factor 3.5
3. Basin Size >500 Ac = Peaking Factor 2.5

However, when examining the impacts on an interceptor sewer, the cumulative tributary area generally exceeds 500 acres in size. As such, a peaking factor of 2.5 was applied to each subbasin within the ultimate service area, regardless of individual size.

4.06 ULTIMATE SERVICE AREA FLOW CONTRIBUTIONS

For each subbasin within the ultimate service area, the flow contribution was calculated by distributing the total area by the assumed land uses as described in Section 4.02 and then multiplying each area by the peak wastewater contribution factor described above. For example, the flow contribution for Subbasin R-6 was calculated as follows:

Subbasin R-6, Total Area: 165 acres

Subbasin Land Use Composition:

- 25% R-1: 41 Ac x 0.90 gpm/Ac = 38 gpm
- 50% C-1: 83 Ac x 2.10 gpm/Ac = 174 gpm
- 25% I-1: 41 Ac x 3.50 gpm/Ac = 150 gpm

Total Peak Flow Contribution: 38+174+150 = 362 gpm

A summary of the calculations for each subbasin is shown in Appendix C.

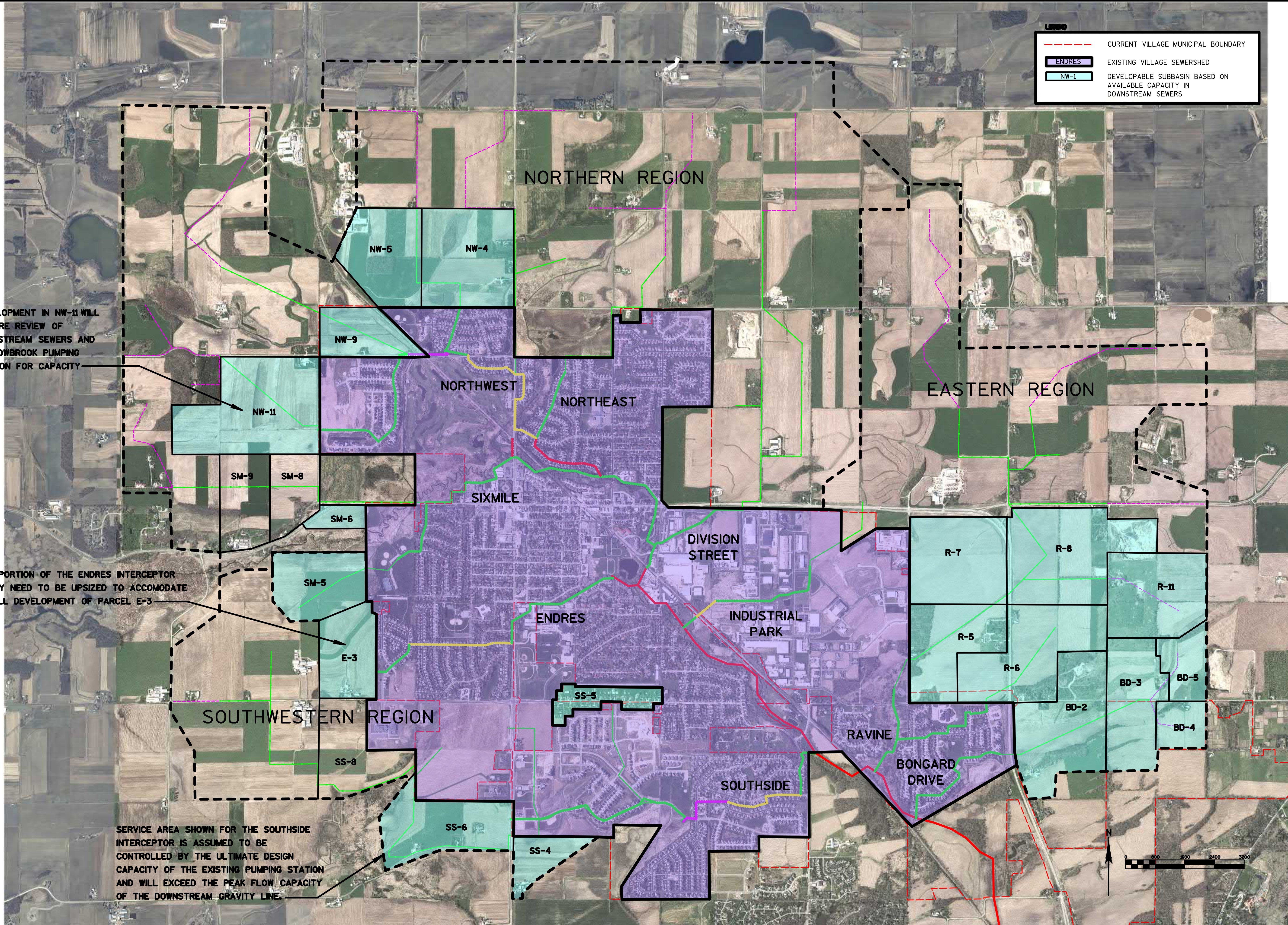
4.07 AVAILABLE CAPACITY SERVICE AREA

After the flow contributions for each subbasin were calculated, a schematic was assembled to determine the cumulative effects on the downstream sewers. Each interceptor was analyzed to determine the critical capacity segment, or the segment of piping with the lowest theoretical flow capacity (see Section 3.03). Existing flow contributions were applied to the schematic, and subbasins within the ultimate service area were added in logical order until downstream capacity was exceeded. For the purposes of this study, critical capacity was assumed to be reached once the lowest capacity was exceeded, and further analysis of allowable surcharging was not completed.

Figure 4.07-1 shows the approximate limits of the Available Capacity Service Area, or the area available for future development based on balancing the theoretical future peak flows and available downstream capacity. This area would be feasible for development following the assumed subbasin composition shown in Figure 4.02-1 and the interceptor extension routes shown in Figure 4.01-1.

4.08 ULTIMATE SERVICE AREA SEWER SIZING

As discussed in previous sections, one goal of this study was to provide viable routes to extend the existing interceptor sewer system to accommodate future development. These extensions are shown in Figures 4.01-1 through 4.01-4. Each of these extensions was then analyzed to determine the appropriate pipe sizing necessary to accommodate full development of all upstream subbasins within the ultimate service area. Note that this exercise differs from the available capacity service area discussion in Section 4.07, in that the pipe extensions were sized to accommodate all upstream subbasins, regardless of impacts to downstream sewers.



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AVAILABLE CAPACITY SERVICE LIMIT

SANITARY SEWER COMPREHENSIVE PLAN

VILLAGE OF WAUNAKEE
DANE COUNTY, WISCONSIN

JOB NO.
1602.100

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FIGURE
4.07-1

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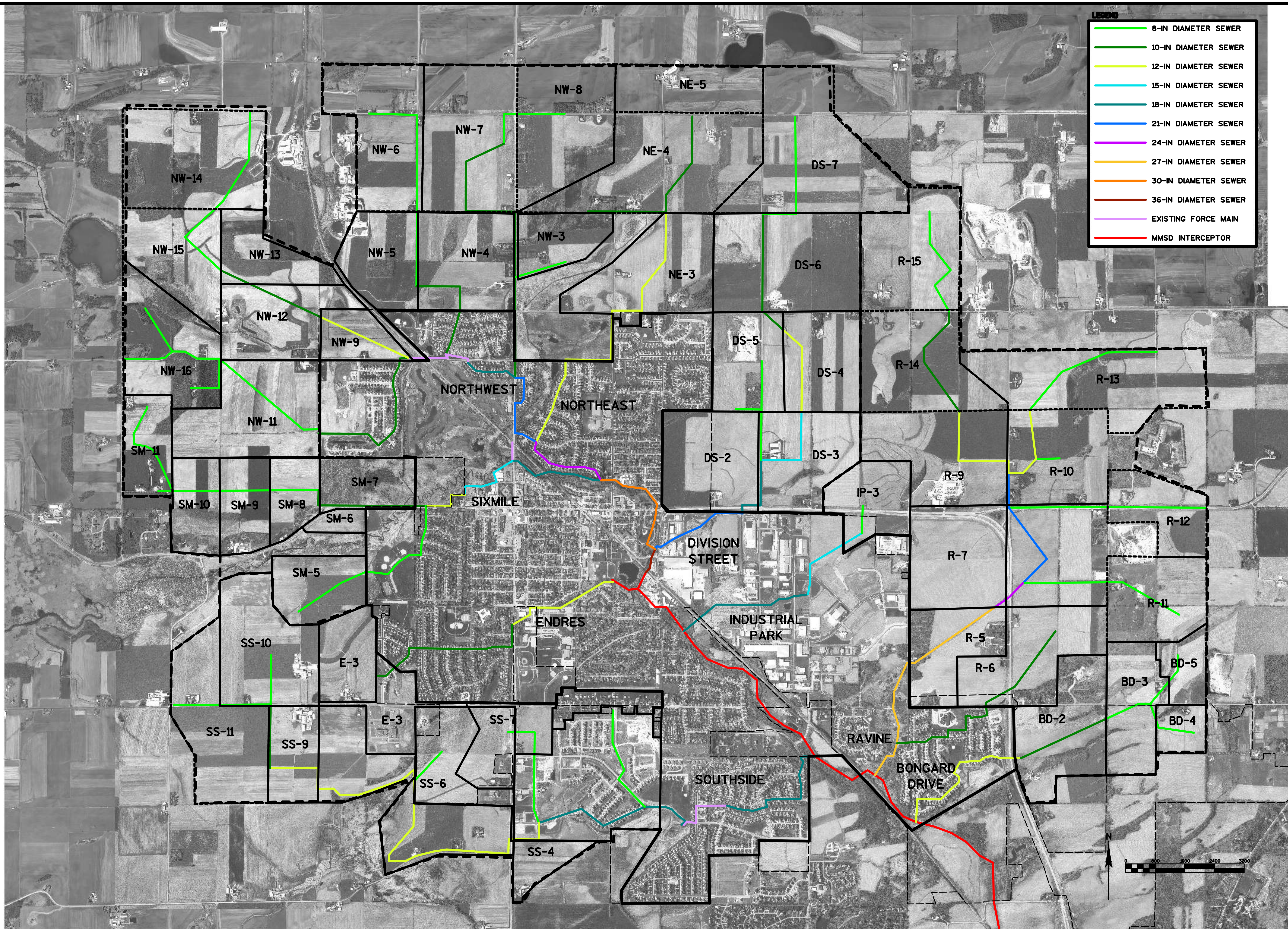
Figure 4.08-1 shows the approximate pipe sizes for each existing interceptor and the extensions required to serve the ultimate service area. Note that the flows for each subbasin were applied over the entire stretch of pipe serving that basin, and proposed developments will need to be investigated to determine the actual limits of each run of pipe. In addition, it was assumed that pipes with a diameter of 8 inches and up to 24 inches would be placed at minimum grade allowed by Wisconsin Administrative Code NR110. Pipes with diameters larger than 24 inches were assumed to be placed at a grade of 0.08 percent as a minimum achievable grade for normal construction practices. Assumed minimum grades are shown in Table 4.08-1.

Pipe Diameter (in)	Minimum Slope (%)	Theoretical Capacity (gpm)
8	0.40%	323
10	0.28%	490
12	0.22%	706
15	0.15%	1,057
18	0.12%	1,538
21	0.10%	2,117
24	0.08%	2,703
27	0.08%	3,701
30	0.08%	4,901
36	0.08%	7,968
42	0.08%	12,019

Table 4.08-1 Interceptor Sizing Calculations

Steeper grades may be available to serve future developments and may allow for a decrease in interceptor diameter while still providing the necessary capacity.



ULTIMATE SERVICE AREA INTERCEPTOR SIZING

SANITARY SEWER COMPREHENSIVE PLAN

VILLAGE OF WAUNAKEE
DANE COUNTY, WISCONSIN

NO.	REVISIONS	DATE:

JOB NO. 1602.100

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FIGURE 4.08-1

SECTION 5
CONCLUSIONS AND RECOMMENDATIONS

5.01 EXISTING SYSTEM INFILTRATION AND INFLOW (I/I)

The results of the flow monitoring completed for this study indicate a moderate amount of I/I is being received by the existing system. However, the limited monitoring coverage and timeframe made it difficult to identify the areas of highest contribution. The oldest portions of a sewer system generally contribute the highest I/I flows through damaged piping, illicit connections, root infiltration, and leaking structures. Reducing I/I to the system would provide additional capacity for sewage flows and may help alleviate critical capacity issues in some lines. A comprehensive I/I study should be completed to refine these findings and develop a list of prioritized improvements.

5.02 ULTIMATE SERVICE AREA POTENTIAL

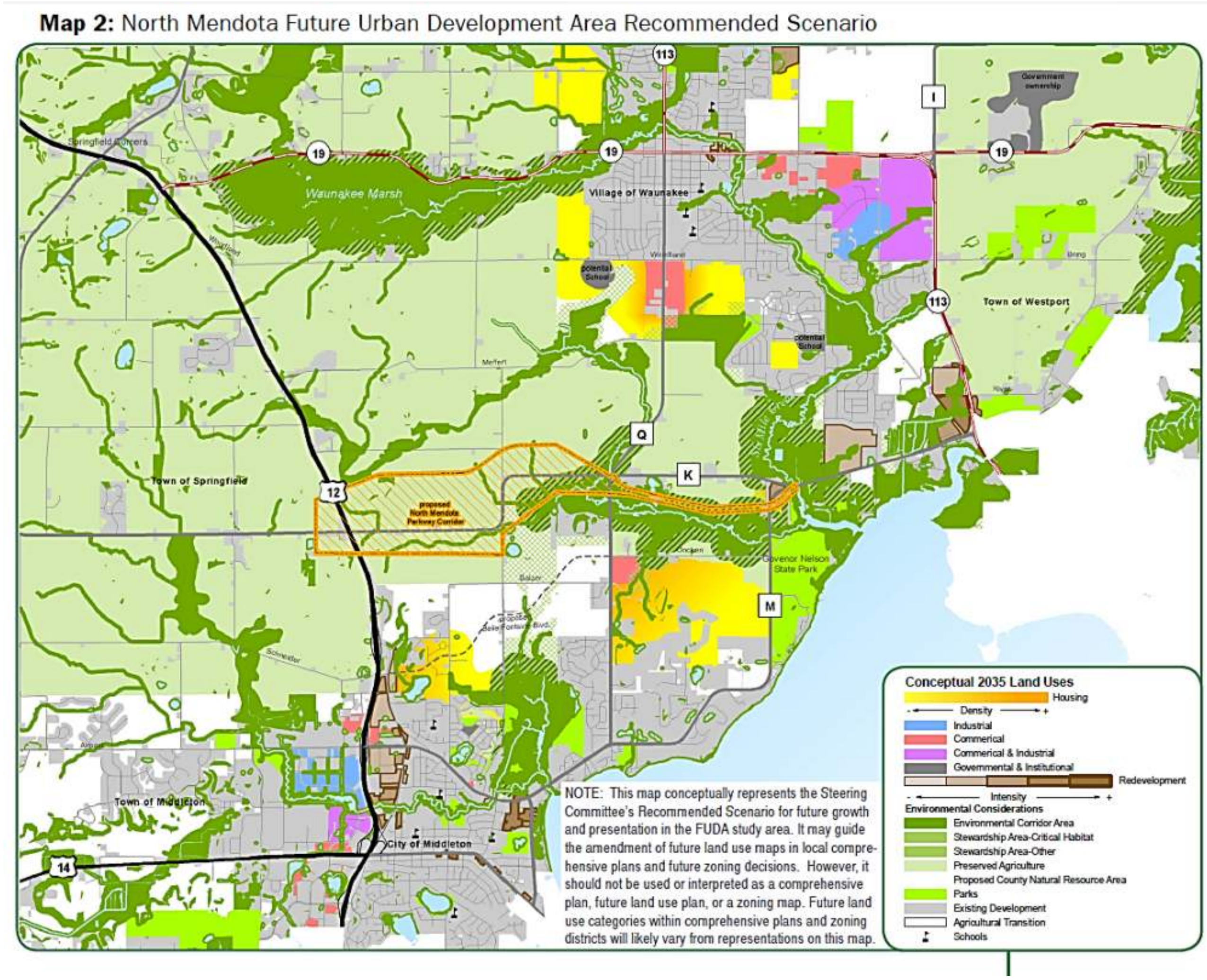
The existing sewer system arrangement appears capable of geographically serving the approximately 11,000-acre area shown in Figure 4.01-1 via gravity sewer with improvements to the existing pumping stations. Using the calculated theoretical population equivalent for R-1 properties at 6.19 residents per acre, this area equates to approximately 68,000 residents. Significant improvements would be required to existing infrastructure to meet this ultimate demand. However, the flow capacity of the current infrastructure limits development potential to the approximately 6,200-acre area shown (population equivalent 38,400) in Figure 4.07-1.

As development occurs and capital improvement projects to existing facilities are completed, improvements should be made as described in Section 5.06 to maximize the amount of area able to be served by the impacted interceptor sewer.

It is important to remember when reviewing the immediate service area boundary maps and tables, that lands shown to be immediately serviced, depending on timing of development of other areas of that region. However, development of several of these basins would require improvement to downstream sewer facilities.

5.03 NORTH MENDOTA FUTURE URBAN DEVELOPMENT AREA (FUDA) STUDY (JANUARY 2013)

The Village, City of Middleton, Town of Westport, and Town of Springfield have recently completed the North Mendota FUDA Study as a joint effort to evaluate the region for future development potential. Part of this study determined the conceptual land uses for development within the region, and these areas are highlighted Map 2 (page 5) of the study, which is shown in Figure 5.03-1.



Source: North Mendota FUDA Study

Figure 5.03-1 North Mendota FUDA Land Use Map (January 2013)

The areas highlighted in yellow represent the potential for residential development, and these areas are generally included within the limits of the available capacity service area shown in Figure 4.07-1. The main exception would be the property in the northeastern corner of the Village adjacent to the Waunakee Heights Neighborhood, which is highlighted for potential residential development and redevelopment of an existing quarry. This area is represented by future development subbasin DS-5 shown in Figure 5.03-2 (from Figure 4.01-4).

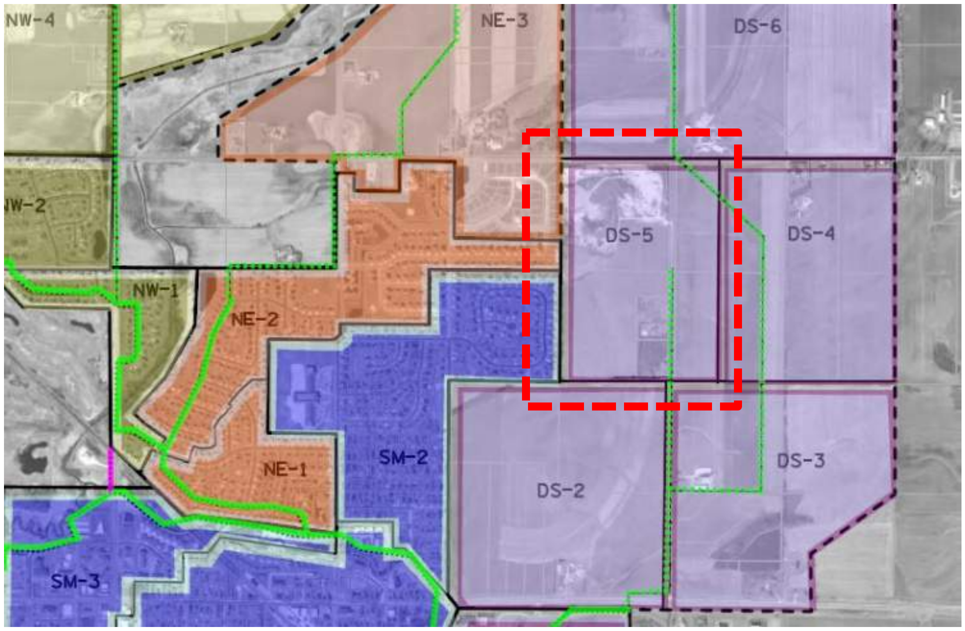


Figure 5.03-2 Quarry Area Properties

Development of these parcels are limited for future development by existing capacity in the downstream sewers, which currently carry flows from the Waunakee Industrial Park and, in particular, Scientific Protein Labs. Future development upstream of the Division Street interceptor should be evaluated to determine the potential land use and impacts to downstream sewers, and upgrades to existing facilities should consider development of the remaining ultimate service area of the interceptor. Pipe sizing for full development of the ultimate service area can be found in Figure 4.08-1.

5.04 AREAS OF POTENTIAL PENDING DEVELOPMENT

Through discussions with Village staff and experience with recent Village developments, we understand that certain areas within the Village are currently considered to have the highest potential for development in the near future, which are discussed further in detail.

A. Potential Development Area 1–Easy Street North Property

This property consists of future development subbasins NE-3 and NW-3, as shown in Figure 5.04-1 (from Figure 4.01-4), and would be serviced by extensions to the Northeast and Northwest Interceptors, respectively.

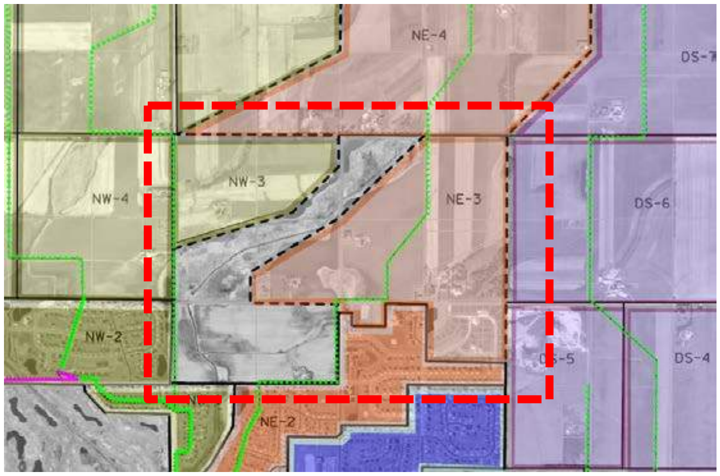


Figure 5.04-1 Easy Street North Property

Development of these parcels is restricted by available capacity in the Northwest Interceptor. Improvements to downstream sewers show in Figure 4.08-1 or construction of a relief sewer along Century Avenue as discussed in Section 5.03 could provide the additional capacity necessary to accommodate development of these parcels.

B. Potential Development Area 2–Breunig Farm Area Property

This area is situated just south of the Waunakee Heights Neighborhood and adjacent to Division Street to the west, Schumacher Road to the east, and STH 19 to the south. This parcel is contained within future development subbasin DS-2 as shown in 5.04-2 (from Figure 4.01-4).

Figure 5.04-2 Breunig Farm Area Property

This parcel could be served by an extension of the Division Street Interceptor Sewer or partially via sewer extensions from the SM-2 zone. Development of this property is limited by capacity in the existing Division Street Interceptor Sewer, which serves the existing Waunakee Industrial Park and, in particular, Scientific Protein Labs. Future development upstream of the Division Street interceptor should be evaluated to determine the potential land use and impacts to downstream sewers, and upgrades to existing facilities should consider development of the remaining ultimate service area of the interceptor. Pipe sizing for full development of the ultimate service area can be found in Figure 4.08-1.

C. <u>Potential Development Area 3–Kennedy Drive Property</u>

This area, as shown in Figure 5.04-3 (from Figure 4.01-2), is generally bounded by the Bongard Drive Interceptor Service Area on the north, Kennedy Drive on the south, the existing Wisconsin and Southern Railroad to the west, and STH 113 to the east.

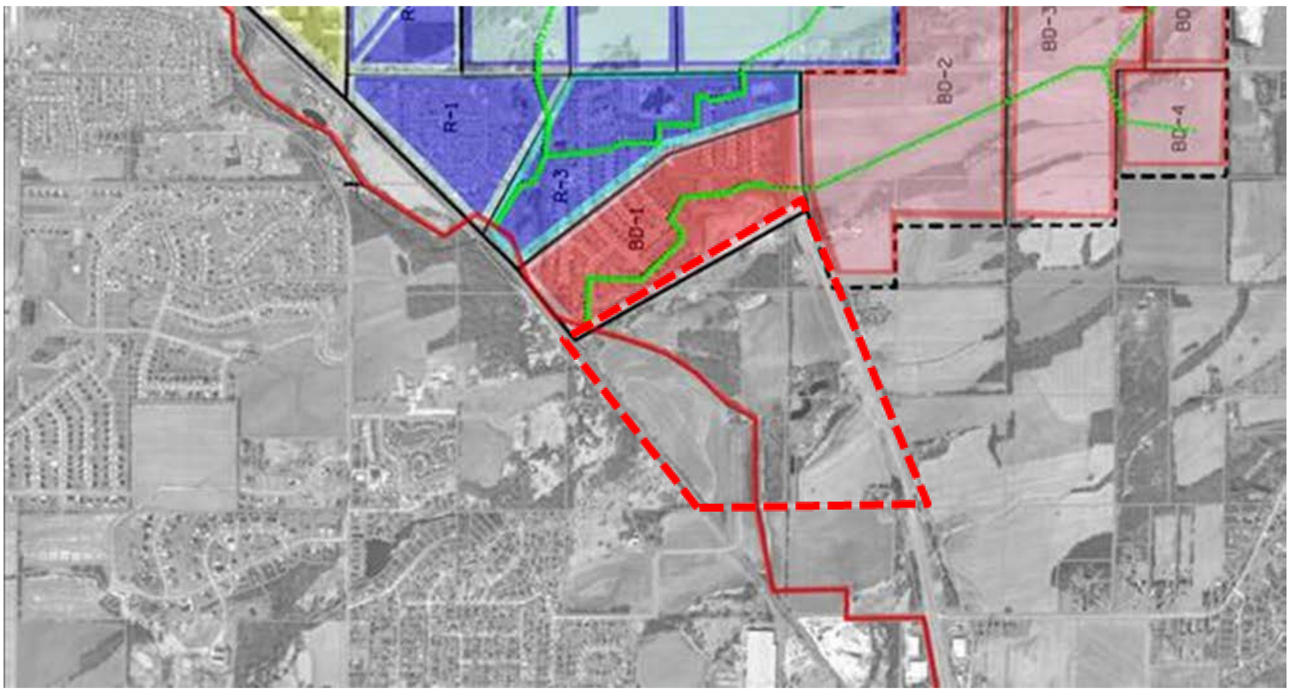


Figure 5.04-3 Kennedy Drive Property

This property would be served directly via the existing MMSD interceptor and would not impact existing Village interceptor sewers. Future development in this area (and any areas of development anticipated east of STH 113) should be studied separately in order to determine the appropriate land use and pipe capacity requirements.

D. Potential Development Area 4–Meffert Road Area

Represented by areas SS-4 and a portion of SS-6, these areas are serviceable via a recent extension of the South Side Interceptor along Water Wheel Drive. Improvements in these basins may require improvements to the Ashlawn Pumping Station and/or associated downstream sewers. The area is shown in Figure 5.04-4.

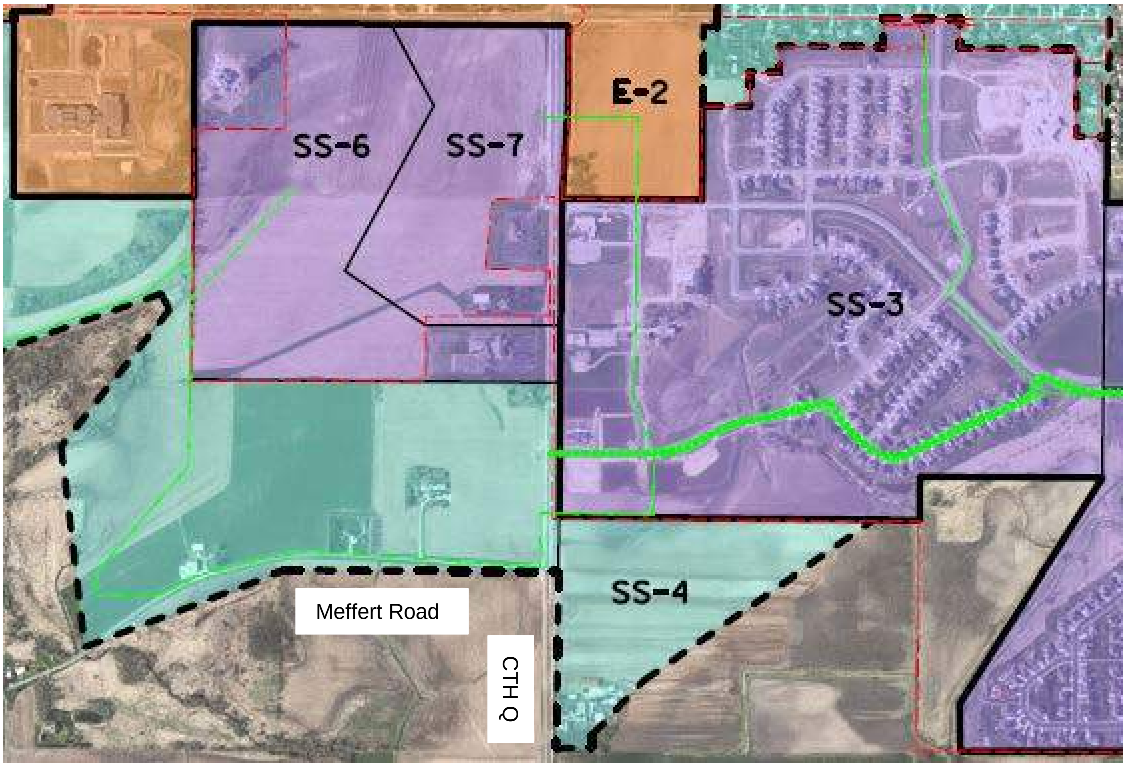


Figure 5.04-4 Meffert Road Area

5.05 FUTURE EVALUATION

Given the current pace of development within the Village and the variety of potential future development, it is recommended that thorough reviews and updates of this plan be completed every five years to address the needs of the system at that time and make any necessary adjustments to the plan.

5.06 CONCLUSIONS AND RECOMMENDATIONS

A. Northern Region

1. Conclusions

The Northwest and Northeast areas are restricted from future growth by downstream sewers in the Sixmile Creek Subdivision. The northerly portion of the Division Street Interceptor area is restricted from future growth mainly because of the high concentration of flows from one industry, Scientific Protein Labs.

The Westbridge neighborhood drains to the Westbridge Pumping Station which has a current pumping capacity of 300 gpm. The estimated peak flow from the development is 139 gpm. Existing available capacity is limited by the remaining pump capacity. Further development tributary to this pump station needs to consider the available capacity of the pump station and downstream sewers mentioned above.

2. Recommendations

a. Immediate

Monitor the Sixmile Creek Subdivision sewers for capacity issues with both Meadowbrook and Westbridge Pumping Stations operating. Control pumping rate of Westbridge pumping station by use of variable speed controls to temporarily reduce peak flow impacts.

b. Future and Ultimate

(1) Replace or install parallel sewers in the Sixmile Creek Subdivision to allow additional development north of Easy Street.

(2) Construct relief sewer between West Verleen Avenue and Sixmile Creek interceptor along North Century Avenue to free up capacity in that sewer for expansion into the northeast area.

(3) Construct private interceptor for Scientific Protein Labs, or increase pipe size downstream of Scientific Protein Labs to MMSD interceptor, to free up additional capacity in the Division Street sewer to serve lands north of STH 19.

(4) Increase capacity of Westbridge Pumping Station when needed to accommodate further expansion of lands serviced by this pumping station.

B. <u>Southwest Region</u>

1. Conclusions

The development of this region is limited to the current Ashlawn Pumping Station flow rate and by the existing sewer capacities in the Southbridge neighborhood. The trigger for upgrades to both has been considered to be the expansion of sewer to serve lands west of CTH Q once the Kilkenny Farms plats are fully occupied. That sewer extension occurred in 2016 along Water Wheel Drive, and another in 2018, midway between Water Wheel Drive and Peaceful Valley Parkway, indicating development there is imminent. Upgrades for the pumping station will likely include pump replacement. Upgrades to the downstream sewers will be more involved. Recently, Waunakee Utilities has agreed to take on the costs and implementation of the expansion of the pumping station and downstream sewer capacity, when it becomes necessary. Development agreements created for lands west of CTH Q will need to contain provisions that allow for capacity improvement charges to be assessed to those lands when the improvements are made.

2. Recommendations

a. Immediate

No improvements are needed for the existing service area.

b. Future and Ultimate

(1) Monitor pumping station flows and capacity of sewers along Tierney Drive and easement between Tierney Drive and Foggy Mountain Pass.

(2) Replace impellers and controls in Ashlawn Pumping Station pumps when expanding service area west of CTH Q.

(3) Increase the size of downstream sewers though Southbridge or add a parallel sewer to increase capacity for expanding development.

(4) Study alternative route for force main and gravity sewers to relieve the Southbridge sewer, possibly along Peaceful Valley Parkway and through Dane County Park lands.

(5) Monitor Endres interceptor sewer before expanding service area into the E-3 zone.

C. Southeast Region

1. Conclusions

Bongard Drive interceptor has the capacity to serve to its ultimate service area boundary. The Industrial Park interceptor is approaching its capacity with the addition of subbasin IP-3 and the Octopi Brewing point load, inhibiting its ability to serve lands north of STH 19. The Ravine Interceptor has the capacity to serve a significant amount of new development in that subbasin but not the entire subbasin.

2. Recommendations

a. Immediate

No improvements are needed for the existing service area.

b. Future and Ultimate

(1) Monitor Industrial Park interceptor upon full development of the Business Park before considering expansion of that sewer to serve lands north of STH 19 (IP-3).

(2) Consider a parallel sewer or replacement with larger sewers of the Ravine interceptor from Arboretum Drive to the MMSD interceptor for development that occurs in this region north of STH 19. Also, consider installing larger pipes for the next expansion in anticipation of meeting the ultimate service area.

APPENDIX A
EXISTING CAPACITY

Waunakee Utilities
Sewer System Capacity Analysis
Existing Capacity Calculations
09/27/18

Legend
Field Measured Data
Field/Record Data Transition
Record Data
Critical Capacity

Interceptor Name	Road Name	Sewer Line From MH	Sewer Line To MH	Pipe Size IN	Length LF	Capacity Upstream Invert	Capacity Downstream Invert	Slope %	Theoretical Capacity CFS	Theoretical Capacity GPM	Upstream Manhole Station LF	Upstream Manhole Invert EL	Upstream Manhole Crown EL
Bongard Drive	Easement	152057	152058	10	301	874.84	874.00	0.28%	1.16	521	+	874.84	875.67
	Hanover Place	152058	152059	10	264	873.97	873.23	0.28%	1.16	521	3+01	873.97	874.80
		152059	152061	10	261	873.20	872.47	0.28%	1.16	521	5+65	873.20	874.03
		152061	152062	10	225	872.44	871.81	0.28%	1.16	521	8+26	872.44	873.27
		152062	152071	10	190	871.78	871.25	0.28%	1.16	521	10+51	871.78	872.61
	Private Drive	152071	152078	10	98	871.22	870.95	0.28%	1.16	521	12+41	871.22	872.05
		152078	152027	10	312	870.92	870.05	0.28%	1.16	521	13+39	870.92	871.75
	Hanover Trail	152027	152028	10	148	870.00	869.59	0.28%	1.16	521	16+51	870.00	870.83
		152028	152029	10	105	869.55	869.26	0.28%	1.16	521	17+99	869.55	870.38
		152029	152030	10	74	869.22	869.01	0.28%	1.16	521	19+04	869.22	870.05
		152030	152031	10	273	868.97	868.21	0.28%	1.16	521	19+78	868.97	869.80
		152031	152032	10	297	868.17	867.34	0.28%	1.16	521	22+51	868.17	869.00
		152032	152033	10	128	867.30	866.94	0.28%	1.16	521	25+48	867.30	868.13
		152033	152034	10	300	866.90	866.06	0.28%	1.16	521	26+76	866.90	867.73
	Easement	152034	152047	10	323	866.02	865.12	0.28%	1.16	521	29+76	866.02	866.85
	Bongard Drive	152047	153002	10	275	865.02	863.59	0.52%	1.58	710	32+99	865.02	865.85
		153002	MMSD	10	343	863.52	855.56	2.32%	3.34	1,500	35+74	863.52	864.35
		MMSD		10		855.56					39+17	855.56	856.39
R-6 Extension	Montadon Avenue	152069	152068	12	214	889.53	888.76	0.36%	2.14	961	+	889.53	890.53
	Gile Drive	152068	152067	12	160	888.66	888.08	0.36%	2.14	961	2+14	888.66	889.66
		152067	152066	12	170	887.98	887.37	0.36%	2.14	961	3+74	887.98	888.98
		152066	152064	12	113	887.27	886.86	0.36%	2.14	961	5+44	887.27	888.27
		152064	152065	12	50	886.76	886.58	0.36%	2.14	961	6+57	886.76	887.76
	Easement	152065	152077	12	135	886.48	885.99	0.36%	2.14	961	7+07	886.48	887.48
		152077	152076	12	356	885.89	884.60	0.36%	2.14	961	8+42	885.89	886.89
		152076	152075	12	215	884.50	883.73	0.36%	2.14	961	11+98	884.50	885.50
		152075	152074	12	250	883.63	882.73	0.36%	2.14	961	14+13	883.63	884.63
		152074	152016	12	191	882.63	881.94	0.36%	2.14	961	16+63	882.63	883.63
	Dartmouth Drive	152016	152015	12	300	881.84	880.76	0.36%	2.14	961	18+54	881.84	882.84
		152015	152014	12	140	880.66	880.12	0.39%	2.23	1,001	21+54	880.66	881.66
		152014	152013	12	200	880.02	879.30	0.36%	2.14	961	22+94	880.02	881.02
		152013	152012	12	223	879.20	878.40	0.36%	2.14	961	24+94	879.20	880.20
		152012	152001	12	75	878.30	878.03	0.36%	2.14	961	27+17	878.30	879.30
	Easement	152001	161043	12	287	878.03	875.63	0.84%	3.27	1,468	27+92	878.03	879.03
		161043	161041	12	166	875.53	871.55	2.40%	5.52	2,478	30+79	875.53	876.53
		161041		12		875.53					32+45	875.53	876.53
Ravine	Easement	94002	94001	18	380	875.33	874.20	0.30%	5.75	2,581	+	875.33	876.83
		94001	161044	18	93	874.20	873.58	0.67%	8.60	3,860	3+80	874.20	875.70
		161044	161042	18	480	873.58	872.20	0.29%	5.66	2,541	4+73	873.58	875.08
		161042	161041	18	480	872.07	870.60	0.31%	5.85	2,626	9+53	872.07	873.57
		161041	161040	18	384	870.55	867.63	0.76%	9.16	4,112	14+33	870.55	872.05
		161040	161001	18	170	867.00	866.66	0.20%	4.70	2,110	18+17	867.00	868.50
		161001	R3	18	195	866.56	866.17	0.20%	4.70	2,110	19+87	866.56	868.06
		R3	R2	18	125	866.07	865.82	0.20%	4.70	2,110	21+82	866.07	867.57
		R2	R1	18	174	865.72	865.37	0.20%	4.70	2,110	23+07	865.72	867.22
		R1	MMSD	18	213	865.20	863.50	0.80%	9.40	4,220	24+81	865.20	866.70
		MMSD		18		863.50					26+94	863.50	865.00
Southside Extension	Water Wheel Drive	Y	X	15	252	881.00	880.44	0.22%	3.03	1360		881.00	882.25
		X	W	15	152	880.41	880.08	0.22%	3.03	1360		880.41	881.66
		W	V	15	89	880.05	879.85	0.22%	3.03	1360		880.05	881.30
		V	U	15	100	879.82	879.59	0.23%	3.10	1392		879.82	881.07
		U	S	15	289	879.56	878.93	0.22%	3.03	1360		879.56	880.81
		S	R	15	138	878.90	878.60	0.22%	3.03	1360		878.90	880.15
		R	Q	15	350	878.57	877.81	0.22%	3.03	1360		878.57	879.82
		Q	P	15	166	877.78	877.41	0.22%	3.03	1360		877.78	879.03
		P	O	15	140	877.38	877.07	0.22%	3.03	1360		877.38	878.63
	Shannon Way	O	N	15	152	877.04	876.71	0.22%	3.03	1360		877.04	878.29
		N	M	15	263	876.68	876.10	0.22%	3.03	1360		876.68	877.93
		M	L	15	74	876.07	875.91	0.22%	3.03	1360		876.07	877.32
		L	K	15	75	875.88	875.71	0.23%	3.10	1392		875.88	877.13
		K	J	15	75	875.68	875.51	0.23%	3.10	1392		875.68	876.93
		J	I	15	75	875.48	875.31	0.23%	3.10	1392		875.48	876.73
		I	H	15	75	875.28	875.11	0.23%	3.10	1392		875.28	876.53
		H	G	15	160	875.08	874.73	0.22%	3.03	1360		875.08	876.33
		G	F	15	226	874.70	874.20	0.22%	3.03	1360		874.70	875.95
		F	E	15	226	874.17	873.67	0.22%	3.03	1360		874.17	875.42
		E	D	15	289	873.64	873.01	0.22%	3.03	1360		873.64	874.89
		D	C	15	67	872.98	872.83	0.22%	3.03	1360		872.98	874.23
		C	B	15	165	872.80	872.44	0.22%	3.03	1360		872.80	874.05
		B	A	15	262	872.44	871.83	0.23%	3.10	1392		872.44	873.69
	Peaceful Valley Parkway	A	EX95	15	141	871.80	871.50	0.21%	2.96	1329		871.80	873.05
		174054	174053	12	200	872.29	871.73	0.28%	1.89	849	+	872.29	873.29
		174053	174053	12	175	871.63	871.13	0.29%	1.92	862	2+00	871.63	872.63
		174053	174051	12	260	871.03	870.30	0.28%	1.89	849	3+75	871.03	872.03
		174051	174037	12	230	870.20	869.50	0.30%	1.95	876	6+35	870.20	871.20
		174037	174038	12	103	869.40	868.99	0.40%	2.25	1,010	8+65	869.40	870.40
		174038		12		868.99					9+68	868.99	869.99
Southside	Shenandoah Drive	163005	163004	12	333	931.74	931.01	0.22%	1.67	750	+	931.74	932.74
	Easement	163004	163003	12	220	931.01	930.53	0.22%	1.67	750	3+33	931.01	932.01
		163003	163002	12	197	930.53	930.09	0.22%	1.67	750	5+53	930.53	931.53
		163002	163001	12	256	930.09	929.53	0.22%	1.67	750	7+50	930.09	931.09
		163001	162039	12	163	929.53	929.17	0.22%	1.67	750	10+06	929.53	930.53
	Tierney Drive	162039	162025	12	177	929.17	928.78	0.22%	1.67	750	11+69	929.17	930.17
		162025	162024	12	105	928.78	928.51	0.26%	1.82	817	13+46	928.78	929.78
		162024	162022	12	233	928.46	928.00	0.20%	1.59	714	14+51	928.46	929.46
		162022	162021	12	139	927.93	927.53	0.29%	1.92	862	16+84	927.93	928.93
		162021	162018	12	170	927.50	926.10	0.82%	3.23	1,450	18+23	927.50	928.50
		162018	162017	12	286	926.00	917.96	2.81%	5.97	2,680	19+93	926.00	927.00
	Savannah Way	162017	162016	12	390	917.86	916.30	0.40%	2.25	1,010	22+79	917.86	918.86
		162016	162013	12	270	916.30	915.22	0.40%	2.25	1,010	26+69	916.30	917.30
	Athens Court	162013	162011	12	70	915.15	914.87	0.40%	2.25	1,010	29+39	915.15	916.15
	Easement	162011	162009	12	167	914.80	914.13	0.40%	2.25	1,010	30+09	914.80	915.80
		162009	162007	12	90	914.10	899.60	16.11%	14.30	6,419	31+76	914.10	915.10
	Woodland Drive	162007	162005	12	96	899.50	896.43	3.20%	6.37	2,860	32+66	899.50	900.50

	Woodland Drive	162005	162003	12	122	896.43	892.50	3.22%	6.39	2,869		33+62	896.43	897.43
	Easement	162003	162001	12	74	892.40	890.22	2.95%	6.12	2,747		34+84	892.40	893.40
		162001	A	12	175	890.12	876.86	7.58%	9.81	4,404		35+58	890.12	891.12
		A	MMSD	12	180	876.52	874.00	1.40%	4.22	1,895		37+33	876.52	877.52
		MMSD		12		874.00						39+13	874.00	875.00
Industrial Park Extension	Hogan Road	91018	91017	8	395	945.25	943.67	0.40%	0.76	342		+	945.25	945.92
		91017	102003	8	367	943.57	942.09	0.40%	0.76	342		3+95	943.57	944.24
		102003	102002	8	326	941.99	940.64	0.41%	0.77	346		7+62	941.99	942.66
		102002	102001	8	340	940.54	939.24	0.38%	0.74	333		10+88	940.54	941.21
	Uniek Drive	102001	IP1	8	165	939.14	938.48	0.40%	0.76	342		14+28	939.14	939.81
		IP1	91006	8	131	938.38	937.86	0.40%	0.76	342		15+93	938.38	939.05
		91006	91005	8	340	937.76	936.40	0.40%	0.76	342		17+24	937.76	938.43
		91005	91004	8	71	936.31	936.03	0.39%	0.75	337		20+64	936.31	936.98
		91004	91003	10	69	935.93	935.65	0.41%	1.40	629		21+35	935.93	936.76
		91003	91002	10	292	935.55	934.38	0.40%	1.39	624		22+04	935.55	936.38
		91002	91001	10	295	934.28	933.10	0.40%	1.39	624		24+96	934.28	935.11
		91001	94026	10	345	933.00	931.62	0.40%	1.39	624		27+91	933.00	933.83
	Lillian Street	94026	91013	10	300	931.52	930.27	0.42%	1.42	638		31+36	931.52	932.35
		91013	91014	10	329	930.19	924.40	1.76%	2.91	1,307		34+36	930.19	931.02
		91014	91015	10	329	924.30	913.82	3.19%	3.91	1,755		37+65	924.30	925.13
		91015	91016	10	329	913.72	908.54	1.57%	2.75	1,235		40+94	913.72	914.55
		91016	92029	10	200	908.44	904.44	2.00%	3.10	1,392		44+23	908.44	909.27
		92029		10		904.44						46+23	904.44	905.27
Industrial Park	Lillian Street	92030	92029	8	183	906.09	904.44	0.90%	1.15	517		+	906.09	906.76
	Easement	92029	92031	10	222	904.34	901.53	1.27%	2.47	1,109		1+83	904.34	905.17
		92031	92009	10	222	901.34	898.99	1.06%	2.26	1,015		4+05	901.34	902.17
		92009	92008	10	400	898.89	896.92	0.49%	1.53	687		6+27	898.89	899.72
	Moravian Valley Drive	92008	92007	12	146	896.85	896.50	0.24%	1.75	786		10+27	896.85	897.85
	Easement	92007	92006	12	260	896.38	895.81	0.22%	1.67	750		11+73	896.38	897.38
		92006	92005	12	280	895.70	895.08	0.22%	1.67	750		14+33	895.70	896.70
	Raemisch Road	92005	92004	12	283	894.96	893.91	0.37%	2.17	974		17+13	894.96	895.96
	Easement	92004	92003	12	385	891.00	889.00	0.52%	2.57	1,154		19+96	891.00	892.00
		92003	92002	12	400	889.00	887.93	0.27%	1.85	831		23+81	889.00	890.00
		92002	92001	12	400	887.93	887.22	0.18%	1.51	678		27+81	887.93	888.93
		92001	94075	12	325	887.22	886.40	0.25%	1.78	799		31+81	887.22	888.22
		94075	94074	12	400	886.40	885.46	0.23%	1.71	768		35+06	886.40	887.40
		94074	MMSD	12	400	885.46	884.46	0.25%	1.78	799		39+06	885.46	886.46
		MMSD		12		884.46						43+06	884.46	885.46
Endres	Aldora Lane	73009	73008	8	333	939.93	938.40	0.46%	0.82	369		+	939.93	940.60
		73008	73007	8	237	938.39	937.44	0.40%	0.76	342		3+33	938.39	939.06
		73007	73006	8	86	937.43	937.10	0.38%	0.74	333		5+70	937.43	938.10
		73006	73001	8	152	937.10	936.03	0.70%	1.01	454		6+56	937.10	937.77
	Centennial Parkway	73001	74084	8	180	936.03	935.25	0.43%	0.79	355		8+08	936.03	936.70
		74084	74076	8	185	935.25	934.50	0.41%	0.77	346		9+88	935.25	935.92
		74076	74075	8	107	934.50	934.11	0.36%	0.73	328		11+73	934.50	935.17
		74075	74053	8	218	934.11	933.13	0.45%	0.81	364		12+80	934.11	934.78
		74053	74052	8	395	933.13	931.50	0.41%	0.77	346		14+98	933.13	933.80
		74052	74051	8	258	931.50	930.50	0.39%	0.75	337		18+93	931.50	932.17
		74051	Holiday	10	320	930.47	929.08	0.43%	1.44	647			930.47	931.30
		Holiday	74021	10	913	929.03	913.37	1.72%	2.87	1,289			929.03	929.86
	Centennial Parkway	74021	74020	10	182	913.37	912.97	0.22%	1.03	463		49+51	913.37	914.20
	S. Century Avenue	74020	74005	10	326	912.81	911.25	0.48%	1.52	683		51+33	912.81	913.64
		74005	74001	10	261	911.20	910.50	0.27%	1.14	512		54+59	911.20	912.03
	Endres	74001	83033	10	313	910.48	909.14	0.43%	1.44	647		57+20	910.48	911.31
		83033	Endres	10	163	908.92	908.01	0.56%	1.64	737		60+33	908.92	909.75
		Endres	83032	10	128	907.98	907.41	0.45%	1.47	660		61+96	907.98	908.81
		83032	83031	12	184	907.25	906.83	0.23%	1.71	768		63+24	907.25	908.25
	Easement	83031	83030	12	240	906.68	906.03	0.27%	1.85	831		65+08	906.68	907.68
		83030	83029	12	325	906.03	904.89	0.35%	2.11	948		67+48	906.03	907.03
		83029	83028	12	350	904.89	903.65	0.35%	2.11	948		70+73	904.89	905.89
		83028	82011	12	302	903.65	902.59	0.35%	2.11	948		74+23	903.65	904.65
		82011	82006	12	302	902.59	901.53	0.35%	2.11	948		77+25	902.59	903.59
		82006	82005	12	375	901.53	900.22	0.35%	2.11	948		80+27	901.53	902.53
		82005	82003	12	400	900.22	898.82	0.35%	2.11	948		84+02	900.22	901.22
		82003	82002	12	179	898.82	898.19	0.35%	2.11	948		88+02	898.82	899.82
		82002	MMSD	12	56	898.19	898.00	0.34%	2.08	934		89+81	898.19	899.19
		MMSD		12		898.00						90+37	898.00	899.00
Sixmile	Ripp Road	72002	72001	8	349	921.22	919.87	0.39%	0.75	337		+	921.22	921.89
		72001	71071	8	302	919.87	918.63	0.41%	0.77	346		3+49	919.87	920.54
	Dorn Drive	71071	71067	8	365	918.63	915.64	0.82%	1.09	490		6+51	918.63	919.30
		71067	71064	8	355	915.64	914.22	0.40%	0.76	342		10+16	915.64	916.31
		71064	71050	8	261	914.22	913.08	0.44%	0.80	360		13+71	914.22	914.89
		71050	64021	8	350	913.08	912.03	0.30%	0.66	297		16+32	913.08	913.75
	Main Street	64021	64020	10	340	912.03	911.01	0.30%	1.20	539		19+82	912.03	912.86
		64020	64019	10	340	911.01	909.99	0.30%	1.20	539		23+22	911.01	911.84
	Easement	64019	64018	10	300	909.99	909.09	0.30%	1.20	539		26+62	909.99	910.82
		64018	64017	10	240	909.09	908.37	0.30%	1.20	539		29+62	909.09	909.92
		64017	64016	10	230	908.37	907.38	0.43%	1.44	647		32+02	908.37	909.20
		64016	64015	10	200	907.38	907.08	0.15%	0.85	382		34+32	907.38	908.21
		64015	64010	10	225	907.08	906.41	0.30%	1.20	539		36+32	907.08	907.91
	Sunset Lane	64010	64009	10	200	906.41	905.81	0.30%	1.20	539		38+57	906.41	907.24
		64009	64003	10	240	905.81	905.38	0.18%	0.93	418		40+57	905.81	906.64
	Easement	64003	64002	10	256	905.13	904.59	0.21%	1.00	449		42+97	905.13	905.96
		64002	64001	15	600	904.59	903.27	0.22%	3.03	1,360		45+53	904.59	905.84
	Dane Interceptor	64001	53087	15	134	903.27	902.97	0.22%	3.03	1,360		51+53	903.27	904.52
	Easement	53087	53072	18	304	902.57	902.11	0.15%	4.07	1,827		52+87	902.57	904.07
		53072	53071	18	309	902.11	901.70	0.13%	3.79	1,702		55+91	902.11	903.61
	RR Crossing	53071	53088	18	373	901.70	901.00	0.19%	4.58	2,056		59+00	901.70	903.20
	Easement	53088	53070	18	400	901.00	900.40	0.15%	4.07	1,827		62+73	901.00	902.50
		53070	53069	18	278	900.40	899.99	0.15%	4.07	1,827		66+73	900.40	901.90
		53069	53068	18	500	899.99	899.23	0.15%	4.07	1,827		69+51	899.99	901.49
		53068	53021	18	231	899.23	898.88	0.15%	4.07	1,827		74+51	899.23	900.73
	Northwest/Northeast Interceptor	53021	54045	21	310	898.68	898.31	0.12%	5.49	2,465		76+82	898.68	900.43
	Madison Street Crossing	54045	54044	21	59	898.31	898.24	0.12%	5.49	2,465		79+92	898.31	900.06
	Easement	54044	54043	21	200	898.24	897.93	0.16%	6.34	2,846		80+51	898.24	899.99
		54043	54042	21	284	897.93	897.59	0.12%	5.49	2,465		82+51	897.93	899.68
		54042	54041	21	342	897.59	897.18	0.12%	5.49	2,465		85+35	897.59	899.34
		54041	54009	21	510	897.18	896.57	0.12%	5.49	2,465		88+77	897.18	898.93
	Main Street Crossing	54009	81037	21	226	896.57	896.30	0.12%	5.49	2,465		93+87	896.57	898.32
	Easement	81037	81036	21	395	896.30	893.69	0.66%	12.87	5,777		96+13	896.30	898.05
		81036	81029	21	500	893.69	892.19	0.30%	8.68	3,896		100+08	893.69	895.44
		81029	81017	21	257	891.99	891.22	0.30%	8.68	3,896		105+08	891.99	893.74

	Division Street Interceptor	81017	81016	24	251	891.02	890.27	0.30%	12.39	5,562		107+65	891.02	893.02
	Easement	81016	81013	24	272	890.27	889.46	0.30%	12.39	5,562		110+16	890.27	892.27
	RR Crossing	81013	81045	24	160	889.46	889.00	0.29%	12.18	5,467		112+88	889.46	891.46
	Easement	81045		24		889.00						114+48	889.00	891.00
Division Street	Nord Drive	92028	92027	10	300	928.08	918.40	3.23%	3.94	1,769		+	928.08	928.91
		92027	92026	10	400	918.40	907.00	2.85%	3.70	1,661		3+00	918.40	919.23
	Easement	92026	92025	10	175	907.00	906.05	0.54%	1.61	723		7+00	907.00	907.83
		92025	81026	10	360	906.05	903.00	0.85%	2.02	907		8+75	906.05	906.88
		81026	81025	10	400	903.00	900.10	0.72%	1.86	835		12+35	903.00	903.83
		81025	81022	10	67	900.10	899.48	0.93%	2.11	948		16+35	900.10	900.93
	Marshall Drive	81022	81021	10	320	899.48	898.39	0.34%	1.28	575		17+02	899.48	900.31
	Easement	81021	81020	10	310	898.39	897.25	0.37%	1.33	597		20+22	898.39	899.22
		81020	81019	12	113	896.70	896.14	0.50%	2.52	1,132		23+32	896.70	897.70
		81019	81018	12	130	896.14	895.20	0.72%	3.02	1,356		24+45	896.14	897.14
		81018	81017	12	86	895.20	892.00	3.72%	6.87	3,084		25+75	895.20	896.20
	Sixmile Interceptor Connection	81017		12		892.00						26+61	892.00	893.00
Northwest Extension	Countryside Crossing	61045	61044	8	61	922.14	921.83	0.51%	0.86	386		+	922.14	922.81
		61044	61043	8	192	921.76	920.91	0.44%	0.80	360		+61	921.76	922.43
		61043	61042	8	259	920.91	919.77	0.44%	0.80	360		2+53	920.91	921.58
		61042	61016	8	379	919.69	918.09	0.42%	0.78	351		5+12	919.69	920.36
		61016	61015	10	171	912.59	912.11	0.28%	1.16	521		8+91	912.59	913.42
		61015	LS	10	169	912.11	911.64	0.28%	1.16	521		10+62	912.11	912.94
		LS		10		911.64						12+31	911.64	912.47
Northwest	Kopp Road	61013	61012	12	247	922.90	922.36	0.22%	1.67	750		+	922.90	923.90
		61012	61010	12	295	922.31	921.66	0.22%	1.67	750		2+47	922.31	923.31
	Vanderbilt Drive	61010	61009	12	122	921.61	921.34	0.22%	1.67	750		5+42	921.61	922.61
		61009	61008	12	305	921.29	920.69	0.20%	1.59	714		6+64	921.29	922.29
		61008	61007	12	274	920.69	920.14	0.20%	1.59	714		9+69	920.69	921.69
		61007	61006	12	400	920.14	919.34	0.20%	1.59	714		12+43	920.14	921.14
		61006	61001	12	132	919.34	919.08	0.20%	1.59	714		16+43	919.34	920.34
	Easement	61001	52031	12	265	919.08	918.54	0.20%	1.59	714		17+75	919.08	920.08
	Legnds Drive	52031	52026	12	300	918.54	917.94	0.20%	1.59	714		20+40	918.54	919.54
	Lochmoor Drive	52026	52024	12	335	917.92	917.25	0.20%	1.59	714		23+40	917.92	918.92
		52024	52023	12	248	917.25	916.75	0.20%	1.59	714		26+75	917.25	918.25
		52023	52022	12	97	916.75	916.56	0.20%	1.59	714		29+23	916.75	917.75
		52022	52021	12	170	916.56	916.22	0.20%	1.59	714		30+20	916.56	917.56
	N. Century Avenue	52021	NW1	12	180	916.22	915.86	0.20%	1.59	714		31+90	916.22	917.22
		NW1	53089	12	365	915.86	915.12	0.20%	1.59	714		33+70	915.86	916.86
		53089	53067	12	250	914.12	913.62	0.20%	1.59	714		37+35	914.12	915.12
	Verleen Street	53067	53066	12	170	913.62	913.27	0.21%	1.63	732		39+85	913.62	914.62
		53066	53064	12	190	913.27	912.89	0.20%	1.59	714		41+55	913.27	914.27
	Northeast Interceptor	53064	53048	12	276	912.89	912.34	0.20%	1.59	714		43+45	912.89	913.89
	Fairbrook Drive	53048	53047	12	200	912.34	911.94	0.20%	1.59	714		46+21	912.34	913.34
		53047	53046	12	248	911.94	911.44	0.20%	1.59	714		48+21	911.94	912.94
		53046	53044	12	260	911.44	910.78	0.25%	1.78	799		50+69	911.44	912.44
		53044	53043	12	224	910.78	910.50	0.12%	1.23	553		53+29	910.78	911.78
		53043	53041	12	165	910.50	910.29	0.13%	1.28	575		55+53	910.50	911.50
		53041	53034	12	309	910.29	909.63	0.21%	1.63	732		57+18	910.29	911.29
		53034	53031	12	308	909.63	908.94	0.22%	1.67	750		60+27	909.63	910.63
		53031	53023	12	299	908.94	908.28	0.22%	1.67	750		63+35	908.94	909.94
	Easeement	53023	53022	12	278	908.29	907.67	0.22%	1.67	750		66+34	908.29	909.29
		53022	53021	12	52	907.67	904.30	6.48%	9.07	4,071		69+12	907.67	908.67
	Sixmile Interceptor	53021		12		904.30						69+64	904.30	905.30
Northridge Extension	Stone Edge Court	51056	51055	8	55	933.89	933.67	0.40%	0.76	342		+	933.89	934.56
		51055	51054	8	84	933.57	933.23	0.40%	0.76	342		+55	933.57	934.24
		51054	51053	8	201	933.13	932.32	0.40%	0.76	342		1+39	933.13	933.80
		51053	51052	8	342	932.22	930.85	0.40%	0.76	342		3+40	932.22	932.89
	Easement	51052	51061	8	256	930.75	929.72	0.40%	0.76	342		6+82	930.75	931.42
		51061	51071	8	169	929.72	927.29	1.44%	1.45	651		9+38	929.72	930.39
	Skyview Drive	51071	51070	8	210	927.23	926.31	0.44%	0.80	360		11+07	927.23	927.90
	Badger Lane	51070	51042	8	216	926.31	925.34	0.45%	0.81	364		13+17	926.31	926.98
	Easement	51042	51041	8	231	925.34	923.95	0.60%	0.94	422		15+33	925.34	926.01
		51041	51037	8	332	923.92	922.62	0.39%	0.75	337		17+64	923.92	924.59
	Madison Street	51037	51036	8	100	922.62	922.44	0.18%	0.51	229		20+96	922.62	923.29
		51036	52019	8	150	922.44	921.84	0.40%	0.76	342		21+96	922.44	923.11
	Greenbriar Drive	52019	52017	8	299	921.84	920.63	0.40%	0.76	342		23+46	921.84	922.51
		52017	52016	8	108	920.63	920.20	0.40%	0.76	342		26+45	920.63	921.30
		52016	52013	8	230	920.20	919.28	0.40%	0.76	342		27+53	920.20	920.87
		52013	52012	8	198	919.28	918.49	0.40%	0.76	342		29+83	919.28	919.95
		52012	52006	8	145	918.49	917.91	0.40%	0.76	342		31+81	918.49	919.16
		52006	52005	8	335	917.91	916.60	0.39%	0.75	337		33+26	917.91	918.58
		52005		8		916.60						36+61	916.60	917.27
Northeast	N. Fairbrook Drive	52005	52003	12	313	916.60	915.91	0.22%	1.67	750		+	916.60	917.60
		52003	52002	12	378	915.91	915.08	0.22%	1.67	750		3+13	915.91	916.91
		52002	52001	12	129	915.08	914.80	0.22%	1.67	750		6+91	915.08	916.08
		52001	53060	12	195	914.80	914.37	0.22%	1.67	750		8+20	914.80	915.80
		53060	53053	12	316	914.37	913.68	0.22%	1.67	750		10+15	914.37	915.37
		53053	53050	12	340	913.68	912.96	0.21%	1.63	732		13+31	913.68	914.68
		53050	53048	12	312	912.96	912.37	0.19%	1.55	696		16+71	912.96	913.96
	Northwest Interceptor	53048		12		912.37						19+83	912.37	913.37

APPENDIX B
EXISTING FLOWS

Waunakee Utilities
Sewer System Capacity Analysis
Developed Area Flow Calculations
09/27/18

	Contributing Area Classification								Point Source						Total Peak Flow				
	Residential				Commercial		Industrial		Contribution										
	Area	Dwelling	Equivalent	Flow	Area	Flow	Area	Flow	Flow	Total Area	Total Flow	Peaking	Base Peak Flow	Peak I/I					
Basin ID	**Ac**	**Units**	**Population**	**GPD**	**Ac**	**GPD**	**Ac**	**GPD**	**GPD**	**Ac**	**GPD**	**Factor**	**GPD**	**GPD**	**GPD**	**GPM**	**GPM/Ac**	**CFS**	**CFS/Ac**
BD-1	101.0	192	517	41,360	0.0	0	0.0	0	0	101.0	41,360	2.50	103,400	34,946	138,346	97	1.0	0.22	0.00218
R-1	55.0	110	296	23,680	0.0	0	0.0	0	0	55.0	23,680	2.50	59,200	19,030	78,230	55	1.0	0.13	0.00237
R-2	2.0	4	11	880	15.0	18,000	0.0	0	14,575	17.0	33,455	2.50	83,638	5,882	89,520	63	3.7	0.15	0.00883
R-3	84.0	132	356	28,480	18.0	21,600	0.0	0	0	102.0	50,080	2.50	125,200	35,292	160,492	112	1.1	0.25	0.00246
R-4	80.0	113	304	24,320	0.0	0	0.0	0	0	80.0	24,320	2.50	60,800	27,680	88,480	62	0.8	0.14	0.00175
SS-1	146.0	281	756	60,480	0.0	0	0.0	0	0	146.0	60,480	2.50	151,200	50,516	201,716	141	1.0	0.32	0.00220
SS-2	153.0	234	630	50,400	0.0	0	0.0	0	0	153.0	50,400	2.50	126,000	52,938	178,938	125	0.8	0.28	0.00184
SS-3	270.0	383	1,031	82,480	43.0	51,600	0.0	0	3,500	313.0	137,580	2.50	343,950	108,298	452,248	315	1.0	0.71	0.00227
SS-6	69.6	116	313	25,040	0.0	0	0.0	0	0	69.6	25,040	2.50	62,600	24,079	86,679	61	0.9	0.14	0.00202
SS-7	47.1	493	986	78,880	16.8	20,184	0.0	0	0	63.9	99,064	2.50	247,660	22,117	269,777	188	2.9	0.42	0.00658
IP-1	0.0	0	0	0	3.5	4,200	200.0	300,000	0	203.5	304,200	2.50	760,500	70,411	830,911	578	2.8	1.29	0.00634
IP-2	0.0	0	0	0	0.0	0	50.0	75,000	9,300	50.0	84,300	2.50	210,750	17,300	228,050	159	3.2	0.36	0.00720
IP-3		0	0	0	0.0	0	0.0	0	0	0.0	0	2.50	0	0	0	0	0.0	0.00	0.00000
E-1	111.0	325	875	70,000	3.0	3,600	0.0	0	0	114.0	73,600	2.50	184,000	39,444	223,444	156	1.4	0.35	0.00308
E-2	205.0	422	1,636	130,880	30.4	36,480	20.0	30,000	50,225	255.4	247,585	2.50	618,963	88,369	707,332	492	1.9	1.10	0.00431
DS-1	3.0	16	44	3,520	2.0	2,400	70.0	105,000	216,000	75.0	326,920	2.50	817,300	25,950	843,250	586	7.8	1.31	0.01747
SM-1	130.0	250.0	1,023	81,840	37.8	45,301	0.0	0	30,825	167.8	157,966	2.50	394,915	58,042	452,957	315	1.9	0.71	0.00424
SM-2	152.0	288.0	775	62,000	0.0	0	0.0	0	12,775	152.0	74,775	2.50	186,938	52,592	239,530	167	1.1	0.38	0.00250
SM-3	120.0	212.0	571	45,680	32.0	38,400	0.0	0	3,500	152.0	87,580	2.50	218,950	52,592	271,542	189	1.2	0.43	0.00283
SM-4	120.0	237.0	638	51,040	0.0	0	0.0	0	0	120.0	51,040	2.50	127,600	41,520	169,120	118	1.0	0.27	0.00225
NE-1	55.9	144.0	388	31,040	0.0	0	0.0	0	0	55.9	31,040	2.50	77,600	19,342	96,942	68	1.2	0.16	0.00287
NE-2	130.0	275.0	740	59,200	0.0	0	0.0	0	0	130.0	59,200	2.50	148,000	44,980	192,980	135	1.0	0.31	0.00239
NE-3	44.6	94.0	253	20,240	0.0	0	0.0	0	0	44.6	20,240	2.50	50,600	15,432	66,032	46	1.0	0.11	0.00247
NW-1	52.0	168.0	452	36,160	0.0	0	0.0	0	0	52.0	36,160	2.50	90,400	17,992	108,392	76	1.5	0.17	0.00327
NW-2	70.0	90.0	243	19,440	0.0	0	0.0	0	0	70.0	19,440	2.50	48,600	24,220	72,820	51	0.7	0.12	0.00172
NW-10	135.0	283.0	762	60,960	0.0	0	0.0	0	0	135.0	60,960	2.50	152,400	46,710	199,110	139	1.0	0.31	0.00230
MMSD-1	126.0	424.0	1,141	91,280	0.0	0	1.0	1,500	14,100	127.0	106,880	2.50	267,200	43,942	311,142	217	1.7	0.49	0.00386
MMSD-2	132.0	272.0	732	58,560	0.0	0	0.0	0	0	132.0	58,560	2.50	146,400	45,672	192,072	134	1.0	0.30	0.00228
		1,482.0																	
Subtotals	2,594.2	5,558.0	15,473.0	1,237,840.0	201.5	241,765.0	341.0	511,500.0	354,800.0	3,136.7	2,345,905.0			1,085,288.0					

Point Source Contributions

Basin ID	Facility Name	Total	Unit	Flow GPD
R-2	Arboretum Elementary School	511.0	Students	12,775.0
E-2	Waunakee Manor	104.0	Beds	5,200.0
E-2	Waunakee High School	1,101.0	Students	27,525.0
MMSD-1	Waunakee Middle School	564.0	Students	14,100.0
DS-1	Scientific Protein Labs (SPL)			216,000.0
SM-1	Cannery Row Senior Living Center	131.0	Beds	6,550.0
SM-1	St. John the Baptist School	100.0	Students	2,500.0
SM-1	Heritage Elementary School	311.0	Students	7,775.0
SM-1	Intermediate School	560.0	Students	14,000.0
SM-2	Prairie Elementary School	511.0	Students	12,775.0
R-2	Brightstar Senior Living Center	36.0	Beds	1,800.0
SS-3	At Home Again Senior Living	70.0	Beds	3,500.0
E-2	Woodland School	700.0	Students	17,500.0
IP-2	Octopi Brewing			9,300.0

Redevelopment

Basin	Facility Name	R-5	Persons/DU	Equivalent Pop. Persons	Commercial SF
SM-1	Madison/Main	74	2.0	148	3,700
SM-1	Lone Girl	0	0	0	17,000
SM-1	Lamphouse	101	2.0	202	12,000
				350	32,700

Woodland Crest

Basin	Facility Name	R-5	Persons/DU	Equivalent Pop. Persons	Commercial, AC
E-2	Residential	150	2.0	300	
E-2	Residential	100	1	200	
E-2	Commercial				30
				500	30

APPENDIX C
FUTURE FLOWS AND CALCULATIONS

Waunakee Utilities
Sewer System Capacity Analysis
Undeveloped Area Flow Calculations
09/27/18

Undeveloped Land - Zoning Breakdown (Ac) and Loading Factor (GPM/Ac)																				
Total Peak Wastewater Contribution, < 250 Acres			1.42			1.58			2.23			5.57			2.10			3.50		
Total Peak Wastewater Contribution, 250-500 Acres			1.24			1.39			1.96			4.89			2.10			3.50		
Total Peak Wastewater Contribution, > 500 Acres			0.90			1.00			1.42			3.53			2.10			3.50		

		Total	Environm	R-1			R-2			R-3			R-5			C-1			I-1			Total Basin Peak Flow	
		Area	ental		Area	Flow		Area	Flow		Area	Flow		Area	Flow		Area	Flow		Area	Flow		
Basin ID	Downstream Interceptor	Ac	%	% Area	Ac	GPM	% Area	Ac	GPM	% Area	Ac	GPM	% Area	Ac	GPM	% Area	Ac	GPM	% Area	Ac	GPM	GPM	GPD
BD-2	Bongard Drive	160	0%	60%	96	87		0	0		0	0		0	0	40%	64	135		0	0	222	319,680
BD-3	Bongard Drive	110	0%	100%	110	99		0	0		0	0		0	0		0	0		0	0	99	142,560
BD-4	Bongard Drive	40	0%	100%	40	36		0	0		0	0		0	0		0	0		0	0	36	51,840
BD-5	Bongard Drive	50	0%	100%	50	45		0	0		0	0		0	0		0	0		0	0	45	64,800
R-2	Ravine	35	0%	0%	0	0		0	0		0	0		0	0	100%	35	74		0	0	74	106,560
~~R-4~~	~~Ravine~~	~~80~~	~~0%~~	~~50%~~	~~40~~	~~36~~		~~0~~	~~0~~		~~0~~	~~0~~		~~0~~	~~0~~	~~50%~~	~~40~~	~~84~~		~~0~~	~~0~~		~~0~~
R-5	Ravine	120	0%	80%	96	87		0	0		0	0		0	0	20%	24	51		0	0	138	198,720
R-6	Ravine	165	0%	25%	41	38		0	0		0	0		0	0	50%	83	174	25%	41	150	362	521,280
R-7	Ravine	165	0%	0%	0	0		0	0		0	0		0	0	50%	83	174	50%	83	290	464	668,160
R-8	Ravine	170	0%	0%	0	0		0	0		0	0		0	0	50%	85	179	50%	85	300	479	689,760
R-9	Ravine	190	0%	50%	95	86		0	0		0	0	25%	48	168	25%	48	100		0	0	354	509,760
R-10	Ravine	175	0%	50%	88	79		0	0		0	0	25%	44	155	25%	44	92		0	0	326	469,440
R-11	Ravine	175	0%	100%	175	158		0	0		0	0		0	0		0	0		0	0	158	227,520
R-12	Ravine	100	0%	100%	100	90		0	0		0	0		0	0		0	0		0	0	90	129,600
R-13	Ravine	240	0%	100%	240	216		0	0		0	0		0	0		0	0		0	0	216	311,040
R-14	Ravine	190	0%	100%	190	171		0	0		0	0		0	0		0	0		0	0	171	246,240
R-15	Ravine	185	0%	100%	185	167		0	0		0	0		0	0		0	0		0	0	167	240,480
~~SS-3~~	~~Southside~~	~~270~~	~~15%~~	~~65%~~	~~176~~	~~158~~		~~0~~	~~0~~		~~0~~	~~0~~		~~0~~	~~0~~	~~20%~~	~~54~~	~~114~~		~~0~~	~~0~~		~~0~~
SS-4	Southside	40	0%	50%	20	18		0	0		0	0		0	0	50%	20	42		0	0	60	86,400
SS-5	Southside	38	0%	100%	38	35		0	0		0	0		0	0		0	0		0	0	35	50,400
SS-6	Southside	156	0%	80%	125	113		0	0		0	0		0	0	20%	31	66		0	0	179	257,760
~~SS-7~~	~~Southside~~	~~83~~	~~0%~~	~~80%~~	~~66~~	~~60~~		~~0~~	~~0~~		~~0~~	~~0~~		~~0~~	~~0~~	~~20%~~	~~17~~	~~35~~		~~0~~	~~0~~		~~0~~
SS-8	Southside	110	0%	80%	88	80		0	0		0	0		0	0	20%	22	47		0	0	127	182,880
SS-9	Southside	80	0%	80%	64	58		0	0		0	0		0	0	20%	16	34		0	0	92	132,480
SS-10	Southside	180	0%	80%	144	130		0	0		0	0		0	0	20%	36	76		0	0	206	296,640
SS-11	Southside	190	0%	80%	152	137		0	0		0	0		0	0	20%	38	80		0	0	217	312,480
IP-2	Industrial Park	110	0%	0%	0	0		0	0		0	0		0	0		0	0	100%	110	390	390	561,600
IP-3	Industrial Park	50	0%	0%	0	0		0	0		0	0		0	0		0	0	100%	50	180	180	259,200
~~E-2~~	~~Endres~~	~~14~~	~~0%~~	~~50%~~	~~7~~	~~7~~		~~0~~	~~0~~		~~0~~	~~0~~		~~0~~	~~0~~	~~50%~~	~~7~~	~~15~~		~~0~~	~~0~~		~~0~~
E-3	Endres	85	0%	80%	68	62	20%	17	17		0	0		0	0		0	0		0	0	79	113,760
SM-5	Sixmile	100	0%	100%	100	90		0	0		0	0		0	0		0	0		0	0	90	129,600
SM-6	Sixmile	20	0%	0%	0	0		0	0		0	0		0	0	100%	20	42		0	0	42	60,480
SM-7	Sixmile	80	60%	40%	32	29		0	0		0	0		0	0		0	0		0	0	29	41,760
SM-8	Sixmile	60	0%	80%	48	44		0	0		0	0		0	0	20%	12	26		0	0	70	100,800
SM-9	Sixmile	80	0%	80%	64	58		0	0		0	0		0	0	20%	16	34		0	0	92	132,480
SM-10	Sixmile	75	0%	80%	60	54		0	0		0	0		0	0	20%	15	32		0	0	86	123,840
SM-11	Sixmile	80	0%	100%	80	72		0	0		0	0		0	0		0	0		0	0	72	103,680
DS-2	Division Street	160	0%	60%	96	87	20%	32	32		0	0	20%	32	113		0	0		0	0	232	334,080
DS-3	Division Street	150	25%	50%	75	68	15%	23	23		0	0	10%	15	53		0	0		0	0	144	207,360
DS-4	Division Street	130	0%	80%	104	94	20%	26	26		0	0		0	0		0	0		0	0	120	172,800
DS-5	Division Street	120	0%	80%	96	87	20%	24	24		0	0		0	0		0	0		0	0	111	159,840
DS-6	Division Street	240	0%	80%	192	173	20%	48	48		0	0		0	0		0	0		0	0	221	318,240
DS-7	Division Street	270	0%	80%	216	195	20%	54	54		0	0		0	0		0	0		0	0	249	358,560
NW-3	Northwest	80	0%	100%	80	72		0	0		0	0		0	0		0	0		0	0	72	103,680
NW-4	Northwest	155	15%	85%	132	119		0	0		0	0		0	0		0	0		0	0	119	171,360
NW-5	Northwest	110	0%	80%	88	80	20%	22	22		0	0		0	0		0	0		0	0	102	146,880
NW-6	Northwest	190	0%	80%	152	137	20%	38	38		0	0		0	0		0	0		0	0	175	252,000
NW-7	Northwest	230	0%	80%	184	166	20%	46	46		0	0		0	0		0	0		0	0	212	305,280
NW-8	Northwest	200	0%	80%	160	144	20%	40	40		0	0		0	0		0	0		0	0	184	264,960
NW-9	Northwest	60	0%	100%	60	54		0	0		0	0		0	0		0	0		0	0	54	77,760
~~NW-10~~	~~Northwest~~	~~135~~	~~0%~~	~~100%~~	~~135~~	~~122~~		~~0~~	~~0~~		~~0~~	~~0~~		~~0~~	~~0~~		~~0~~	~~0~~		~~0~~	~~0~~		~~0~~
NW-11	Northwest	200	0%	100%	200	180		0	0		0	0		0	0		0	0		0	0	180	259,200
NW-12	Northwest	140	15%	85%	119	108		0	0		0	0		0	0		0	0		0	0	108	155,520
NW-13	Northwest	75	0%	100%	75	68		0	0		0	0		0	0		0	0		0	0	68	97,920
NW-14	Northwest	230	0%	100%	230	207		0	0		0	0		0	0		0	0		0	0	207	298,080
NW-15	Northwest	135	15%	85%	115	104		0	0		0	0		0	0		0	0		0	0	104	149,760
NW-16	Northwest	170	20%	80%	136	123		0	0		0	0		0	0		0	0		0	0	123	177,120
NE-3	Northeast	125	0%	80%	100	90	20%	25	25		0	0		0	0		0	0		0	0	115	165,600
NE-4	Northeast	270	0%	80%	216	195	20%	54	54		0	0		0	0		0	0		0	0	249	358,560
NE-5	Northeast	120	0%	80%	96	87	20%	24	24		0	0		0	0		0	0		0	0	111	159,840

Waunakee Utilities
Sewer System Capacity Analysis
Theoretical Flow Calculations
09/27/18

Land Use Classification		**R-1**	**R-2**	**R-3**	**R-5**	**C-1**	**I-1**
Minimum Lot Area per Dwelling Unit	SF	9500.00	8500.00	6000.00	2400.00		
Public Use Area per Acre (100%)	SF	9500.00	8500.00	6000.00	2400.00		
Dwelling Unit per Gross Acre		2.30	2.57	3.63	9.08		
Population Equivilant per Dwelling Unit		2.69	2.69	2.69	2.69		
Population Equivilant per Gross Acre		6.19	6.92	9.77	24.43		
Wastewater Contribution	GCD	80.00	80.00	80.00	80.00		
Wastewater Contribution per Gross Acre	GPD/Ac	495.20	553.60	781.60	1,954.40	1,200.00	2,000.00
Peaking Factor / Gross Acre, < 250 Acres		4.00	4.00	4.00	4.00	2.50	2.50
Peaking Factor / Gross Acre, 251-500 Acres		3.50	3.50	3.50	3.50	2.50	2.50
Peaking Factor / Gross Acre, > 500 Acres		2.50	2.50	2.50	2.50	2.50	2.50
Peak Wastewater Contribution, < 250 Acres	GPD/Ac	1,980.80	2,214.40	3,126.40	7,817.60	3,000.00	5,000.00
Peak Wastewater Contribution, 250-500 Acres	GPD/Ac	1,733.20	1,937.60	2,735.60	6,840.40	3,000.00	5,000.00
Peak Wastewater Contribution, > 500 Acres	GPD/Ac	1,238.00	1,384.00	1,954.00	4,886.00	3,000.00	5,000.00
Minimum Frontage per Dwelling Unit	LF	90.00	70.00	80.00	80.00		
Sewer Main Length per Gross Acre	LF/Ac	103.50	89.95	145.20	363.20		
Typical Sewer Diameter	IN	8.00	8.00	8.00	8.00		
Sewer Lateral Length per Gross Acre (60 LF per DU)	LF/Ac	138.00	154.20	217.80	544.80		
Typical Lateral Diameter	IN	4.00	4.00	4.00	4.00		
Equivilant Sewer Length per Gross Acre (8" main, 4" lateral)	IN-Mile/Ac	0.27	0.26	0.39	0.97		
Infiltration Rate	GPD/IN-Mile-Ac	200.00	200.00	200.00	200.00		
Infiltration Contribution	GPD/Ac	50.00	50.00	80.00	190.00	24.00	40.00
Total Peak Wastewater Contribution, < 250 Acres	GPD/Ac	2,030.80	2,264.40	3,206.40	8,007.60	3,024.00	5,040.00
Total Peak Wastewater Contribution, 250-500 Acres	GPD/Ac	1,783.20	1,987.60	2,815.60	7,030.40	3,024.00	5,040.00
Total Peak Wastewater Contribution, > 500 Acres	GPD/Ac	1,288.00	1,434.00	2,034.00	5,076.00	3,024.00	5,040.00
Total Peak Wastewater Contribution, < 250 Acres	GPM/Ac	1.42	1.58	2.23	5.57	2.10	3.50
Total Peak Wastewater Contribution, 250-500 Acres	GPM/Ac	1.24	1.39	1.96	4.89	2.10	3.50
Total Peak Wastewater Contribution, > 500 Acres	GPM/Ac	0.90	1.00	1.42	3.53	2.10	3.50
Total Peak Wastewater Contribution, < 250 Acres	CFS/Ac/D	0.00315	0.00351	0.00497	0.01239	0.00468	0.00780
Total Peak Wastewater Contribution, 250-500 Acres	CFS/Ac/D	0.00276	0.00308	0.00436	0.01088	0.00468	0.00780
Total Peak Wastewater Contribution, > 500 Acres	CFS/Ac/D	0.00200	0.00222	0.00315	0.00786	0.00468	0.00780

STRAND
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Madison, WI 53715
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www.strand.com

July 31, 2024

Mr. Tim Herlitzka, General Manager
Village of Waunakee
322 Moravian Valley Road
Waunakee, WI 53597

Re: 2024 Flow Monitoring Program
Village of Waunakee, Wisconsin (Village)

Dear Tim,

This letter summarizes the results and recommendations regarding the 2024 Flow Monitoring Program.

Introduction

In spring 2024, the Village initiated a flow metering study (Study). The Village is located in Dane County in south-central Wisconsin, and it has a population of approximately 15,150 as of the July 2023 United States Census Bureau Estimate. Enclosed Figure 1 illustrates the Village's existing sanitary sewer system and sewer service area (SSA). The existing sanitary sewer system consists of approximately 330,000 linear feet (LF) of gravity sanitary sewer ranging in size from 8 to 15 inches in diameter. There are three sanitary sewer pumping stations in the Village's SSA. The locations of the pumping stations and force mains are shown in Figure 1.

This project scope includes the following elements:

1. Phase 1–Collection system flow metering and data analysis

2. Phase 2–Identifications and evaluation of recommended collection system improvements

The following tasks were included as part of the overall project:

1. Flow metering site selection

2. Equipment installation and calibration

3. Data collection

4. Equipment removal

5. Data analysis

6. Preparation of results summary

Abbreviations and Definitions

ADWF	average dry weather flow
ft	feet
gpm	gallons per minute
ID	identification
I/I	infiltration and inflow
in	inch
MH	manhole
PF	Peaking Factor
Strand	Strand Associates, Inc.®

Flow Metering Locations

Three temporary flow meters were installed to evaluate the flow rates and available capacity. The evaluation focused on the 12-inch gravity sewer along Fairbrook Drive. This area is located on the north side of the Village and is downstream of potential future development areas to the north and west. Preliminary flow meter locations were identified and reviewed with Village personnel.

Meter A was located at MH SN052017 within an 8-inch gravity sewer on Greenbrier Drive. This location was selected to monitor flows within the local sewers of the North Ridge neighborhood and determine available capacity to allow future development north of Easy Street anticipated to connect to the 8-inch gravity sewer on North Madison Street via a new pumping station and force main. Available capacity would be used to size the future pumping station.

Meter B was located at MH SN053064 within a 12-inch gravity sewer on West Verleen Avenue. This location was selected to monitor flows from the area's tributary to the Meadowbrook and Westbridge pumping stations, as well as portions of the Sixmile Creek neighborhood. This location was selected to determine the amount of flow entering the Fairbrook Drive sewer from the west prior to combining with flows from the north.

Meter C was located at MH SN053031 within the 12-inch Fairbrook Drive sewer, which was the primary sewer of interest for the Study. This location includes flow from Meters A and B, as well as local sewers that flow into the 12-inch Fairbrook Drive sewer upstream of Pinehurst Court.

Table 1 lists the temporary flow meter ID, pipe diameter, and installation location (MH ID) for each flow meter. Enclosed Figure 2 displays the locations of the three flow metering locations.

Meter ID	Pipe Diameter (in)	Installation Location (MH)	Installation Location
A	8	SN052017	Greenbrier Drive
B	12	SN053064	West Verleen Avenue
C	12	SN053031	Fairbrook Drive

Table 1–Flow Meter Locations

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Equipment Installation and Removal

The flow meters were installed in the collection system from March 26 to May 23, 2024. For each location, a MH entry was made, and the equipment was placed into operation and calibrated. Calibration consisted of taking a manual level reading in the sewer and comparing it to the level reading of the flow meter. Software provided with the flow meters allowed the user to enter the correct level reading, thereby calibrating the unit.

Later in the day following initial installation, each flow metering site was revisited, and another MH entry was made. Levels were measured and compared to meter readings. If necessary, the calibration of each meter was adjusted. Usually, after the first adjustment, the meters will stay calibrated. However, an adjustment can be necessary because the internal electronics of the flow meters adjust to the in-situ temperature and humidity conditions of the sanitary sewer. When the meters were removed, a MH entry was made, and the equipment was calibrated one last time to check the readings of the equipment throughout the Study. Readings taken during calibration of the removal phase were within 10 percent at each meter location, consistent with the level of accuracy of the equipment.

Equipment Maintenance and Data Collection

After the initial installation and subsequent calibration checks, the flow meters were visited once every 2 weeks. Data was collected from each unit, and a visual check of data quality was made to review the meters were operating correctly. Typical maintenance activities included cleaning the level transducer and reviewing that batteries and desiccant were in adequate condition.

Following each data collection, a more thorough evaluation of the data was performed. This included performing a mass balance on the data and comparing the results to make sure the meter results made sense relative to other upstream and/or downstream meters. The data was also reviewed to evaluate the response from rainfall events.

Data Summary

A dry weather flow analysis was performed to determine the dry weather flow characteristics of each metering basin. The dry weather flows for the meter sites were calculated using 15-minute flow data collected from a dry weather period during the metering program. During the week of April 27 to May 3, 2024, the Village experienced relatively dry weather. At each site, the flow data from each 15-minute interval was averaged to create an overall weekly dry weather hydrograph for the site. From this hydrograph, the overall average dry weather flow rate was established.

It should be noted that flow levels tributary to Meter A, located on Greenbrier Drive, were consistently lower than the threshold of the area-velocity meters to record a velocity, which is necessary for the device to calculate a flow rate. The level readings were consistently between 1 and 2 inches, suggesting that the sewer has significant capacity remaining. Flows reported in the summary tables were taken when levels were high enough for the device to record velocity measurements and, therefore, calculate flow rates. It should also be noted that during initial and final calibration checks, the manual level readings were considered accurate to the level readings reported by the metering device. These results were not unexpected because the tributary area is relatively small and I/I was found to be negligible.

Theoretical full pipe capacities were calculated by inputting pipe diameters and slopes from Record Drawings of the sewers where meters were installed into Manning's equation. A roughness coefficient of 0.013, which can generally be considered conservative for the polyvinyl chloride pipes found in the Study area, was used as the standard for existing sanitary sewers.

A summary of ADWF at each metering location is provided in Table 2.

Meter ID	Pipe Diameter (in)	Theoretical Full Pipe Capacity (gpm)	ADWF (gpm)	Percent Capacity (%)
A	8	345	13	4
B	12	718	50	7
C	12	750	103	14

Table 2–Dry Weather Flow Summary

A peak flow analysis was performed to determine the maximum average flow rate during a 15-minute interval at each metering basin. The peak flow was compared to the average flow rate during the time period of March 26 to May 23, 2024, to determine the observed peaking factor for that 15-minute interval. A summary of these values for all three meters is provided in Table 3. There were four rainfall events that totaled more than 0.5 inches of rainfall during the Study period. The largest observed rainfall event during the temporary flow metering Study had a 6-month recurrence interval and occurred on April 1, 2024.

Meter ID	Pipe Diameter (in)	Theoretical Full Pipe Capacity (gpm)	Peak 15-Minute Flow (gpm)	Observed PF	Percent Capacity (%)
A	8	345	37	2.85	11
B	12	718	206	3.81	29
C	12	750	283	2.64	38

Table 3–Wet Weather Flow Summary

Level and flow charts for each meter are located in the enclosed Appendix. The flow chart for Meter A is excluded because of limited velocity readings as previously discussed.

Conclusions and Recommendations

Temporary flow meters were installed in three locations throughout the Village to measure sanitary sewer flows. The flow metering program was conducted to assess available capacity in the existing sanitary sewer infrastructure that would allow for future development to be served. It is reasonable to assume relatively low levels of I/I during wet weather events are observed in the Village's sanitary sewer system in this area because of the lack of response to wet weather events during the monitoring period.

Meter A, at the intersection of Greenbrier Drive and Valderama Court, has approximately 310 gpm of available capacity, or 89 percent, as observed during the 6-month recurrence interval storm on April 1, 2024.

Meter B, at the intersection of West Verleen Avenue and Muirfield Court, has approximately 510 gpm of available capacity, or 71 percent, as observed during the 6-month recurrence interval storm on April 1, 2024.

Meter C, at the intersection of Fairbrook and Pinehurst Drives, has approximately 465 gpm of available capacity, or 62 percent, as observed during the 6-month recurrence interval storm on April 1, 2024. Therefore, the West Verleen Avenue sewer is limited in capacity because of the capacity of the downstream Fairbrook Drive sewer.

Potential next steps include the following:

1. Continue to monitor the Fairbrook Drive sewer as development progresses to the north and west.

2. Monitor and/or evaluate sewers downstream of the Fairbrook Drive sewer to determine available capacity.

3. Monitor and/or evaluate other sewers within the Village to determine potential areas for future development that can be served by the existing sanitary system.

Strand appreciates the continued opportunity to assist the Village with its engineering needs and welcomes questions regarding the flow monitoring program letter.

Sincerely,

STRAND ASSOCIATES, INC.®

R. Kent Straus, P.E.

Ryan M. Yentz, P.E.

c/enc.: Randy Dorn, Waunakee Utilities

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EXISTING SANITARY COLLECTION SYSTEM

VILLAGE OF WAUNAKEE
DANE COUNTY, WISCOSONSIN

STRAND ASSOCIATES®

FIGURE 1

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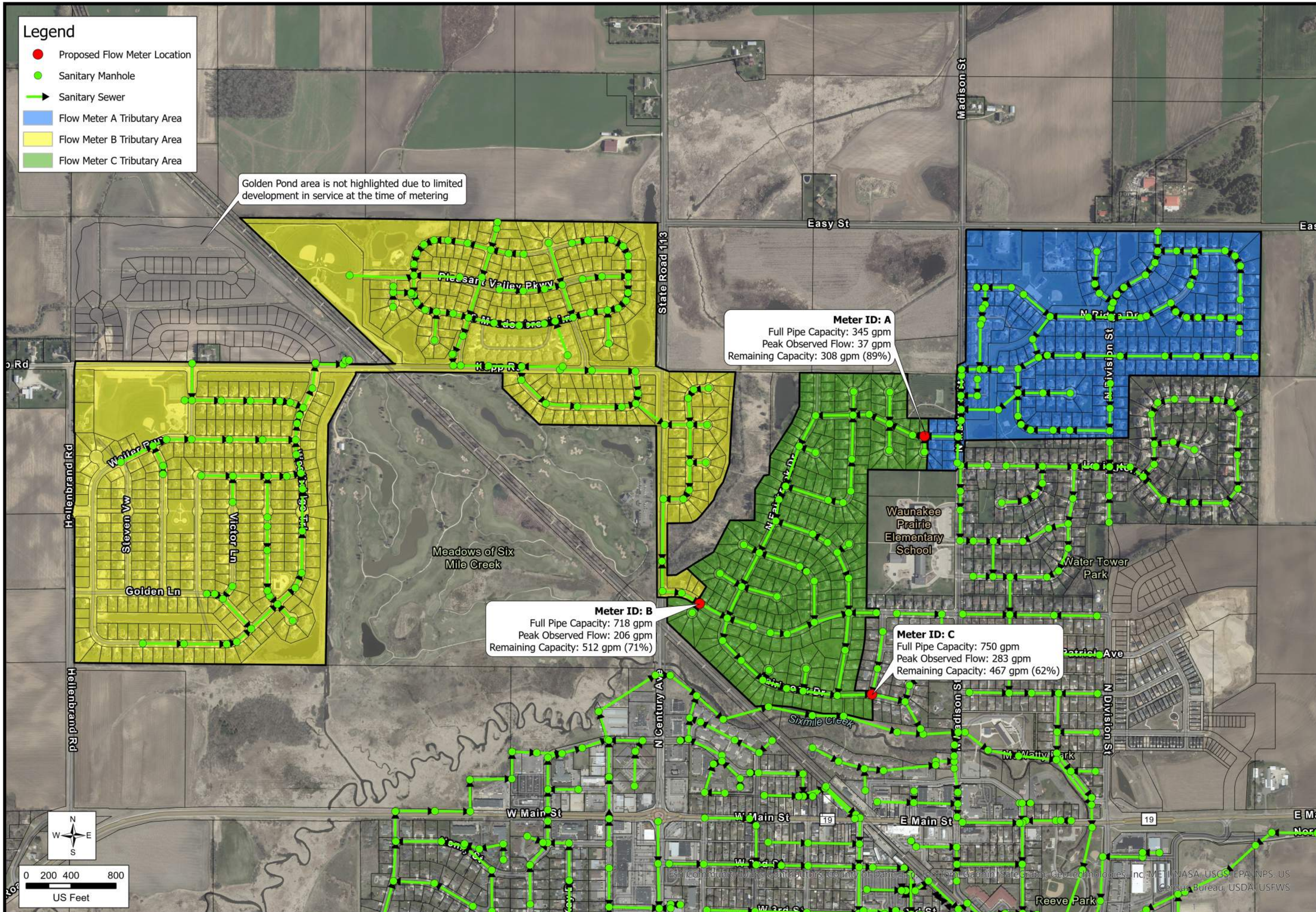


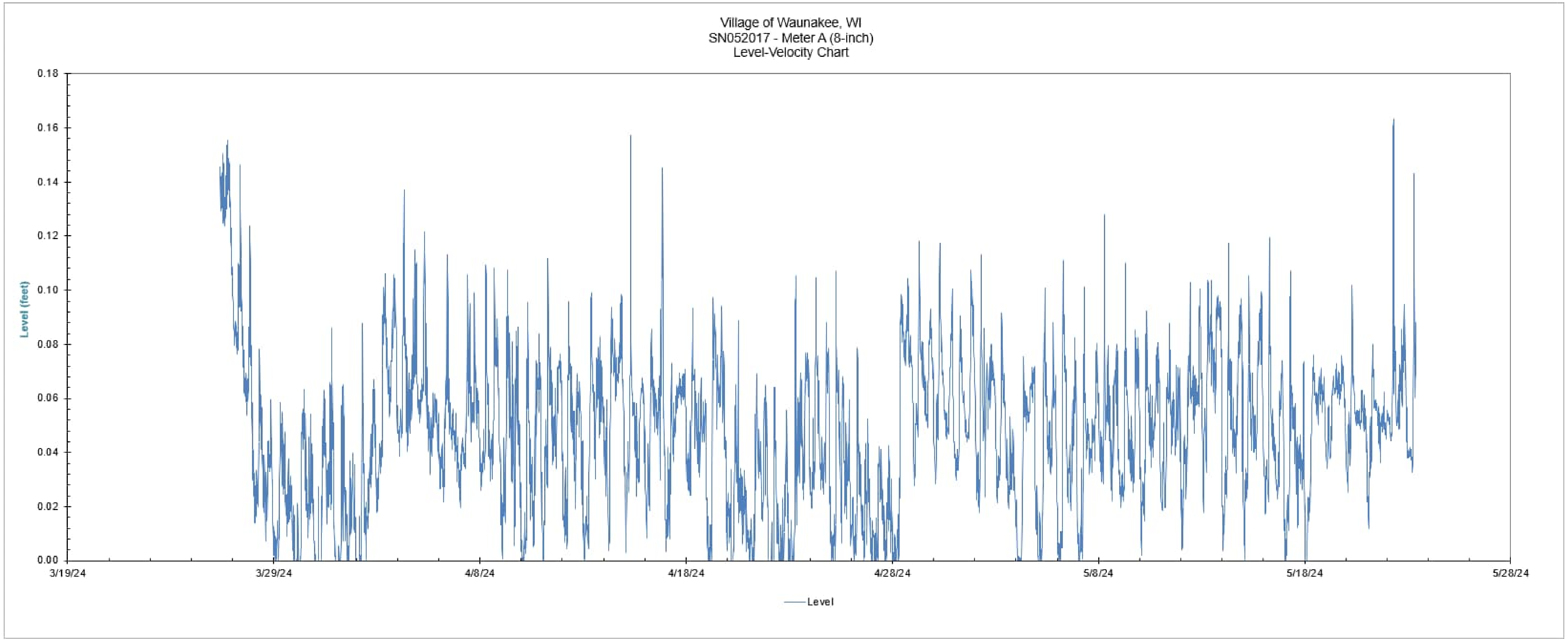
TEMPORARY FLOW METERING LOCATIONS

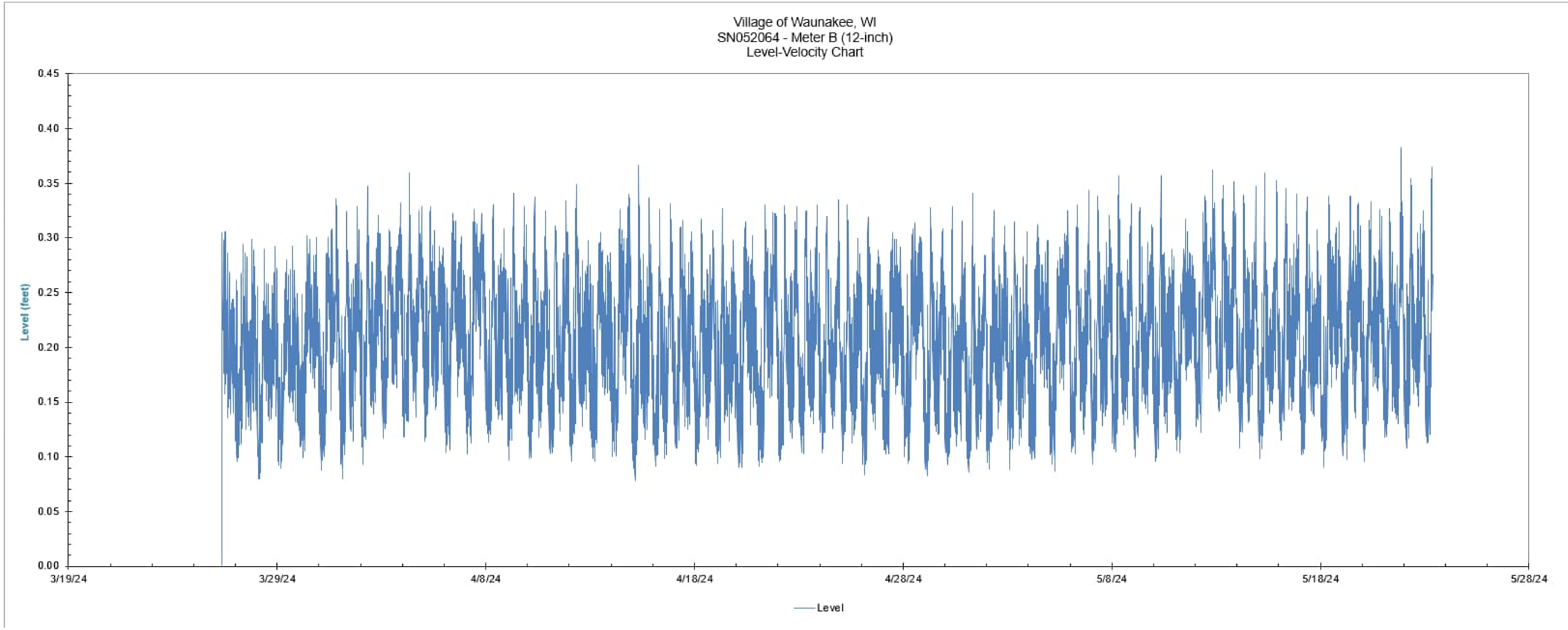
2024 FLOW METERING PROGRAM
VILLAGE OF WAUNAKEE
DANE COUNTY, WISCONSIN

STRAND ASSOCIATES®

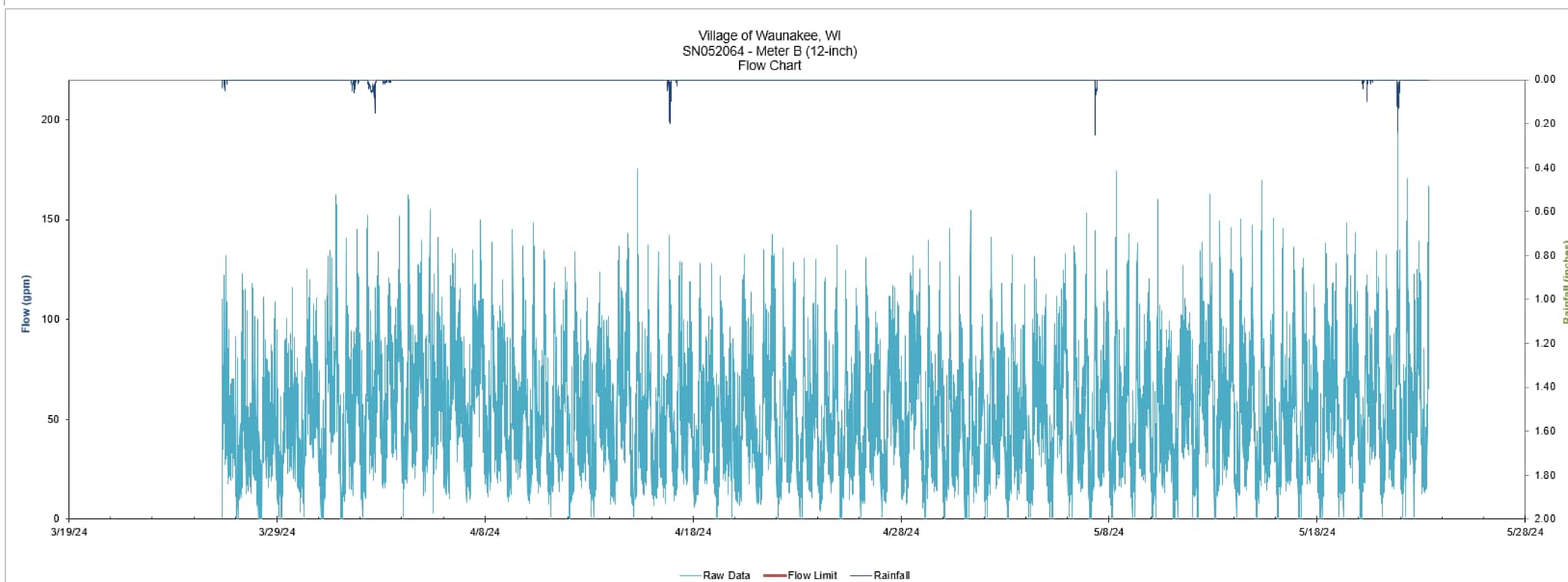
FIGURE 2
1602.165

Path: \\strand.com\Projects\MAD\1600--1699\1602\165\Drawings\GIS\Waunakee Flow Monitoring\Waunakee Flow Monitoring.aprx User: Danc Date: 7/8/2024 Time: 10:39 AM

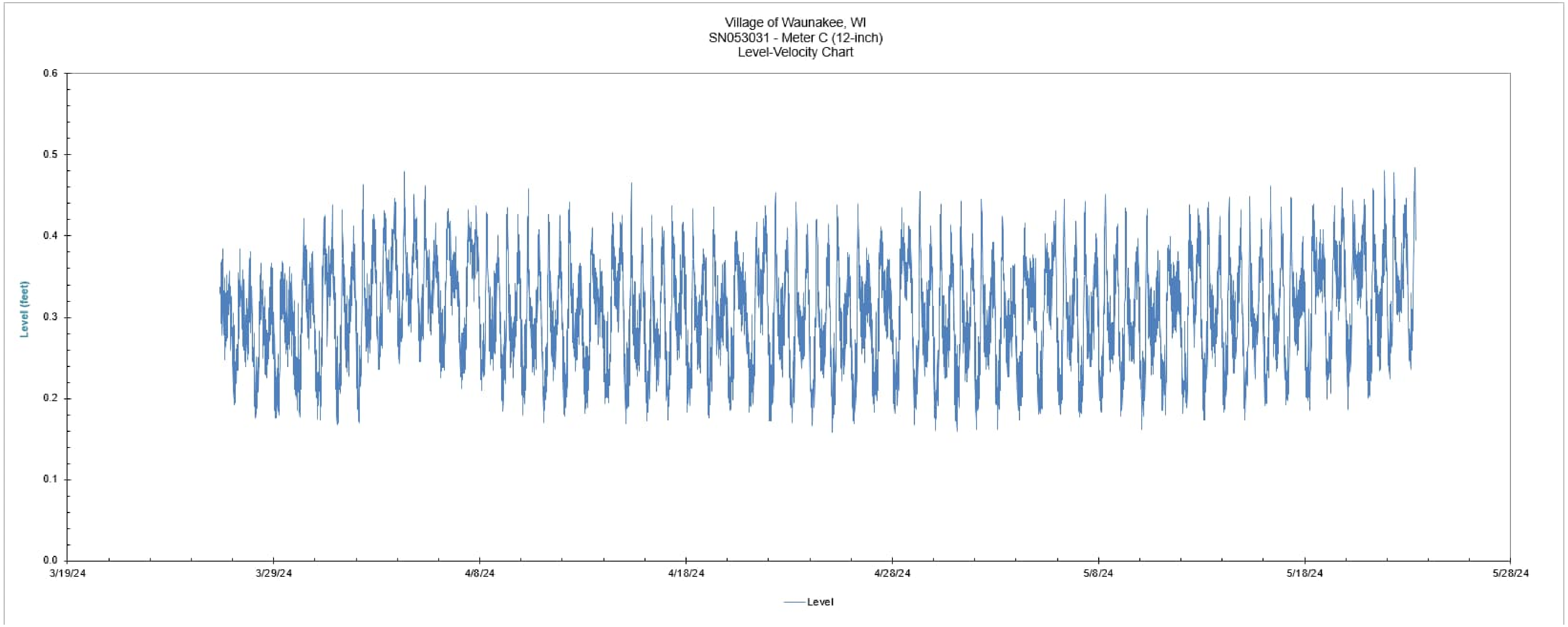

Village of Waunakee, WI
SN052017 - Meter A (8-inch)
Level-Velocity Chart
Level (feet)
0.18
0.16
0.14
0.12
0.10
0.08
0.06
0.04
0.02
0.00
3/19/24
3/29/24
4/8/24
4/18/24
4/28/24
5/8/24
5/18/24
5/28/24
Level

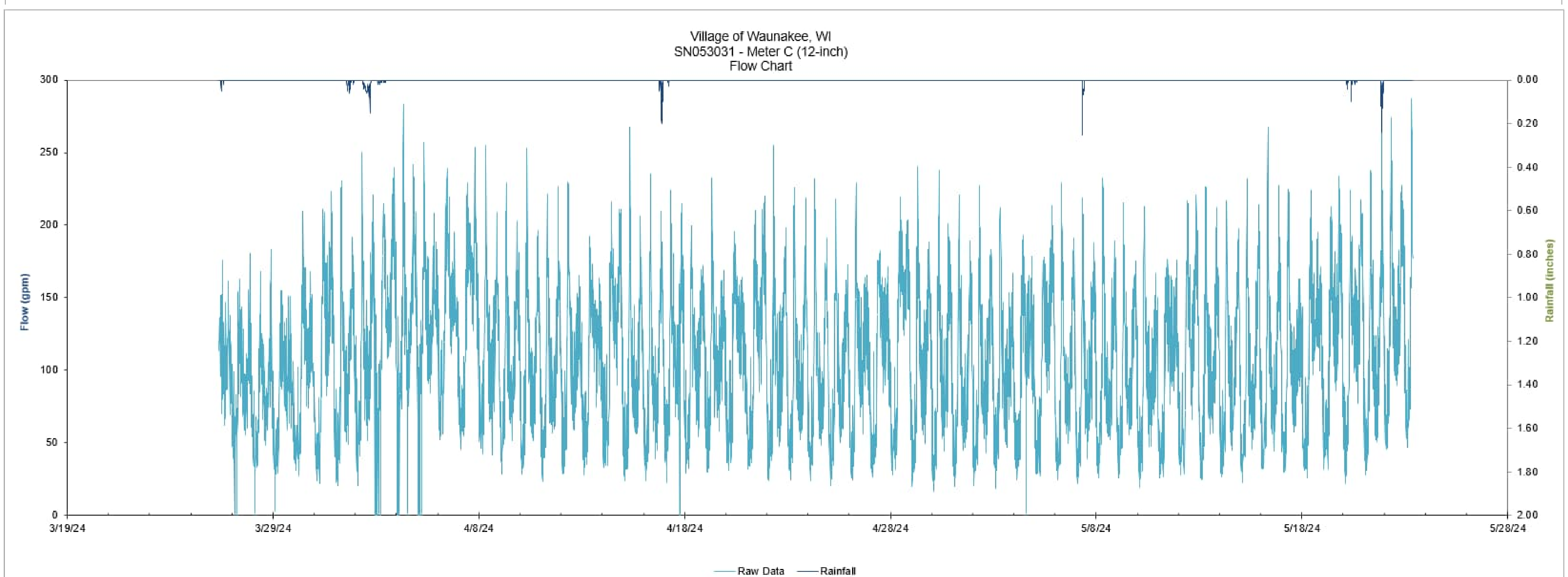


Village of Waunakee, WI
SN052064 - Meter B (12-inch)
Level-Velocity Chart
Level (feet)
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
3/19/24
3/29/24
4/8/24
4/18/24
4/28/24
5/8/24
5/18/24
5/28/24
Level



Village of Waunakee, WI
SN052064 - Meter B (12-inch)
Flow Chart
Flow (gpm)
200
150
100
50
0
Rainfall (inches)
0.00
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
1.80
2.00
3/19/24
3/29/24
4/8/24
4/18/24
4/28/24
5/8/24
5/18/24
5/28/24
Raw Data
Flow Limit
Rainfall


Village of Waunakee, WI
SN053031 - Meter C (12-inch)
Level-Velocity Chart
Level (feet)
0.6
0.5
0.4
0.3
0.2
0.1
0.0
3/19/24
3/29/24
4/8/24
4/18/24
4/28/24
5/8/24
5/18/24
5/28/24
Level


Village of Waunakee, WI
SN053031 - Meter C (12-inch)
Flow Chart
Flow (gpm)
300
250
200
150
100
50
0
Rainfall (inches)
0.00
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
1.80
2.00
3/19/24
3/29/24
4/8/24
4/18/24
4/28/24
5/8/24
5/18/24
5/28/24
Raw Data
Rainfall

Report for

Waunakee Utilities, Village of Waunakee, Wisconsin

Water System Study Update

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12-5-2018

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December 2018

STRAND ASSOCIATES®

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EXECUTIVE SUMMARY

EXECUTIVE SUMMARY

The Village of Waunakee (Village) continues to grow through development of residential, commercial, and industrial areas. Residential growth is the primary driver for expansion of the water supply systems and sanitary sewer system. The Sanitary Sewer Comprehensive Plan has been updated in parallel with this Water System Study Update (2018 Study). Both reports should be referenced as the service area expands.

A. Changes Since 2013 Report

As a backdrop for discussing current and future expansion of the water system, the following summarizes the areas of growth and infrastructure improvements since 2013.

1. Residential Development

The Westbridge development, located west of the Meadows of Sixmile Creek golf course and south of Kopp Road, is a 135-acre residential development with 283 lots. The area is served with water and sewer service along the Kopp Road corridor. A redundant water main connection was also installed from the southern edge of the development to an existing main along Highway 19.

The Northridge subdivision was partially complete at the time of the 2013 report. In the past five years, the northeast portion of the development was completed. Water service is provided by an extension of mains from the primary pressure zone.

The Kilkenny development is located west of the Southbridge neighborhood and east of CTH Q. The area is primarily residential with a small commercial area as discussed below. There are 383 residential lots platted with homebuilding ongoing.

The Kilkenny Farms–West neighborhood is located at the southwest corner of CTH Q and Woodland Drive. The development includes residential, mixed use, and commercial areas. Residential areas are anticipated to include 210 single family dwellings and 400 apartment units. Approximately 16.8 acres will be commercial area.

Arboretum Village is a 113-lot residential development located north of Arboretum Drive and west of Hogan Road.

2 Commercial Development

Kilkenny Farms Commons is a 43 acre commercial/retail development located in the northwest corner of the Kilkenny residential neighborhood in the CTH Q corridor. Proposed businesses here include medical and dental offices, retail shops, dining establishments, and child and elder care facilities. Development in this area is ongoing.

3. Redevelopment Downtown (SM-1 Area)

In 2013, redevelopment at the northeast corner of Madison Street and Main Street produced a 50-unit apartment building with commercial space on the ground floor. It is called Madison/Main development.

In 2015, commercial redevelopment of the former Koltes Lumber property occurred, producing several commercial and restaurant spaces. This site will be referred to as the Lone Girl site, being the current anchor tenant.

Under construction in 2018 is the redevelopment of the north 200 block of East Main Street, which will consist of 105 apartment units and two restaurant/commercial spaces. It is called Lamphouse.

4. Industrial Development

Frank H. Street located in the Waunakee Industrial Park was extended by approximately 550 feet in 2017. The project extended water and sewer service to allow approximately 20 acres of industrial development. Construction of a multi-unit small business building is underway in 2018. Further development in this subbasin is likely when the demand materializes.

In total, 1,483 dwelling units have either been built or will be built in the ongoing areas of development. Using the criteria established in the 2013 Study and the March 21, 2017 letter to Waunakee Utilities (Utility) on water storage (see Appendix), the theoretical water use would be 400,855 gallons per day (gpd) (1,483 du x 3.18 ppl/du x 85 gcd).

Because water use projections are tied directly to population growth, it is worth comparing the build-out populations of the known and ongoing developments to the number of existing service connections.

B. System Overview

The Waunakee Utilities (Utility) operates a water system consisting of five wells, four elevated storage tanks, one ground-level reservoir and booster station, and two local booster stations. The water distribution system includes 61 miles of water main ranging in size from 6- to 12-inch-diameter. The water system is efficiently operated and built to support future growth of the service area. The system is capable of meeting maximum day demands while maintaining adequate operating pressure and fire flow.

C. Summary of Findings

The existing well supply can produce water at a total rate of 5,320 gpm. Prudent system planning should consider the firm capacity as the reliable amount of supply. Firm capacity assumes the largest pumping unit is out of service. The existing firm well supply is 4,020 gpm.

Based on updated population projections and water use trends discussed in Section 3, the projected 2030 maximum day demand is equivalent to a rate of 3,300 gpm. The resulting well surplus of 720 gpm will allow significant growth of the service area before another well is needed. Actual demands should be monitored and compared with available supply as the timing of future development is variable.

The system includes 1,350,000 gallons of storage volume. Section 3 estimates the required storage volume based on 2020 and 2030 water demands. For the 2020 design year, there is small surplus in water storage of approximately 52,500 gallons. However, the storage is forecast to reach a deficit of 235,800 gallons by the year 2030.

Construction continues within existing developments in the Village, and there are specific regions of potential future development the Utility should plan to serve with water. In conjunction with this Water System Study, the Utility completed a *Sanitary Sewer Comprehensive Plan* which evaluates areas that can be served by existing sewer without major downstream capacity improvements. This study uses those areas to estimate water supply needs. Section 3 estimates the water demands associated with these areas including the ultimate service area.

This report investigates the feasibility of serving areas within the ultimate service area from a hydraulic viewpoint. Section 3 includes a discussion of several areas identified by the Village that may develop in the near future regardless of existing sewer capacities. It is important to note the evaluation of potential future service areas does not mean development cannot occur elsewhere within the ultimate service area. However, certain areas will require downstream improvements to the sanitary collection system before development. Similarly, there are areas that could be physically served by existing sewer and water system capacity, but are not likely to develop based on planned land use or other factors.

Section 5 includes a brief summary of the water system model update. The model was not re-calibrated and was not used for system simulation as part of this study update. The findings of the 2013 modeling effort are still valid and that report should be referenced as needed.

Section 6 summarizes the recommendations, implementation time, and cost for system improvements. The Utility should begin planning for a new water storage facility in 2020. A new 400,000-gallon storage facility should be online by 2025 or sooner if development proceeds faster than anticipated. A storage sizing and siting evaluation should be conducted before a final site selection is made. Potential sites for new ground-level storage include the existing Well No. 5 site or a new site obtained as development continues. While a new well may not be needed until after 2030, the Utility should secure a well site as development continues. With a well site secured, the Utility can proceed with Well No. 6 whenever the need arises.

The Utility should update this Water System Study every five years to ensure that infrastructure improvements keep pace with development.

SECTION 1
INTRODUCTION

1.01 PURPOSE

The purpose of this report is to provide an updated analysis to the Village of Waunakee's (Village) and Waunakee Utilities' (Utility) 2013 Water System Study (2013 Study) to account for actual growth that has occurred, and to develop an updated comprehensive plan for growth of its water utility infrastructure.

1.02 SCOPE

The study area includes those portions of the Village currently supplied with municipal water as well as future areas that will require water service.

The scope of the report includes the following elements:

1. Review water use data for years 2013 through 2017 to supplement data summarized in the 2013 Study.

2. Compare Village population projections used in the 2013 Study to actual growth using data on metered connections.

3. Estimate future water demands based on historic water use and population projections.

4. Evaluate future water supply and storage capacity out to the year 2030.

5. Compare current areas of future growth to those reviewed in the 2013 Study and prepare up to three potential system improvements to service proposed areas of development.

6. Develop a Capital Improvement Plan out to 2030, including opinions of probable cost (OPCC) and implementation schedules.

1.03 DEFINITIONS

CARPC	Capital Area Regional Planning Commission
CIP	Capital Improvement Plan
FUDA	Future Urban Development Area
gcd	gallons per capita per day
GIS	geographic information system
gpd	gallons per day
gpm	gallons per minute
hp	horsepower
ISO	Insurance Services Office
mdg	million gallons per day
MSL	mean sea level
OPCC	opinion of probable construction cost

PE	population equivalent
PSC	Public Service Commission
psi	pounds per square inch
TDH	total dynamic head
Utility	Waunakee Utilities
Village	Village of Waunakee
WDNR	Wisconsin Department of Natural Resources
WDOA	Wisconsin Department of Administration

SECTION 2
EXISTING WATER SYSTEM

2.01 WATER SUPPLY SYSTEM SUMMARY

This section summarizes the Village's water supply system. The information is mostly unchanged from the 2013 Study. Figure 2.01-1 shows the updated distribution system.

A. Well Supply

The Utility operates five groundwater wells located throughout the Village. Table 2.01-1 presents the total and firm well capacity of the system, as well as each well operating point. The total well capacity is 7.661 million gallons per day (mgd) or 5,320 gallons per minute (gpm). The firm well capacity, assuming the largest well out of service, is 5.789 mgd or 4,020 gpm.

Well No.	Capacity (gpm)	Capacity (mgd)
1	640	0.922
2	900	1.296
3	1,300	1.872
4	1,280	1.843
5	1,200	1.728
Total Capacity	5,320	7.661
Firm Capacity*	4,020	5.789

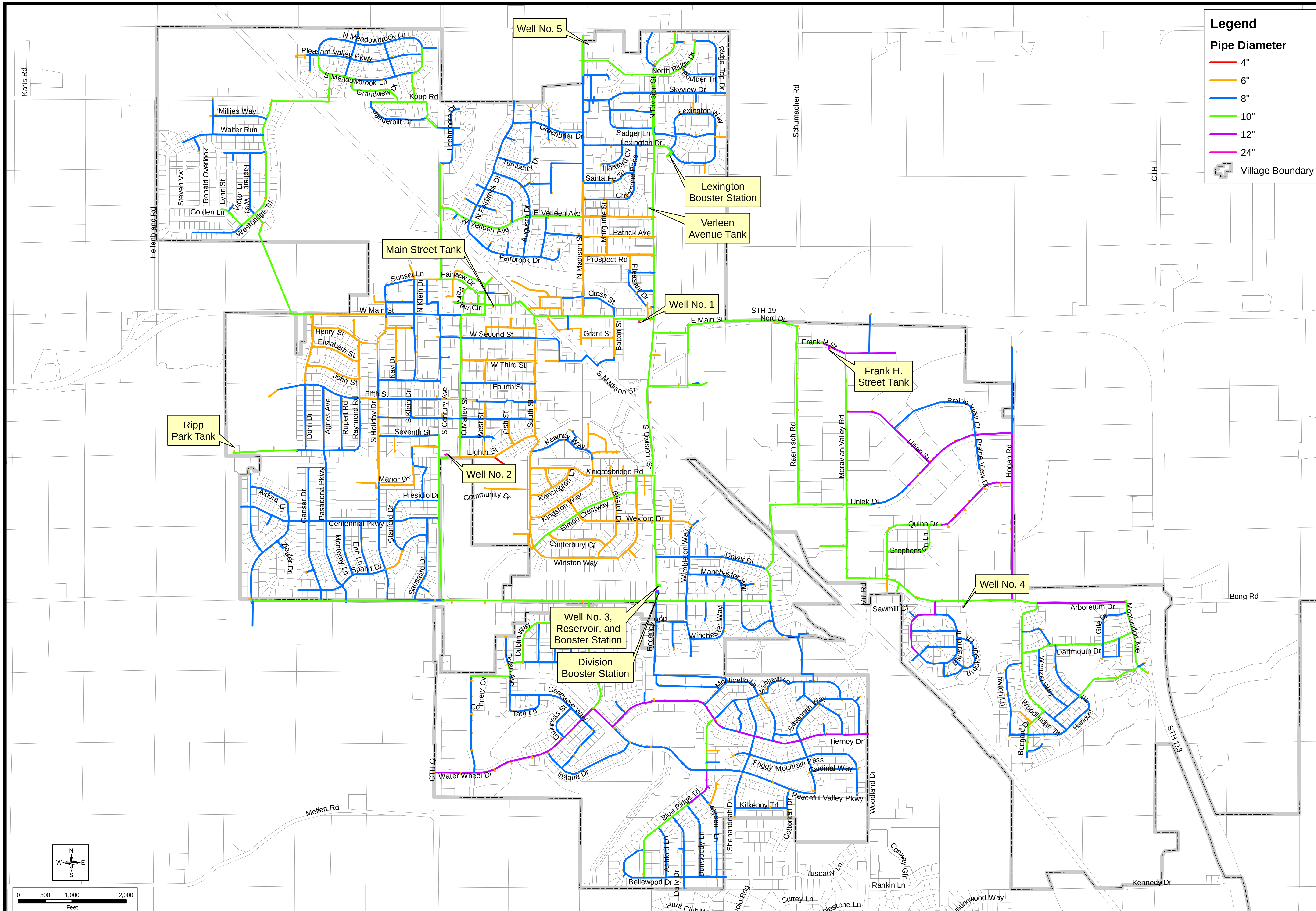
*Assumes Well No. 3 out of service.

Table 2.01-1 Well Capacity

B. Water Storage

The Utility operates a ground-level reservoir that is located adjacent to Well No. 3 and has a capacity of 300,000 gallons. Two booster pumps, each with a capacity of 1,250 gpm, draw water from the reservoir and pump directly to the distribution system.

The Utility also operates four elevated tanks which maintain pressure in the main zone. Table 2.01-2 presents a summary of the elevated tanks. The total storage capacity of all storage facilities is 1,350,000 gallons, including the ground-level reservoir. For future storage calculations and to keep consistent with the 2013 Study, the West Main Street tank volume will be ignored based on its relatively small capacity compared to the total storage capacity of all facilities and the possibility of removing the tank from service in the future.



EXISTING DISTRIBUTION SYSTEM
WATER SYSTEM STUDY UPDATE

WAUNAKEE UTILITIES
VILLAGE OF WAUNAKEE
DANE COUNTY, WISCONSIN

STRAND ASSOCIATES®

FIGURE 2.01-1
1602.124

Path: S:\MAD\1600--1699\1602\124\Drawings\GIS\Figures\Water System 11x17.mxd User: danc Date: 7/3/2018 Time: 8:03:00 AM

Location	Year Constructed	Capacity (gallons)	Overflow Elevation (feet)	Operating Range (feet)
Main Street	1928	50,000	1064.0	30.0
Verleen Avenue	1969	200,000	1064.5	30.0
Ripp Park	1992	300,000	1063.0	32.5
Frank H. Street	2001	500,000	1064.0	37.5
Well No. 3 Reservoir	1987	300,000	N/A	N/A
Total Capacity		1,350,000		

Table 2.01-2 Total Storage Capacity

C. Distribution System

The Village water distribution system includes approximately 70 miles of water main ranging in size from 6 to 12 inches in diameter. There is a Main Pressure Zone, which serves the majority of the distribution system, and two locally boosted pressure zones. The Lexington Drive Locally Boosted Zone is fed by a pumping station with two 500 gpm booster pumps that serves the area around Lexington Drive. The Division Street Locally Boosted Zone is fed by a pumping station with two 500 gpm booster pumps that serves the area around Blue Ridge Trail. Each locally boosted pressure zones contain a 5,000-gallon hydropneumatic tank that maintains system pressure in the zones when the booster pumps are not operating.

SECTION 3
HISTORICAL AND PROJECTED WATER DEMANDS

3.01 GENERAL

This section presents the updated water demands currently satisfied by the Utility and develops a projection of future demands. Updated water use trends are applied to population projections to estimate the future water demands out to the year 2030.

3.02 SERVICE AREA

Water service is presently provided to the corporate boundaries of the Village. Figure 2.01-1 shows the approximate extent of areas served by the Utility. Similar to the 2013 report, it is anticipated that areas of growth will first occur around the periphery of the current corporate boundaries where sanitary sewer can reasonably be extended to serve future growth.

As a backdrop for discussing future expansion of the water system, the following summarizes the areas of growth since 2013.

A. Residential Development

The Westbridge development, located west of the Meadows of Sixmile Creek golf course and south of Kopp Road, is a 135-acre residential development with 283 lots. The area is served with water and sewer service along the Kopp Road corridor. A redundant water main connection was also installed from the southern edge of the development to an existing main along Highway 19.

Phase 1 of the Northridge subdivision was partially complete at the time of the 2013 Study. In the past five years, Phase 2 of the development was completed. Phase 2 added 94 residential lots to the service area. Water service is provided by an extension of mains from the primary pressure zone.

The Kilkenny development is located west of the Southbridge neighborhood and east of Highway Q. The area is primarily residential with a small commercial area along Highway Q. There are 383 residential lots platted with homebuilding ongoing. Water service is provided by the main pressure zone including a backbone of 12-inch diameter pipe.

The Kilkenny Farms–West neighborhood is located at the southwest corner of Highway Q and Woodland Drive. The development includes residential, mixed use, and commercial areas. Residential areas are expected to include 210 single family dwellings and 400 apartment units. Water service will be provided by water main crossing under Highway Q with a 12-inch connection at Water Wheel Drive and a 10-inch connection at Peaceful Valley Parkway.

Arboretum Village is a 113-lot residential development located north of Arboretum Drive and west of Hogan Road. Water service is provided by a network of 8-inch water main.

B. Commercial Development

Kilkenny Farms Commons is a 43-acre commercial/retail development located in the northwest corner of the Kilkenny residential neighborhood. Proposed businesses here include medical and dental offices, retail shops, dining establishments, and elder and child care facilities. Development in this area is ongoing.

The commercial area associated with Kilkenny Farms–West will take up approximately 16.8 acres.

C. Industrial Development

Frank H. Street located in the Waunakee Industrial Park was extended by approximately 550 feet in 2017. The project extended water and sewer service to allow a small area of industrial development in this area. Construction of a multi-unit small business building is underway in 2018. Further development in this subbasin is likely when the demand materializes.

3.03 POPULATION PROJECTIONS

This section compares the population projections used in the 2013 Study to actual growth using data on metered connections. The section then presents the updated projections used to evaluate future water supply and storage capacity out to the year 2030.

A. 2013 Population Projections

From the 2013 Study, the 2013 design population of 12,622 was obtained by linear interpolation of the 2010 census data and the 2015 WDOA population projection. The 2017 population estimate of 14,838 was calculated by using linear interpolation of the 2010 census data and the 2020 Capital Area Regional Planning Commission (CARPC) population estimate. Finally, the 2030 CARPC population projection of 19,693 was used as the 2030 design population. See Figure 3.03-1 for a graph of the projections used in the 2013 Water System Study.

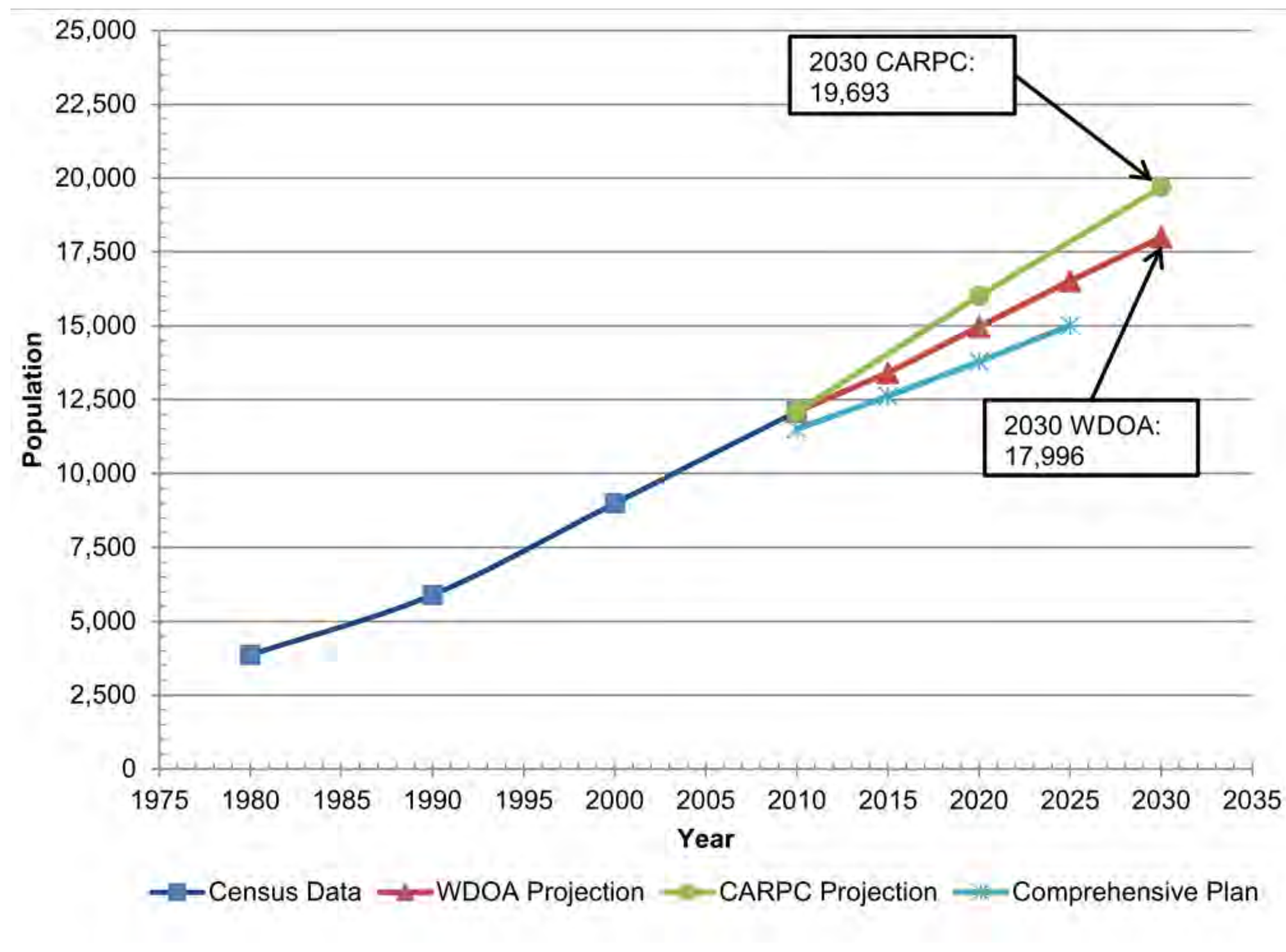


Figure 3.03-1 Population Projections from 2013 Study

B. Actual Growth

Actual population growth was estimated based on the number of known water meter connections and a population density factor. In 2010, the Census population of the Village was 12,097 and there were 3,807 residential service connections. The calculated factor used to estimate the population growth was 3.18 people per residential connection. Table 3.03-1 shows the number of reported residential water services connections since 2010 and the resulting calculated population growth estimate.

Year	Residential Water Service Connections	Calculated Population Growth Estimate
2010	3,807	12,106
2011	3,850	12,243
2012	3,901	12,405
2013	3,979	12,653
2014	4,134	13,146
2015	4,255	13,531
2016	4,367	13,887
2017	4,459	14,180

Table 3.03-1 Metered Connections–Population Estimates

C. Comparison of Projections

The estimated population for 2017 is 14,180. This is just slightly below the CARPC projection shown in the 2013 Water System Study report (14,838), and slightly above the Wisconsin Department of Administration (WDOA) linearly interpolated projection (14,036). This information suggests the Village is growing at a rate consistent with the projections used in the 2013 Study. Therefore, the updated population projections will be based off of the same methodology as used in the 2013 Study.

D. Updated Population Projections

Figure 3.03-2 presents the updated population projections from several sources. The figure is supplemented by United States Census Bureau population data from 1980, 1990, 2000, and 2010, updated estimates and projections from the WDOA, calculated population growth estimates based on water meter data, and the current projections from the CARPC.

Based on the findings of the population projection comparison, and with approval from the Utility, the CARPC population projections will remain as the method used to estimate future water demands. The 2020 CARPC population projection of 16,013 will be used as the 2020 design population. The 2030 CARPC population projection of 19,693 will be used as the 2030 design population.

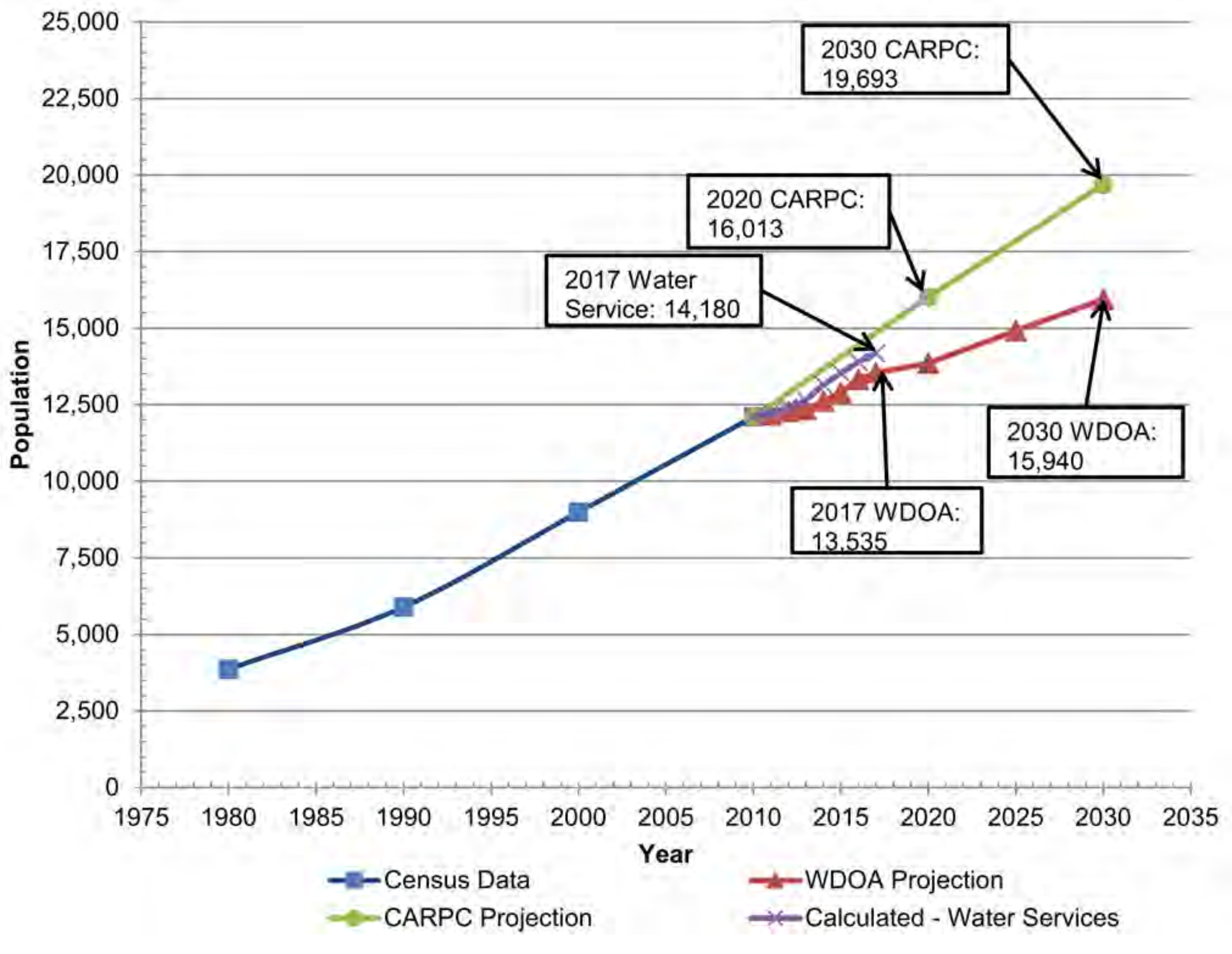


Figure 3.03-2 Population Projections

3.04 WATER SALES AND PUMPAGE

A. Water Use Records

The Utility's historical water use records were obtained from the Wisconsin Public Service Commission (PSC) *Water, Electric, Gas, and Sewer Annual Reports* for the years 1997 through 2017. Table 3.04-1 summarizes the updated historical water pumpage and sales data.

Year	Annual Pumpage (gal)	Average Day Pumpage (gpd)	Maximum Day Pumpage (gpd)	Average Day Sales (gpd)	Sales to Pumpage Ratio	Maximum to Average Day Ratio
1997	368,273,000	1,008,277	2,118,000	937,509	0.93	2.10
1998	313,512,000	858,349	1,625,000	794,133	0.93	1.89
1999	318,050,000	870,773	1,388,000	823,630	0.95	1.59
2000	324,604,000	888,717	1,496,000	825,900	0.93	1.68
2001	347,263,000	950,754	2,260,000	900,085	0.95	2.38
2002	369,893,000	1,012,712	2,690,000	906,946	0.90	2.66
2003	405,842,000	1,111,135	2,471,000	969,489	0.87	2.22
2004	401,266,000	1,098,606	1,757,000	1,033,495	0.94	1.60
2005	483,154,000	1,322,804	3,327,000	1,239,565	0.94	2.52
2006	473,653,000	1,296,791	2,177,000	1,165,325	0.90	1.68
2007	498,221,000	1,364,055	2,808,000	1,216,969	0.89	2.06
2008	467,484,000	1,279,901	2,331,000	1,163,786	0.91	1.82
2009	489,342,000	1,339,745	2,302,000	1,160,868	0.87	1.72
2010	458,631,000	1,255,663	2,127,000	1,145,678	0.91	1.69
2011	478,837,000	1,310,984	2,420,000	1,140,364	0.87	1.85
2012	521,132,000	1,426,782	3,500,000	1,298,349	0.91	2.45
2013	456,563,000	1,250,001	2,779,000	1,132,334	0.91	2.22
2014	480,137,000	1,314,543	2,625,000	1,109,563	0.84	2.00
2015	466,228,000	1,276,463	2,792,000	1,065,005	0.83	2.19
2016	498,400,000	1,364,545	2,543,000	1,103,277	0.81	1.86
2017	444,091,000	1,215,855	3,436,000	1,101,665	0.91	2.83

Table 3.04-1 Water Pumpage and Sales Data

B. Sales to Pumpage Ratio

Figure 3.04-1 presents sales to pumpage ratios from 1997 to 2017. Years 2014 through 2016 observed the three lowest sales to pumpage ratios since 1997 and appear to be outside of the previous trend. Year 2017 provided a sales to pumpage ratio of 0.91, which was trending with the years prior to 2014. A discussion with Utility personnel suggested that the sales to pumpage ratio prior to 2014 better represents the actual sales to pumpage ratio of the system. The sales to pumpage ratio used to calculate future demands will be 88 percent, slightly lower than the 90 percent used for the 2013 Study.

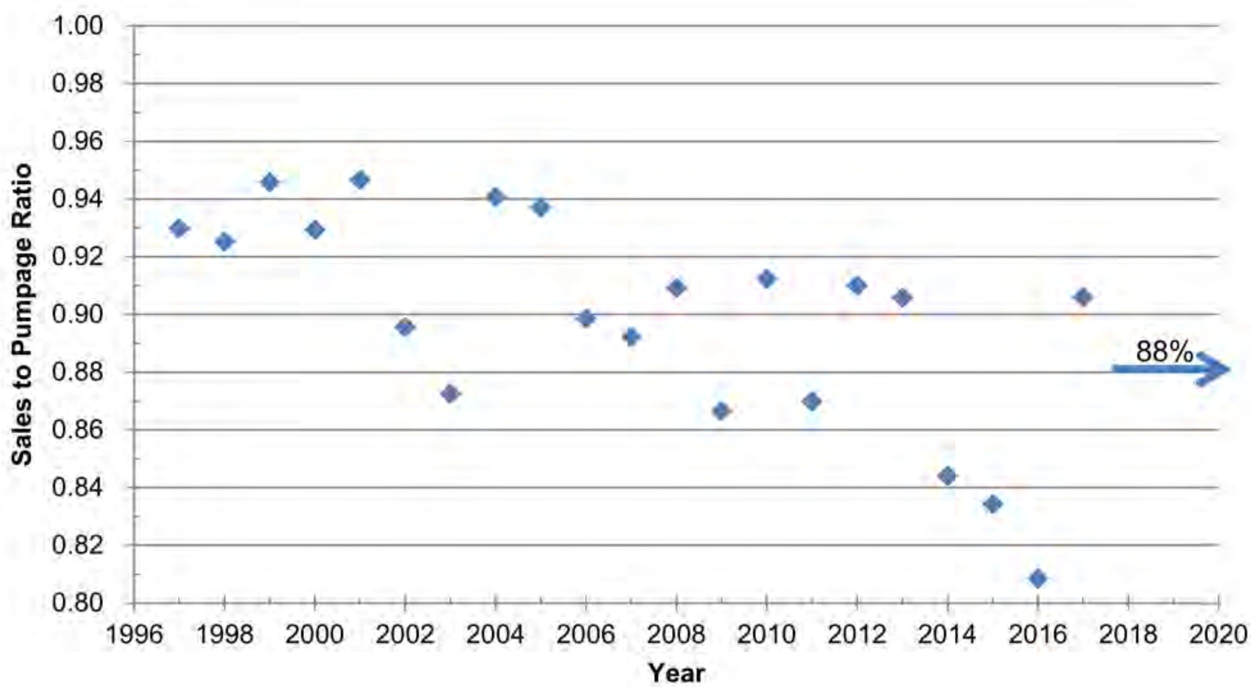


Figure 3.04-1 Sales to Pumpage Ratio

C. Maximum to Average Day Ratio

Figure 3.04-2 presents maximum day to average day demand ratios from 1997 to 2017. The values range from 1.59 to 2.83, with 2.83 occurring in 2017 because of an abnormally high maximum day caused by weather conditions. Ten of the data points have a value higher than 2.0, with two ratios exceeding 2.5. The 2017 ratio can be considered an outlier based on combining a near record maximum day demand with a year of reduced annual sales. Based on this historic data, a maximum to average day ratio of 2.5 will be used to forecast future maximum day demands. This ratio remains unchanged from the 2013 Study.

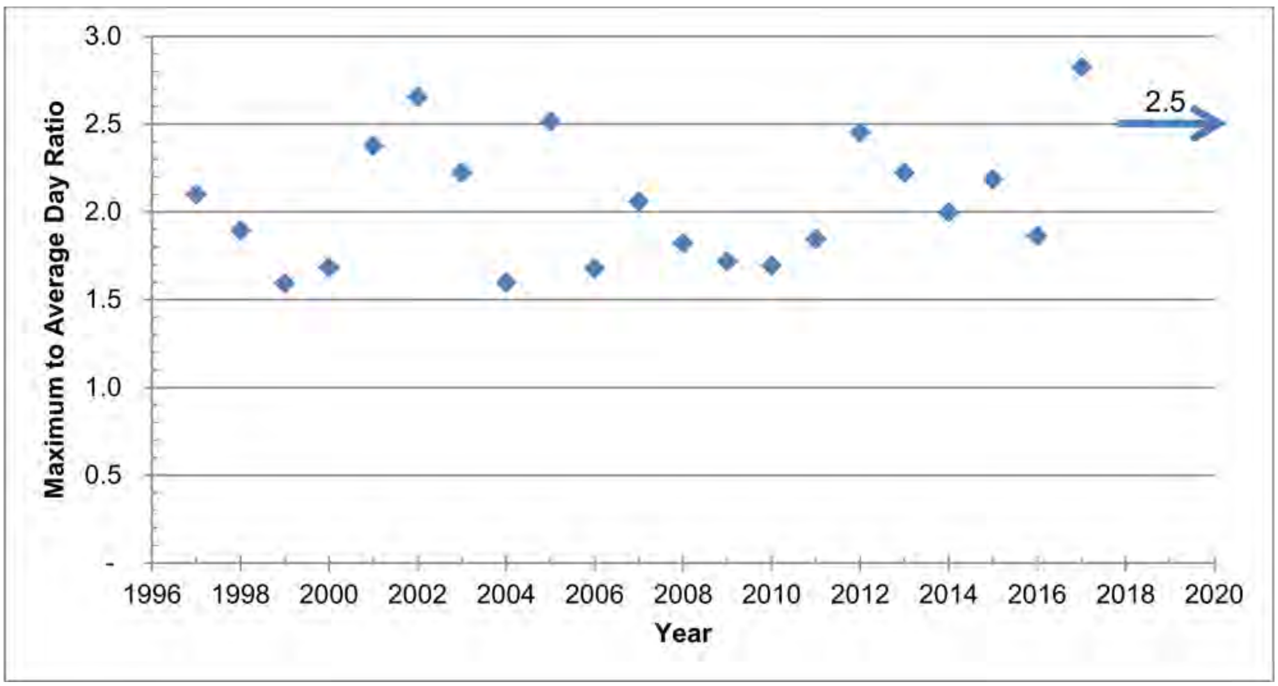


Figure 3.04-2 Maximum Day to Average Day Demand Ratios

D. Water Sales per Capita

Figure 3.04-3 presents total daily water sales per capita since from 1997 to 2017. Historic data continues to show a decreasing trend in per capita water usage since 2005 with the lowest sales per capita occurring in the past five years. A continued and long-term decline in per capita sales is not likely, although it is unknown when the decline will level out. Therefore, a value of 85 gallons per capita per day (gcd), or the average usage from the past five years will be used. This value is lower than the 100 gcd used for the 2013 Study. The general trend in Figure 3.04-3 follows a similar trend in residential sales. Commercial, industrial, and public sales have all been slightly declining similar to residential.

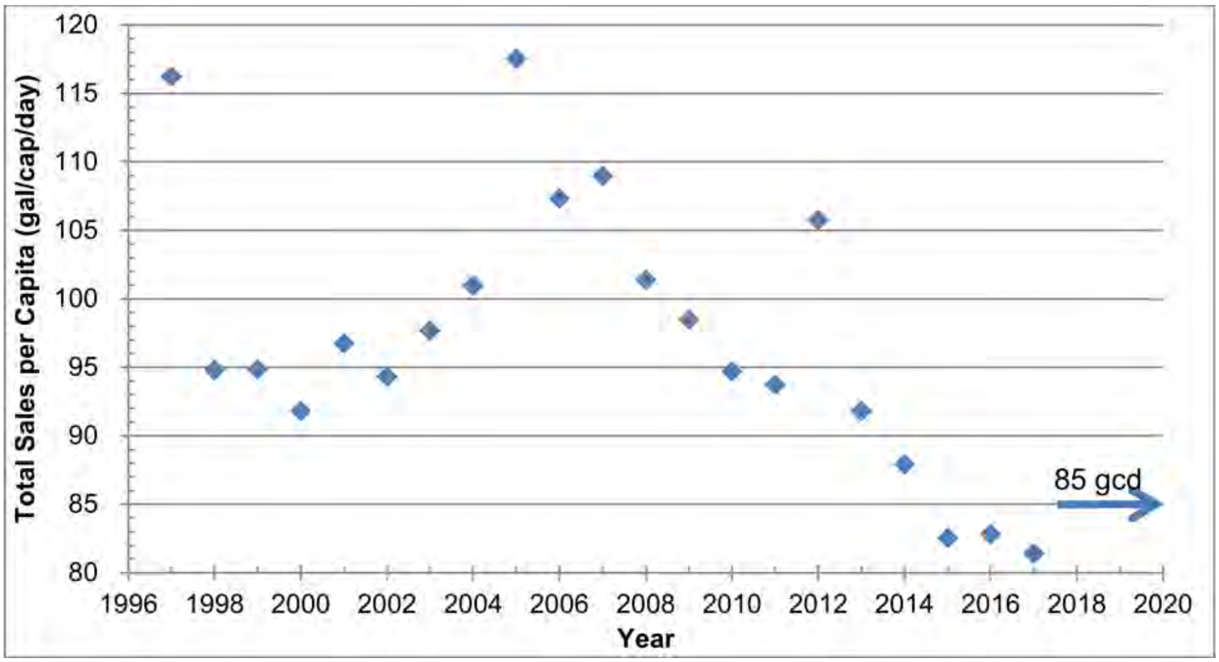


Figure 3.04-3 Per Capita Sales

3.05 2020 PROJECTED DEMANDS

Demand estimates were calculated using the water use trends developed in the previous sections. The projected 2020 and 2030 demands will be used in Section 4 where demands will be compared to available supply.

A. 2020 Average Day

The projected 2020 average day pumpage was calculated by multiplying the design population of 16,013 by the projected total per capita per day sales (85 gcd) and dividing by the corresponding sales to pumpage ratio (88 percent). The estimated average day pumpage is approximately 1,547,000 gallons per day (gpd), or 1,074 gpm.

B. 2020 Maximum Day

1. Domestic

The 2020 maximum day pumpage is estimated to be 3,867,500 gpd and is calculated by multiplying the maximum to average day ratio of 2.5 by the 2020 average day pumpage. This is equal to a demand rate of 2,686 gpm. The system should be capable of satisfying the maximum day demand with firm well supply.

2. Domestic Plus Fire

In order to maintain consistency with previous planning documents and to use the potential maximum credit provided by the Insurance Services Office (ISO), a fire flow of 3,500 gpm for a duration of 3 hours will be assumed.

The total volume of water required to fight a fire on the 2013 maximum day becomes:

Domestic Maximum Day	3,867,500 gallons
Fire (3 hours at 3,500 gpm)	630,000 gallons
Total	4,497,500 gallons

Water for firefighting demands can come from a combination of excess well capacity and water storage facilities.

3.06 2030 PROJECTED DEMANDS

A. 2030 Average Day

The projected 2030 average day pumpage was calculated by multiplying the design population of 19,693 by the projected total per capita per day sales (85 gcd) and dividing by the corresponding sales to pumpage ratio (88 percent). The estimated average day pumpage is approximately 1,902,000 gpd, or 1,321 gpm.

B. 2030 Maximum Day

1. Domestic

The 2030 maximum day pumpage is estimated to be 4,755,000 gpd by applying the maximum to average day ratio of 2.5 to the 2030 average day pumpage. This is equal to a demand rate of 3,302 gpm. Figure 3.06-1 presents the projected average and maximum day demands through 2030.

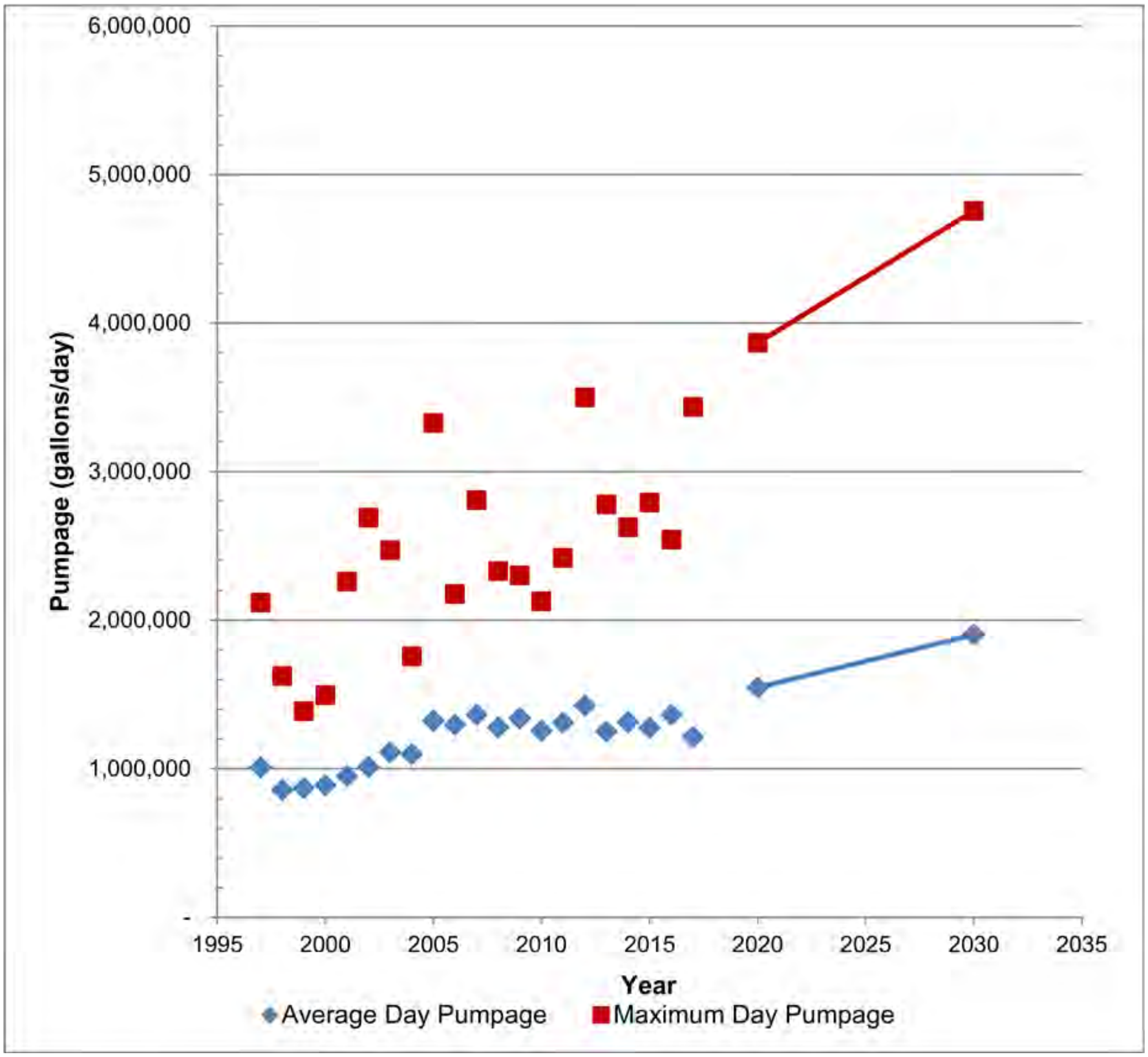


Figure 3.06-1 Projected Average and Maximum Day Demands

2. Domestic Plus Fire

A fire flow demand of 3,500 gpm for a duration of 3 hours was used for calculation purposes. Basic fire flow requirements are based on the amount of water the Village should be able to supply on the day of maximum domestic demand.

The total volume of water required to fight a fire on the 2030 maximum day becomes:

Domestic Maximum Day	4,755,000 gallons
Fire (3 hours at 3,500 gpm)	630,000 gallons
Total	5,385,000 gallons

Water for firefighting demands can come from a combination of excess well capacity and water storage facilities.

C. Pumpage Comparisons

Table 3.06-1 compares the average day, maximum day, and maximum day plus fire pumpage between the 2013 and 2018 reports for the projected 2030 demands. The projected 2030 average day pumpage was reduced by 286,000 gallons and 2030 maximum day and 2030 maximum day plus fire was reduced by 715,000 gallons from the 2013 to the 2018 report.

Although the population and the maximum to average day ratio remained unchanged and the sales to pumpage ratio decreased, the daily pumpage is projected to be less than what it was projected in the 2013 Study. This is primarily because the estimated water usage (sales per capita) decreased from 100 gcd to 85 gcd. This can be seen in the decrease of usage in the industrial and commercial customers.

	2013 Study	**2018 Study**	**Difference**
2030 Average Day	2,188,000	1,902,000	-286,000
2030 Maximum Day	5,470,000	4,755,000	-715,000
2030 Maximum Day Plus Fire	6,100,000	5,385,000	-715,000

Table 3.06-1 Pumpage Comparison (gallons)

3.07 AREAS OF EXPECTED DEVELOPMENT

In 2017, the Village and the Town of Westport adopted a new Comprehensive Plan that includes future land use maps. Similar to the 2013 Future Urban Development Area (FUDA) Study, the comprehensive plan shows potential residential development in yellow.


Legend
Elevation > 960
Pipe Diameter
6"
8"
10"
12"
24"
Legend
Potential Storage Location
Potential Well Location
Existing Well
Existing Tank
Ultimate Service Area
Northern Serviceable Area
Southeastern Serviceable Area
Southwestern Serviceable Area
0 375 750 1,500
Feet
WATER SYSTEM
SERVICE AREAS
WATER SYSTEM STUDY UPDATE
WAUNAKEE UTILITIES
VILLAGE OF WAUNAKEE
DANE COUNTY, WISCONSIN
STRAND ASSOCIATES
FIGURE 3.07-1

A. Review of 2013 Areas of Development

There were four properties described in the 2013 Study that were identified by the Village staff as areas of potential future development. These areas were examined based on the land use shown in the 2013 FUDA map and compared to the map shown in the 2017 Comprehensive Plan.

1. Tierney Quarry Development

This area was previously shown in brown in the FUDA map as redevelopment, but is now shown in yellow in the 2017 Comprehensive Plan as an area of residential development. In addition, there is land area that borders this development that is also shown in yellow in the 2017 Comprehensive Plan. This area should continue to be considered an area of future residential development.

2. Bruenig Property

This area was previously shown as proposed County Natural Resource Area in the FUDA 2013 map. As discussed in the 2013 Study, this property was identified as a future residential development area. The 2017 Comprehensive Plan map shows this area in yellow as an area of residential development. In addition, this area is also shown surrounded by neighboring future residential development areas and should continue to be considered an area of future residential development.

3. Easy Street Property

This area was not shown in the 2013 FUDA map; however, it was identified as an area for future residential development. The 2017 Comprehensive Plan shows this area as future residential development and should continue to be considered an area of future residential development.

4. Kennedy Drive Property

Although previously identified as a potential residential development and unlabeled on the 2013 FUDA map, this area is now labeled as rural preservation in the 2017 Comprehensive Plan and is also within the Joint Planning Area between the Village and the Town of Westport. A change in zoning would be required to allow for this area to develop for residential use.

B. Updated Areas of Future Growth–Meffert Road Area

The corridor along Meffert Road, east and west of CTH Q includes lands developable for residential and commercial use. Water supply to this area can be accomplished by looping water main from the existing 12-inch main along Water Wheel Drive. Additional connections to the north on the west side of CTH Q can be considered as development progresses between Meffert Road and Woodland Drive.

SECTION 4
ADDITIONAL REQUIRED CAPACITY

4.01 GENERAL

This section presents the additional required capacity analysis based on the updated water demands developed in the previous section. The same method used to determine the amount of storage capacity in the 2013 Study was used to provide the updated capacity evaluation and is explained below.

4.02 2020 CAPACITY EVALUATION

A. 2020 Maximum Day

The total pumpage on the 2020 maximum day is estimated to be 3,867,500 gpd (2,686 gpm). The existing firm well capacity as of 2013, is 5,789,000 gpd (4,020 gpm). The Village has a surplus well supply of 1,334 gpm and no additional well capacity is required at this time.

B. 2020 Maximum Day Plus Fire

The total amount of water available to satisfy the maximum day plus fire demand is equal to the firm well capacity plus the water available from storage.

The flow available from storage is equal to the volume of water remaining after accounting for peak hourly demands and normal water level fluctuations. The volume needed for these daily water level variations is assumed to be equivalent to 20 percent of the maximum day demand volume. For 2020 demands, this equates to 773,500 gallons, leaving 226,500 gallons of elevated storage.

All 300,000 gallons of storage at the Well No. 3 reservoir is assumed to be available during the fire event. This volume can be pumped at an effective rate equal to the total booster pump capacity minus the well pump capacity (2,500 gpm - 1,300 gpm = 1,200 gpm).

Section 3.05 discusses the 2020 domestic maximum day plus fire demand conditions for the system. A demand rate of 6,186 gpm (2,686 gpm domestic demand plus 3,500 gpm fire demand) for 3 hours must be satisfied to provide the necessary fire protection. Because a fire can start at any time during the day, the expected domestic use must be taken into account when calculating available capacity.

Maximum Day Demand	- 2,686 gpm
Fire Demand	- 3,500 gpm
Firm Well Capacity	4,020 gpm
Elevated Tank Capacity*	1,258 gpm
Well No. 3 Reservoir Capacity	1,200 gpm
Total	292 gpm

*Storage capacity = (1,000,000 gallons - 773,500 gallons)/180 minutes

During a 180-minute fire event, the system is projected to have a surplus capacity of 292 gpm or approximately 52,560 gallons. Therefore, no additional storage is needed to meet the projected 2020 maximum day plus fire demand.

4.03 2030 CAPACITY EVALUATION

A. 2030 Maximum Day

The total pumpage on the maximum day in 2030 is estimated to be 4,755,000 gpd (3,302 gpm). The existing firm well capacity, is 5,789,000 gpd (4,020 gpm). The existing firm well capacity exceeds the 2030 projected maximum day domestic demands. The Village has a 2030 surplus well supply of 718 gpm and no additional well capacity is required.

B. 2030 Maximum Day Plus Fire

Section 3.06 discusses the 2030 domestic maximum day plus fire demand conditions for the Village. A demand rate of 6,802 gpm (3,302 gpm domestic demand plus 3,500 gpm fire demand) for 3 hours must be satisfied to provide the necessary fire protection. Because a fire can start at any time during the day, the expected domestic use must be taken into account when calculating available capacity.

The flow available from storage is equal to the volume of water remaining after accounting for peak hourly demands and normal water level fluctuations. The volume needed for these daily water level variations is assumed to be equivalent to 20 percent of the maximum day demand volume. For 2030 demands, this equates to 951,000 gallons, leaving 49,000 gallons of elevated storage.

Maximum Day Demand	- 3,302 gpm
Fire Demand	- 3,500 gpm
Firm Well Capacity	4,020 gpm
Elevated Tank Capacity*	272 gpm
Well No. 3 Reservoir Capacity	1,200 gpm
Total	-1,310 gpm

*Storage capacity = (1,000,000 gallons - 951,000 gallons)/180 minutes

During a 180-minute fire event, the system is projected to have a shortage of 1,310 gpm or approximately 235,800 gallons of storage. Therefore, additional storage capacity is required to meet the projected 2030 maximum day plus fire demand. The timing and capacity of additional storage is discussed in Section 6.

SECTION 5
HYDRAULIC MODELING

5.01 GENERAL

The existing WaterGEMS V8i water system model was updated to reflect new water mains and hydrants installed during the time passed since the model was last updated as part of the 2013 Study. The model was updated with owner-provided geographic information system (GIS) database information received in February 2018. Elevations assigned to the junctions and hydrants were developed using 2009 LIDAR 1-foot contours. The water system was updated to be modeled using 2020 projected maximum day demands based on the findings summarized in this report. The model was not re-calibrated as part of this water system study update. A re-calibration of the water model is typically recommended every five years or prior to any major system improvements to ensure accurate simulations.

5.02 EXISTING SYSTEM

The existing water system was modeled based on 2013 maximum day demands. The model was used to confirm adequate operating pressure and to identify any areas of low fire flow.

A. Operating Pressure

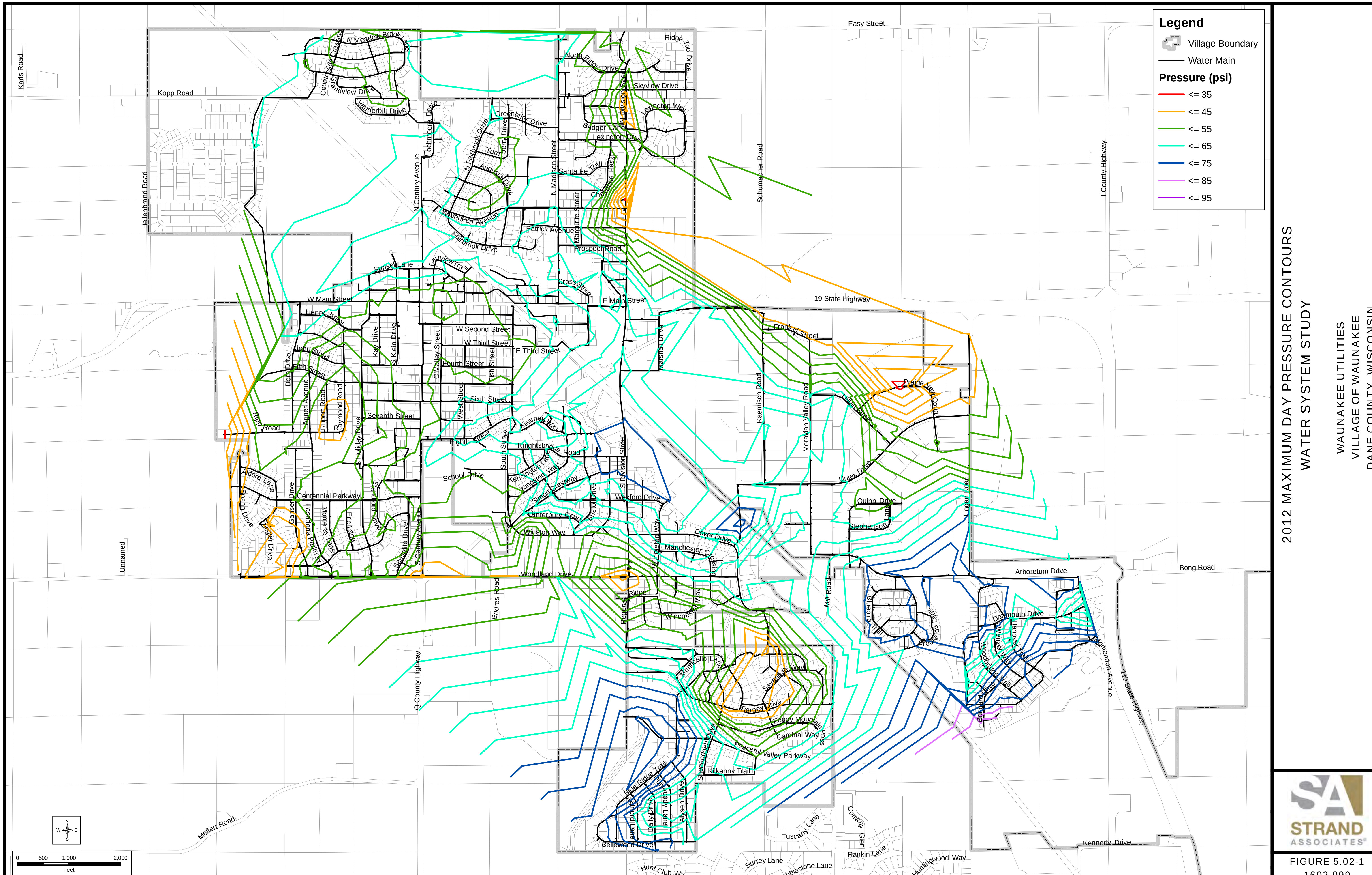
The operating pressure in the main pressure zone was modeled to be between approximately 34 and 82 pounds per square inch (psi). This is nearly within the range of 35 to 100 psi required by Wisconsin Department of Natural Resources (WDNR). Two of the three areas experiencing pressures less than 35 psi are located next to elevated tanks and are caused by high ground elevations. The other area is at the northern dead end of Prairie View Court in the industrial park. This low pressure is also caused by higher ground elevations. Figure 5.02-1 shows contours of pressure generated by the computer model.

B. Fire Flow

The model was used to simulate available fire flows throughout the main pressure zone. The modeled available fire flow based on a minimum system pressure of 20 psi ranged from approximately 550 to 7,700 gpm. Figure 5.02-2 shows contours of available fire flow generated by the computer model.

The lowest available fire flows are found in the Southbridge development generally bounded by Woodland Drive to the east and Blue Ridge Trail to the north. This area is fed by one 8-inch-diameter water main that extends from the intersection of Emerald Grove Lane and Woodland Drive to Blue Ridge Trail. The only other connection in this area is the 8-inch water main feeding the boosted zone near Blue Ridge Trail, which is normally isolated from the remainder of the system. The modeled fire flow in the Southbridge development ranges from approximately 550 to 800 gpm. While 500 gpm is recognized as the minimum recommended fire flow to a residential area, distribution system improvements are recommended to provide a redundant connection and increased fire flow to the area.

The only other areas of low fire flow were found near dead end mains. One such dead end main is located on the south side of Main Street between Baker Street and Water Street. Another dead end main is located in the high school parking lot. These mains should be looped where feasible, but are not considered critical deficiencies.



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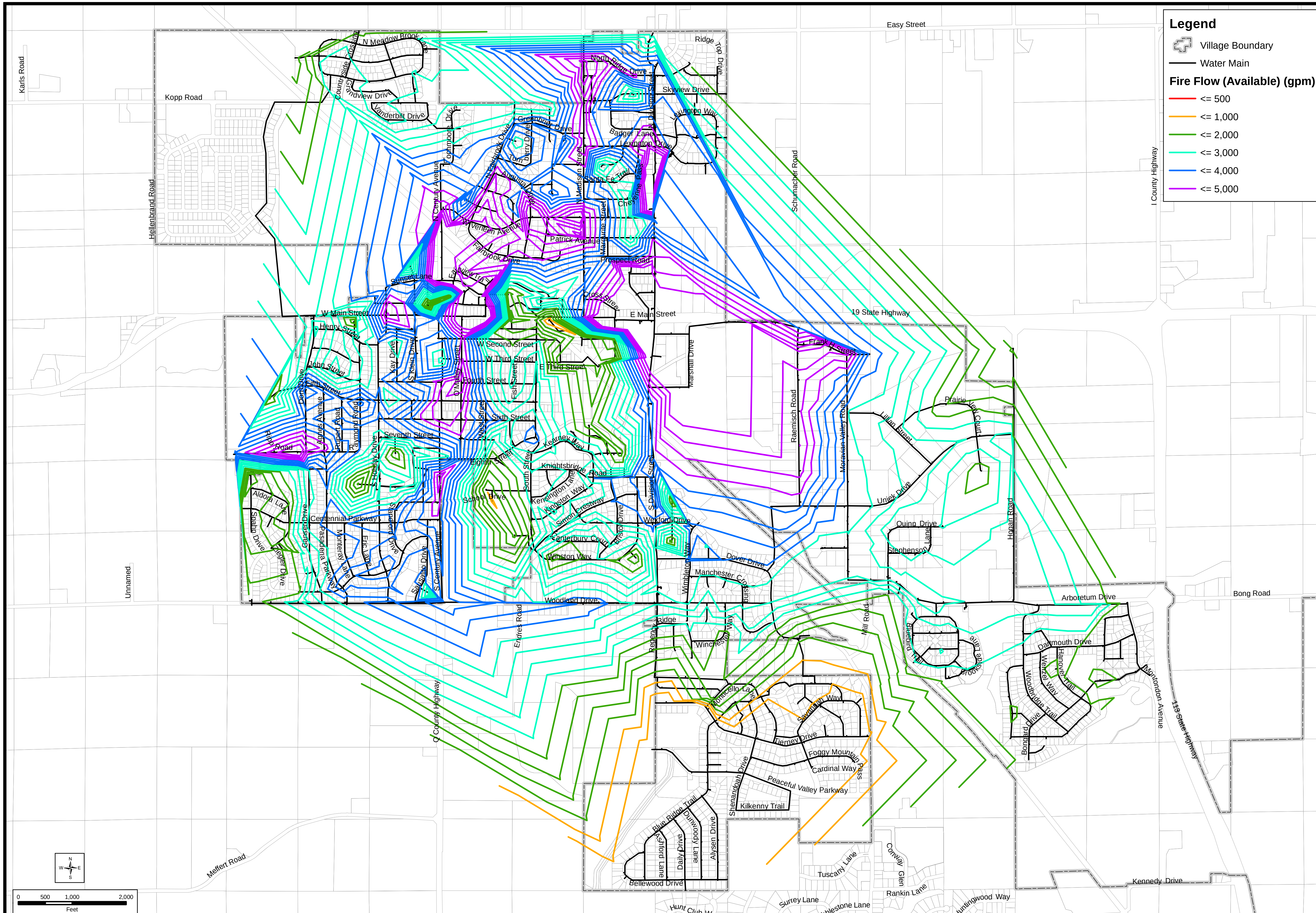

2012 MAXIMUM DAY AVAILABLE FIRE FLOW CONTOURS
WATER SYSTEM STUDY

WAUNAKEE UTILITIES
VILLAGE OF WAUNAKEE
DANE COUNTY, WISCONSIN

STRAND ASSOCIATES®

FIGURE 5.02-2
1602.099

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5.03 AREAS OF FUTURE DEVELOPMENT

The areas of future development discussed in Section 4 were evaluated based on service elevation and hydraulic modeling. To be consistent with previous report sections, the analysis is separated between the areas feasibly served by the existing sanitary sewer system and the ultimate service area.

A. Topographic Review

A review of topographic elevations was conducted to determine which areas of future development can be served by the existing system's main pressure zone. Based on the elevated storage overflow elevation of 1,063 feet mean sea level (MSL), static system pressures of 35 psi and 100 psi correspond to service elevations of approximately 960 feet MSL and 830 feet MSL, respectively. Elevations in this range can be effectively served by the existing main pressure zone.

There are no ground elevations less than 830 feet MSL within the serviceable or ultimate service areas, therefore operating pressures greater than 100 psi are not expected.

There are many areas of elevation higher than 960 feet MSL within the potential future service areas which will require separate pressure zones if developed. Figure 3.07-1 is a map showing the existing service area, potential future service areas, and elevations exceeding 960 feet MSL. The hatching on the map denotes areas that will require separate pressure zone(s) to provide the required service pressure.

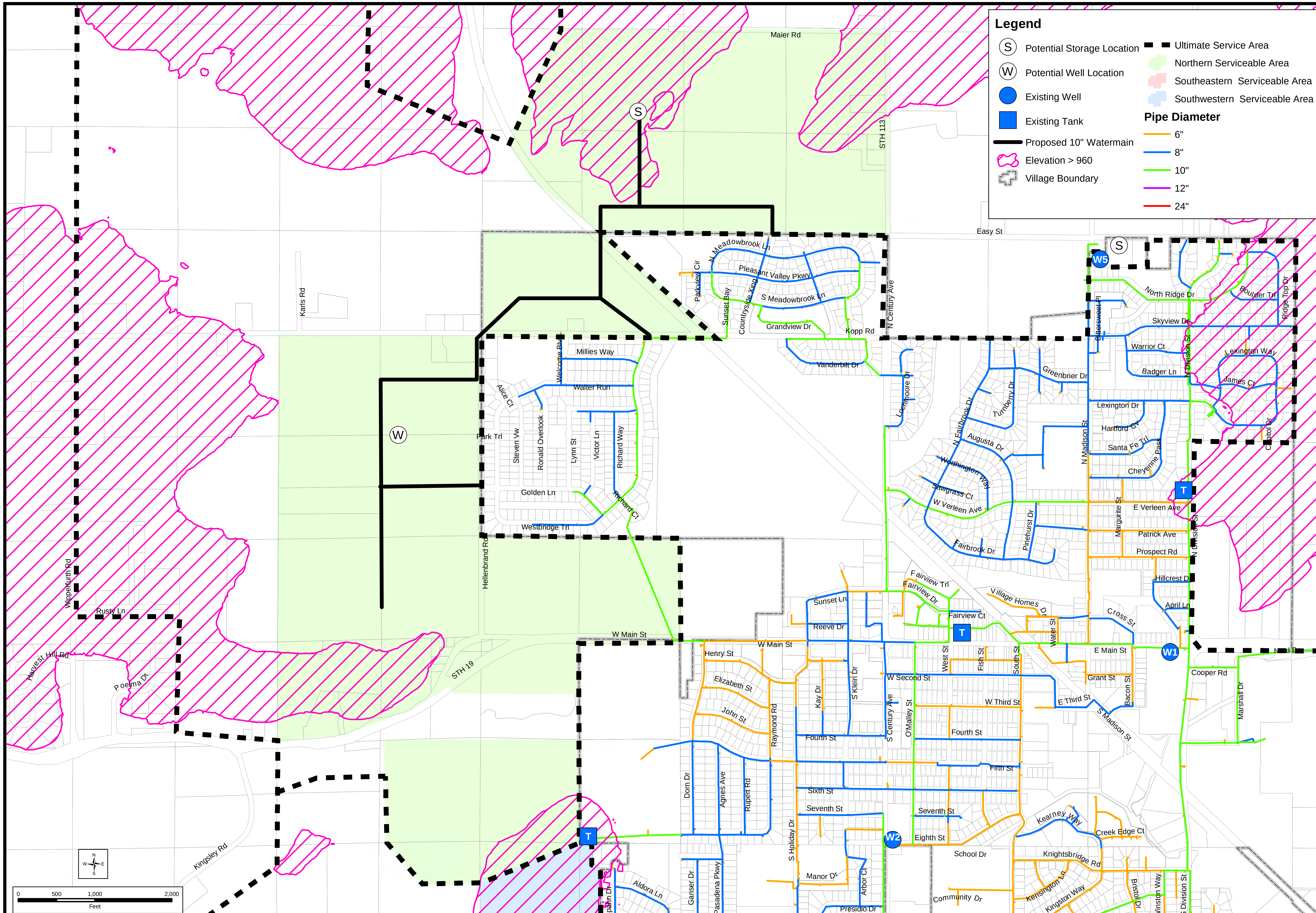
B. Hydraulic Modeling

The hydraulic model was used to simulate operating conditions in areas of potential development. The water demands for each serviceable area presented in Section 4 were entered into the model to simulate maximum day demands in each region. Water main sizes were modeled to determine appropriate pipe diameter based on fire flow and future well and storage tank locations.

1. Northern Region

As discussed in Section 4, the northern region, which is expected to be primarily residential with some small areas of commercial in the southern part of the region, is projected to add 1,387,000 gallons of water demand on a maximum day. Figure 5.03-1 shows the existing ground elevations in the region. Elevations above 960 feet are found along Maier Road and Highway 113, and along Highway 19 in the southwest part of the region. Most development within the northern region can be served by the existing water system's main pressure zone.

Figure 5.03-1 also shows the proposed water main sizes to feed future development. A large portion of this region includes the Westbridge development, which is currently under construction. A 10-inch water main recently installed through the development will provide a critical redundant connection between Kopp Road to the north and West Main Street to the south. This 10-inch main will provide the ability to extend water service to most of the northern region. A network of 10-inch water main will provide adequate water service to the region. The 10-inch water main should be tied into the 10-inch main along Kopp Road. A second connection is recommended at the north end of Countryside Crossing. Supply for the southern-most serviceable area within the northern



NORTHERN REGION SERVICEABLE AREA
WATER SYSTEM STUDY UPDATE

WAUNAKEE UTILITIES
VILLAGE OF WAUNAKEE
DANE COUNTY, WISCONSIN

STRAND ASSOCIATES®

FIGURE 5.03-1
1602.124

Path: S:\MAD\1600--1699\1602\124\Drawings\GIS\Figures\Northern Region 11x17.mxd User: danc Date: 7/17/2018 Time: 2:04:04 PM region (south of Sixmile Creek) can likely be provided by extending a 10-inch water main west from the Ripp Park tank. The exact location and routing of future mains will be dictated by the layouts of proposed development.

Ground elevations in the northern region are favorable for location of a future elevated storage tank. Elevations near 960 feet MSL just west of State Highway 113 and south of Maier Road would allow construction of a tank less than 100 feet tall while serving the main pressure zone. If a tank is desired in this region, the Utility should consider securing a site as development occurs.

The water model was used to simulate operating pressure and fire flow in the region based on the 10-inch water main and connections to the existing system. The operating pressure ranged from approximately 36 psi to 54 psi. The available fire flow ranged from approximately 710 gpm to 1,120 gpm. This is considered adequate for a primarily residential area. With a new elevated tank located where shown in Figure 5.03-1, the operating pressures ranged from approximately 40 psi to 59 psi. The available fire flow ranged from approximately 1,110 gpm to 4,600 gpm.

2. Southwestern Region

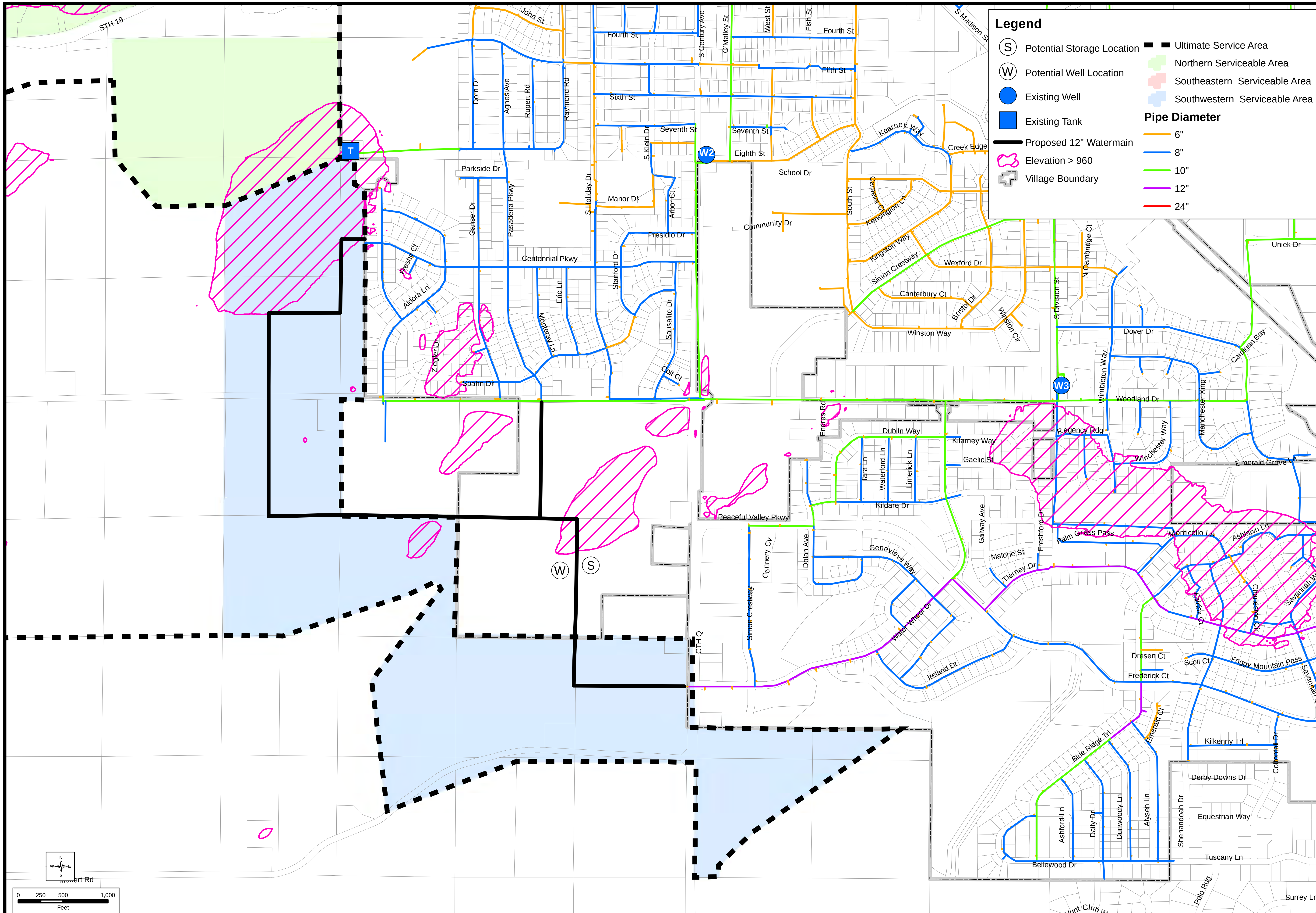
The southwestern region is expected to include mostly residential development with some commercial areas. The region is expected to add 1,216,100 gallons of maximum day demand to the system.

Most of the region could be served by the existing main pressure zone with the exception of two areas. There is an area of high elevation southwest of Ripp Park that would require a locally boosted zone to maintain pressure above 35 psi. There is also a high point in the central part of the region where elevations reach 984 feet MSL.

Figure 5.03-2 shows the elevations in the region along with the recommended water main connections and sizing. Water main extensions needed to serve the region should be tied into existing 10-inch and 12-inch water main where shown. This will provide effective looping and redundancy to the area.

The hydraulic model was used to simulate operating pressure and fire flow in the region. Operating pressures ranged from approximately 37 psi to 67 psi. The lower pressure is caused by the high ground elevations noted on Figure 5.03-2. The available fire flow ranged from approximately 1,530 gpm to 2,620 gpm.

This area can be effectively served by the existing water supply and storage facilities. However, the area offers good locations for future well and storage sites. A new well in this area could be located more than one mile from any existing Utility wells. An elevated tank is not required to serve this area, but if a new tank is desired, the higher elevations noted would be favorable.



WAUNAKEE UTILITIES
VILLAGE OF WAUNAKEE
DANE COUNTY, WISCONSIN

SOUTHWESTERN REGION SERVICEABLE AREA
WATER SYSTEM STUDY UPDATE

STRAND ASSOCIATES®

FIGURE 5.03-2
1602.124

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3. Southeastern Region

Based on land use mapping, the southeastern region is expected to include a mixture of residential, commercial and industrial development. In general, residential development is expected along Bong Road. Commercial and industrial development is expected along the Highway 113 corridor and west toward the existing industrial park. In total, the region would add approximately 949,500 gallons to the maximum day water demand.

Figure 5.03-3 shows the ground elevations in the region. The southeast part of the region has elevations exceeding 960 feet MSL, which would require a separate pressure zone. The northwest part of this region can be served by the existing main pressure zone.

Based on discussions with Utility and Village staff, the only area of this region likely to see development in the foreseeable future is the area west of Highway 113.

At a minimum, water main extensions to serve the region should be tied into existing 12-inch water main at Arboretum Drive and Uniek Drive as shown in Figure 5.03-3.

The hydraulic model was used to simulate operating pressure and fire flow in the region. The operating pressures ranged from approximately 41 psi to 62 psi and the available fire flow ranged from approximately 1,910 gpm to 2,550 gpm.

5.04 ULTIMATE SERVICE AREA

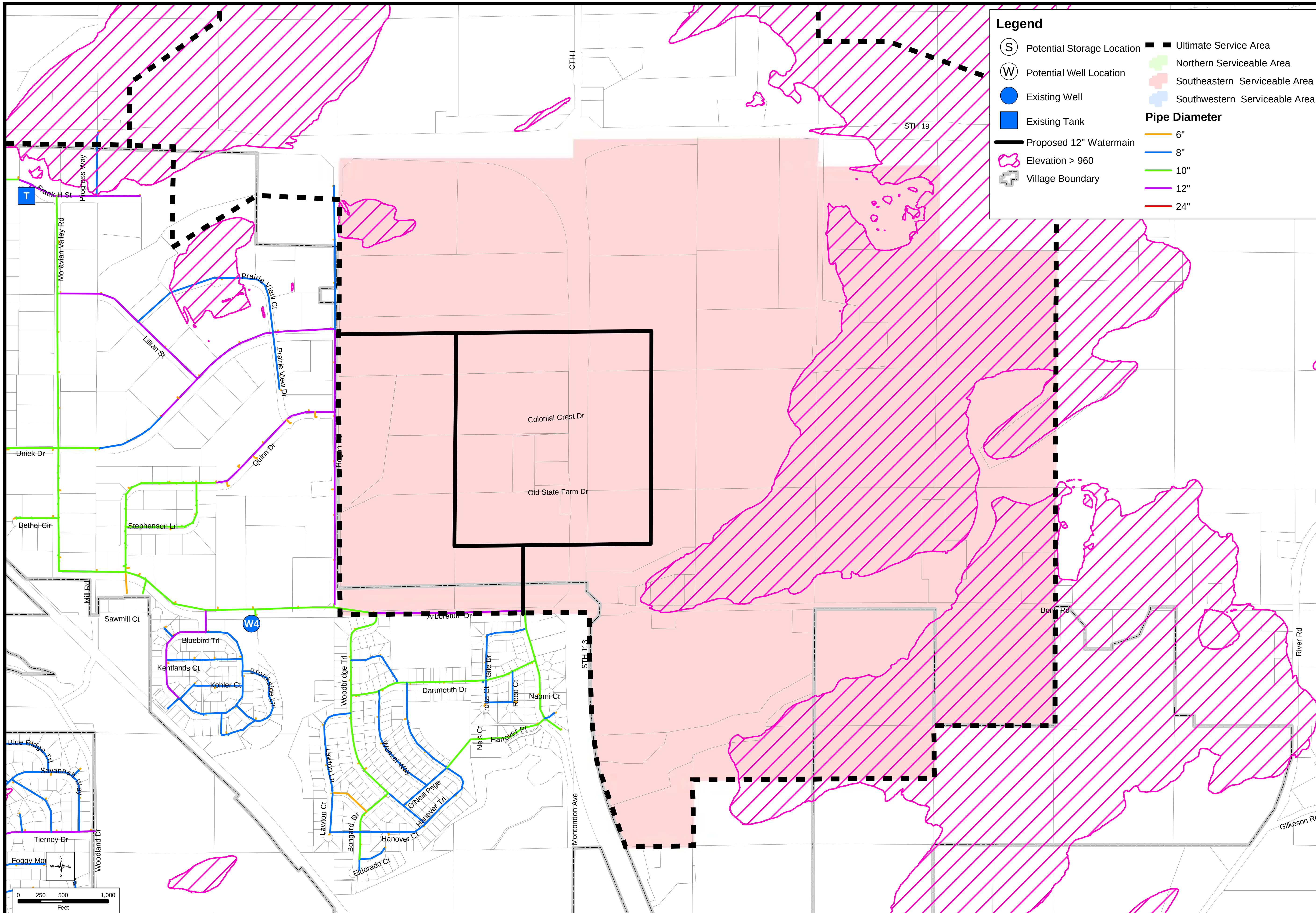
Detailed hydraulic modeling of the ultimate service area would not provide meaningful information because of the uncertain timing of development and land uses. However, an evaluation of service elevations and projected water demands can be used to determine, in general, the extent of improvements needed to serve the areas.

Figure 3.07-1 shows the ultimate service area along with elevations exceeding 960 feet MSL. These areas will require separate pressure zones to maintain acceptable service pressure.

A. Northern Region

The northern region of the ultimate service area is projected to add 7.2 million gallons of maximum day demand, which is equivalent to a rate of 5,000 gpm. Five new wells would be needed to satisfy the demands associated with the region.

As shown on Figure 3.07-1, many parts of the northern region would require creation of a separate pressure zone to provide acceptable service pressure. This includes the area immediately west of the existing Lexington boosted zone. If development occurs in this area, the booster station will need to be upgraded or another booster station built to serve this area.



SOUTHEASTERN REGION SERVICEABLE AREA
WATER SYSTEM STUDY UPDATE

WAUNAKEE UTILITIES
VILLAGE OF WAUNAKEE
DANE COUNTY, WISCONSIN

STRAND ASSOCIATES®

FIGURE 5.03-3
1602.124

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B. Southwestern Region

Much of the southwestern region includes areas that can be feasibly served by the existing sanitary sewer system. Water service to this area is discussed above. The remaining area within the ultimate service area, but outside the current serviceable area is shown on Figure 3.07-1. This entire area shown can be served by the existing main pressure zone. As development proceeds in this region, a network of 10-inch water main should be extended in the ultimate service area. The Utility should secure a site for a new well in this area when development occurs.

C. Southeastern Region

Approximately one half of the southeastern region ultimate service area includes land that can be served by the existing sanitary sewer system as discussed above. The remaining area within the ultimate service area includes ground elevations above 960 feet MSL. Figure 3.07-1 shows the areas that will require separate pressure zones.

5.05 AREAS OF EXPECTED DEVELOPMENT

A. Tierney Quarry Development

This area is located immediately east of Ridge Top Drive and Lexington Way in the northeast part of the Village. As discussed in Section 3, the maximum day demand attributed to this 80-acre area is 137,500 gpd, or 95 gpm.

This entire area is located at elevations above 960 feet MSL and will need to be served by the Lexington booster station.

B. Breunig Property

This property is located south of the quarry and is bound by Highway 19 to the south, Division Street to west and Schumacher Road to the east. As discussed in Section 3, the maximum day demand attributed to this area is 275,000 gpd, or 190 gpm.

A majority of this area is located at elevations above 960 feet MSL and will need to be served by the Lexington booster station or a new booster station. The southern portion of this area near Highway 19 can be served by the main pressure zone.

C. Easy Street Property

There is an area north of Easy Street that is expected to develop as residential area. The area is generally bound by Easy Street to the south, environmental corridor to the north and west, and Schumacher Road to the east. As discussed in Section 3, the maximum day demand attributed to this area is 299,200 gpd, or 210 gpm.

Most of this area can be served by the main pressure zone. A network of 10-inch water main should be extended north from Madison Street to serve this area. If development occurs north of Easy Street and along Schumacher Road, a booster station will be needed to provide acceptable service pressure.

D. Kennedy Drive Property

This area is generally bound by Hanover Trail and Hanover Place to the north, Highway 113 to the east, Kennedy Drive to the south and the railroad to the west. As discussed in Section 3, the maximum day demand attributed to this area is 400,000 gpd, or 280 gpm.

Based on ground elevations, this area can be served from the main pressure zone. Once the land use in this area is determined, water main extensions should be sized accordingly.

E. Meffert Road Area

Water supply to this area can be accomplished by looping water main from the existing 12-inch main along Water Wheel Drive. Additional connections to the north on the west side of CTH Q can be considered as development progresses between Meffert Road and Woodland Drive.

SECTION 6
CONCLUSIONS AND RECOMMENDATIONS

This section summarizes the conclusions and recommendations of this 2018 Study. A list of improvements and anticipated costs is provided along with a discussion of implementation timing.

6.01 CONCLUSIONS

Despite continued growth of the Village's water service area, average day water use over the last 13 years has essentially remained unchanged. This is consistent with most other communities in the region who have seen level or declining water use trends even as populations increase. Maximum day water use is trending upward, likely because of increased population and ongoing expansion of the water system.

The Utility continues to develop its water supply infrastructure with future growth in mind. Annual water main replacements improve aging areas of the system, and areas of development provide opportunities to loop new water main to existing parts of the system.

Based on population projections and water use trends, the system has a well supply surplus of 718 gpm out to the 2030 design year. The surplus will allow significant growth of the service area before another well is needed.

Based on the 2020 design year, there is a small surplus in water storage volume. By 2030 there is a projected storage deficit of approximately 235,800 gallons. The Utility should begin planning for a new water storage facility by the year 2020 and have the facility operational by 2023.

6.02 RECOMMENDATIONS

A. Well Supply

While a new well may not be needed until after 2030, the Utility should secure a well site as development continues. With a well site secured, the Utility can proceed with Well No. 6 whenever the need arises.

B. Storage

The Utility should begin planning for a new water storage facility in 2020. A new 400,000-gallon storage facility should be online by 2023. The location of the new storage facility will be dictated by the location of future growth and available sites. One favorable site is located in the southwest region south of Woodland Drive and west of County Road Q at elevation above 960 feet MSL. Another is the existing Well No. 5 site, which was designed to accommodate a future reservoir and booster pumping facility. A storage sizing and siting evaluation should be conducted before a final site selection is made.

C. Distribution System

As development continues, the Utility should install (or require developers to install) 10-inch and 12-inch feeder mains through each area to be developed. The actual location of these mains will be dictated by the layout of each development and should be reviewed during the planning phase.

The Utility should also continue to replace aged, undersized mains along with the Village's annual street and utility projects. No areas of critical deficiency were found within the existing system that give one area priority over another.

6.03 IMPLEMENTATION AND COST

Table 6.03-1 presents the system improvement recommendations along with opinions of probable cost in 2018 dollars.

Year	Improvement	Cost Opinion
2020	Storage Facility Sizing and Siting Evaluation	$15,000
2022	Construct 400,000-gallon Storage Facility	$1,500,000
2025	Well Siting Evaluation	$15,000
2026	Site Acquisition for Well No. 6	$75,000
2028	Drill Well No. 6	$400,000
2030	Construct Well No. 6 Facility	$1,300,000

Table 6.03-1 Recommendations and Cost

APPENDIX
MARCH 21, 2017 LETTER

Strand Associates, Inc.®
910 West Wingra Drive
Madison, WI 53715
(P) 608-251-4843
(F) 608-251-8655

March 21, 2017

Mr. Tim Herlitzka
Waunakee Utilities
322 Moravian Valley Road
Waunakee, WI 53597

Re: Water Storage Evaluation

Dear Tim,

This letter presents a brief evaluation of water storage needs intended to supplement the Water System Study report completed in April 2013. Based on our conversation, Waunakee Utilities (Utility) wishes to proactively plan for the future water storage project which was identified in the 2013 study. As discussed below, implementing additional storage volume can be delayed several years.

Background

Water storage is needed to satisfy demands during periods where water use is greater than the well pumping capacity. These periods typically include normal daily fluctuations in demand and abnormally high water use during warm weather and firefighting events. The Village of Waunakee (Village) water system currently includes 1.3 million gallons of storage volume.

The 2013 study considered design years of 2013 and 2030 in addition to the ultimate build out of the service area. At the time of the report, the study found the system to have a small surplus in storage volume of approximately 170,500 gallons for the 2013 design year. Based on assumptions detailed in the report, the system was projected to begin seeing a storage deficit in the year 2018 with the deficit growing to approximately 374,400 gallons by the year 2030.

Recent Data

The Village has seen a steady increase in the number of residential water service connections as development continues. The following table shows the reported number of residential water service connections since 2010.

Year	Residential Water Service Connections
2010	3,807
2011	3,850
2012	3,901
2013	3,979
2014	4,134
2015	4,255
2016	4,367

In 2010, the census population of the Village was 12,097 and there were 3,807 residential service connections. For estimating purposes, this yields approximately 3.18 people per residential connection. The 2013 study used an estimated population of 12,622 for the year 2013. The actual number of

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residential connections for that year was 3,979; giving a factor of 3.17 people per residential connection. Applying this factor to the actual number of residential connections reported in 2016 (4,367 connections) gives an estimated population served of 13,875. This is just slightly below the projection used in the 2013 Water System Study report. See enclosed figure from the 2013 study.

Water pumping data collected by the Utility suggests the actual water pumpage has been somewhat less than the projections in the 2013 report. This is likely due to the conservative nature of the methods used in the report and lower than anticipated water use per capita. The following table supplements Table 3.04-1 in the 2013 report. Sales data for 2016 was not provided.

Year	Annual Pumpage (gal)	Average Day Pumpage (gpd)	Maximum Day Pumpage (gpd)	Average Day Sales (gpd)	Sales to Pumpage Ratio	Maximum to Average Day Ratio
2013	456,563,000	1,250,858	2,779,000	1,133,110	0.91	2.22
2014	480,137,000	1,315,444	2,625,000	1,110,323	0.84	2.00
2015	466,228,000	1,277,337	2,792,000	1,065,734	0.87	2.19
2016	498,400,000	1,365,479	2,543,000	---	---	1.86

Enclosed are several pages from the 2013 report with recent data points shown for reference. Using estimated populations based on residential water services, it appears the water sales per capita continue to trend lower. Using the methodology in the 2013 report and the estimated population based on residential service connections, the calculated (estimated) 2016 average day pumpage would be 1.54 million gallons per day (mgd) versus 1.365 mgd actual reported. The report used a factor of 100 gallons per capita per day (gcd). Recent data suggests actual water sales on the order of 90 gcd.

Summary

The above information suggests the Village is growing at a rate consistent with the projections used in the 2013 Water System Study. While the per capita water use has trended lower over the past several years, there will still be a need for additional storage volume based on design maximum day demand and fire protection needs. However, the timing of additional storage can be delayed beyond the dates listed in the 2013 report.

Based on the recent data added to the 2013 report findings, it appears the system will likely begin to see a storage deficit sometime after the year 2020. A storage deficit in the range of 300,000 gallons to 400,000 gallons is expected by 2035. This does not consider major developments or large industrial water users that would result in water use above the estimates developed in the 2013 Water System Study.

Implementing a new storage facility from initial planning to construction completion is a 2-year process. The 2013 report recommended a storage facility sizing and siting evaluation be completed in 2015 ahead of facility construction in 2017. Because neither of these have been completed, we have adjusted these recommendations as shown below.

Based on continued growth of the Village and potential changes to commercial and industrial water use, the water system study should be updated to conduct a more detailed review of water use trends that will impact future storage and well supply needs. Planning for additional storage can be delayed until 2020 or later depending on the results of the water study update.

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Year	Improvement	Cost Opinion
2018	Water System Study Update	$15,000
2020	Storage Facility Sizing and Siting Evaluation	$15,000
2021-2022	Storage Facility Design and Construction	$1,500,000 to $2,000,000
2020	Preliminary Well Siting and Site Acquisition for Well No. 6	$50,000
2030	Drill Well No. 6	$450,000
2031	Construct Well No. 6 Facility	$1,200,000

The approximate cost of $1.5 million represents 400,000 gallons of elevated storage. The approximate cost of $2.0 million reflects ground-level storage with a booster station. The cost of ground-level storage will vary based on construction materials and building size. Ground-level reservoirs can be constructed with cast-in-place concrete, welded steel, bolted steel, or pre-cast wire-wound construction. Assuming 400,000 gallons of storage is needed, cast-in-place concrete offers the most economical option with the most flexibility in design. The booster pumping station and reservoir could be built as one facility using cast-in-place concrete. The other options require separate structures.

Ground-level vs. Elevated Storage

The storage facility sizing and siting evaluation will explore the detailed differences between ground-level storage and elevated storage. Life-cycle costs for each need to consider construction cost and long-term operating costs. Ground-level storage requires booster pumping equipment, more electrical and control gear, and a building. The Utility currently operates 300,000 gallons of ground-level storage at Well No. 3. The Well No. 5 facility was designed to accommodate a future ground-level storage facility on the same site.

Elevated storage does not require dedicated pumping equipment but long-term maintenance includes repainting of steel surfaces which presents significant cost. The Utility currently operates three elevated water storage tanks with a total volume of 1 million gallons.

From a construction cost standpoint, the capital cost of elevated storage is generally less expensive than a ground-level reservoir and booster pumping station. However, the life-cycle costs tend to be closer after accounting for periodic elevated tank painting. There are also non-monetary differences between the two forms of storage that need to be considered including operational flexibility, site availability, and types of existing storage in operation.

If you have any questions or would like to discuss further, please call.

Sincerely,

STRAND ASSOCIATES, INC.®

Michael J. Forslund, P.E.

Enclosure

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3.03 POPULATION PROJECTIONS

Figure 3.03-1 presents United States Census Bureau population data from 1980, 1990, 2000, and 2010. The figure is supplemented by projections from the Wisconsin Department of Administration (WDOA), the Capital Area Regional Planning Commission (CARPC), and the Village of Waunakee's 2009 *Comprehensive Park and Open Space Plan*.

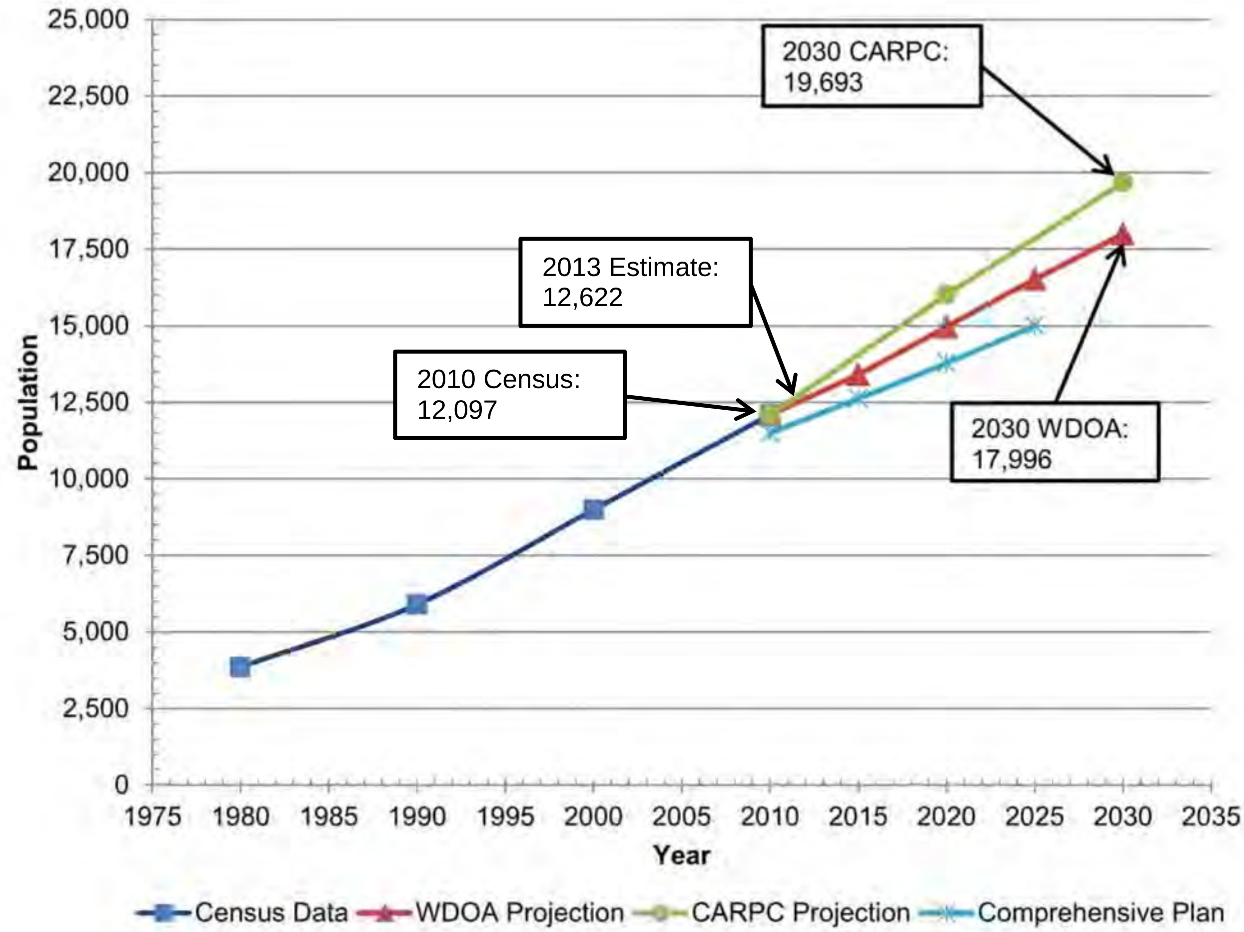

Figure 3.03-1 Population Projections

The Census data shows the Village's population increased by 52.5 percent from 1980 to 1990, 52.5 percent from 1990 to 2000, and 34.5 percent from 2000 to 2010. For the purpose of developing per capita sales, the population for years between the available Census data for 1990, 2000, and 2010 was calculated based on an estimated linear growth rate. The 2010 Census estimated the Village population at 12,097. The WDOA population projection for 2030 is 17,996 while the CARPC population project for 2030 is 19,693. The *Comprehensive Park and Open Space Plan* only projected populations out to 2025, but if the trend were to be linearly extrapolated to 2030, the population estimate would be lower than the WDOA or CARPC projections.

B. Sales to Pumpage Ratio

Figure 3.04-1 presents sales to pumpage ratios since 1997. Sales will be less than pumpage because of unaccounted for water, unmetered sales, leakage, water main breaks, and hydrant flushing. The efficiency has ranged from 87 to 95 percent which is very good. The sales to pumpage ratio used to calculate future demands will be 90 percent. This is a reasonable value to sustain for a well-maintained water system like Waunakee's. If the efficiency cannot be maintained at 90 percent, future demand projections will increase and future water supply improvements may be required sooner.

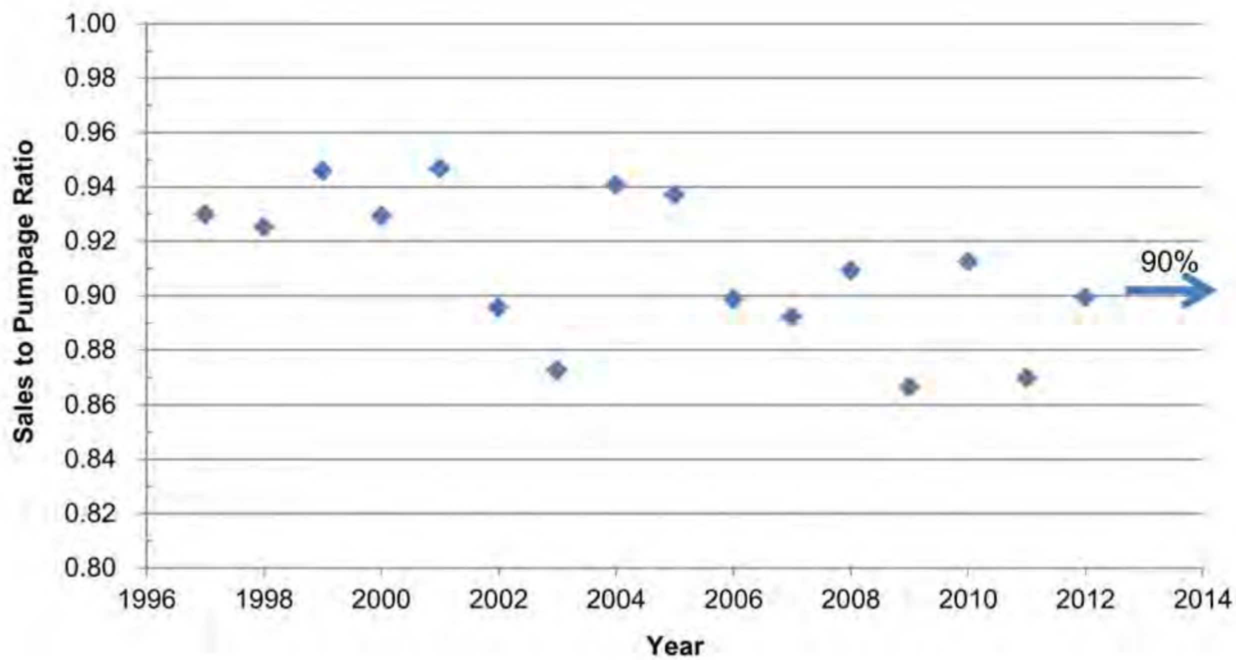

Figure 3.04-1 Sales to Pumpage Ratio

C. Maximum to Average Day Ratio

Figure 3.04-2 presents maximum day to average day demand ratios since 1997. The values range from 1.59 to 2.66. Six of the data points have a value higher than 2.0, with two ratios exceeding 2.5. Based on this historic data a maximum to average day ratio of 2.5 will be used to forecast future maximum day demands.

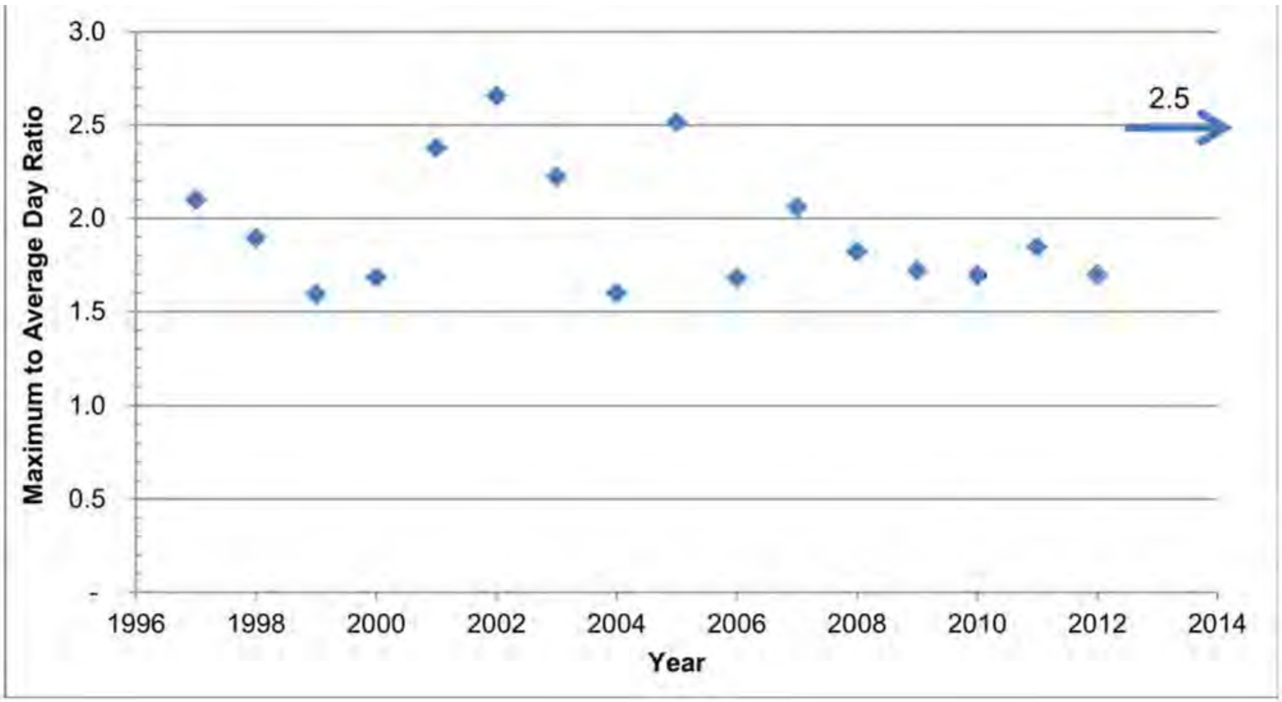

Figure 3.04-2 Maximum Day to Average Day Demand Ratios

D. Water Sales per Capita

Figure 3.04-3 presents the total daily water sales per capita since 1997. Historic data shows a decreasing trend in per capita water usage since 2005. A continued and long-term decline in per capita sales is not likely. Therefore, a value of 100 gallons per capita per day (gcd) will be used to represent the average usage since 1997. The value will account for possible future increases in water usage, similar to the trend that occurred from 2002 to 2007.

The total sales per capita includes residential, commercial, industrial and public sales categories. While a detailed breakdown of each category was outside the scope of this report, a brief review of WPSC data shows the general trend in Figure 3.04-3 follows a similar trend in residential sales. Commercial and public sales have been steady. Industrial sales doubled in 2004 and have seen slight declines over the past eight years.

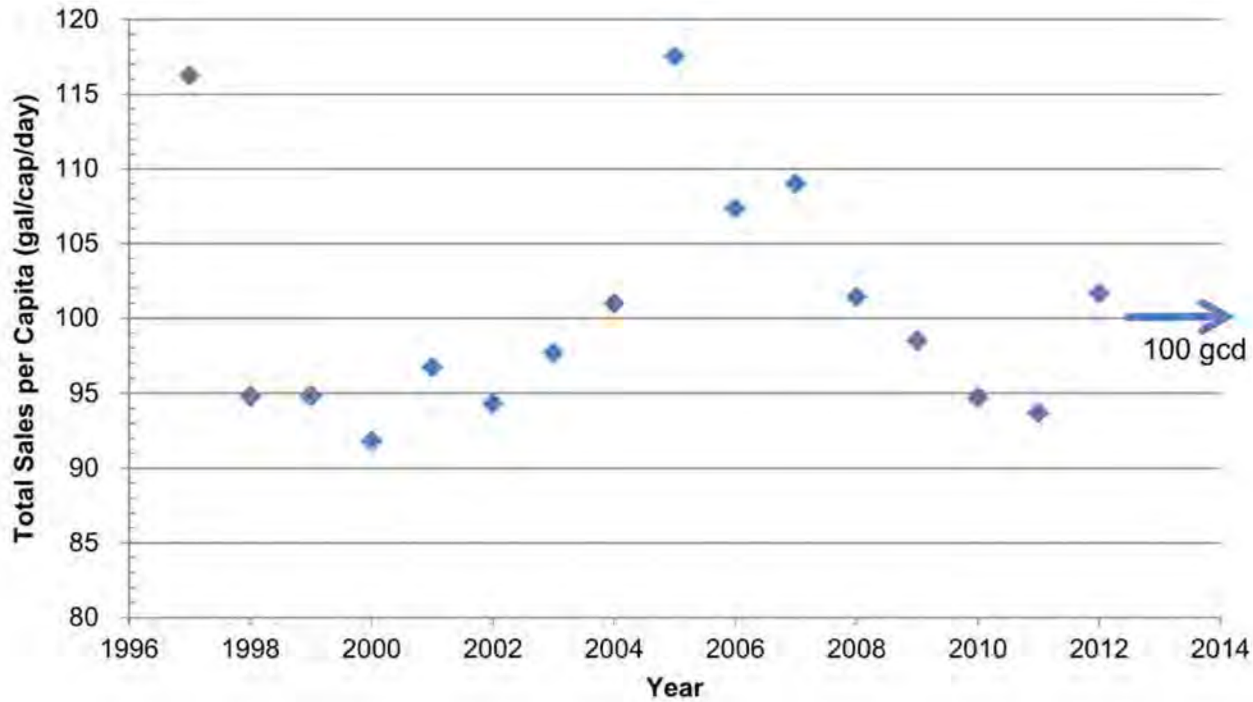

Figure 3.04-3 Per Capita Sales

sales to pumpage ratio (90 percent). The estimated average day pumpage is approximately 2,188,000 gpd, or 1,520 gpm.

B. 2030 Maximum Day

1. Domestic

The 2030 maximum day pumpage is estimated to be approximately 5,470,000 gpd by applying the maximum to average day ratio of 2.5 to the 2030 average day pumpage. This is equal to a demand rate of 3,800 gpm. Figure 3.06-1 presents the projected average and maximum day demands through 2030.

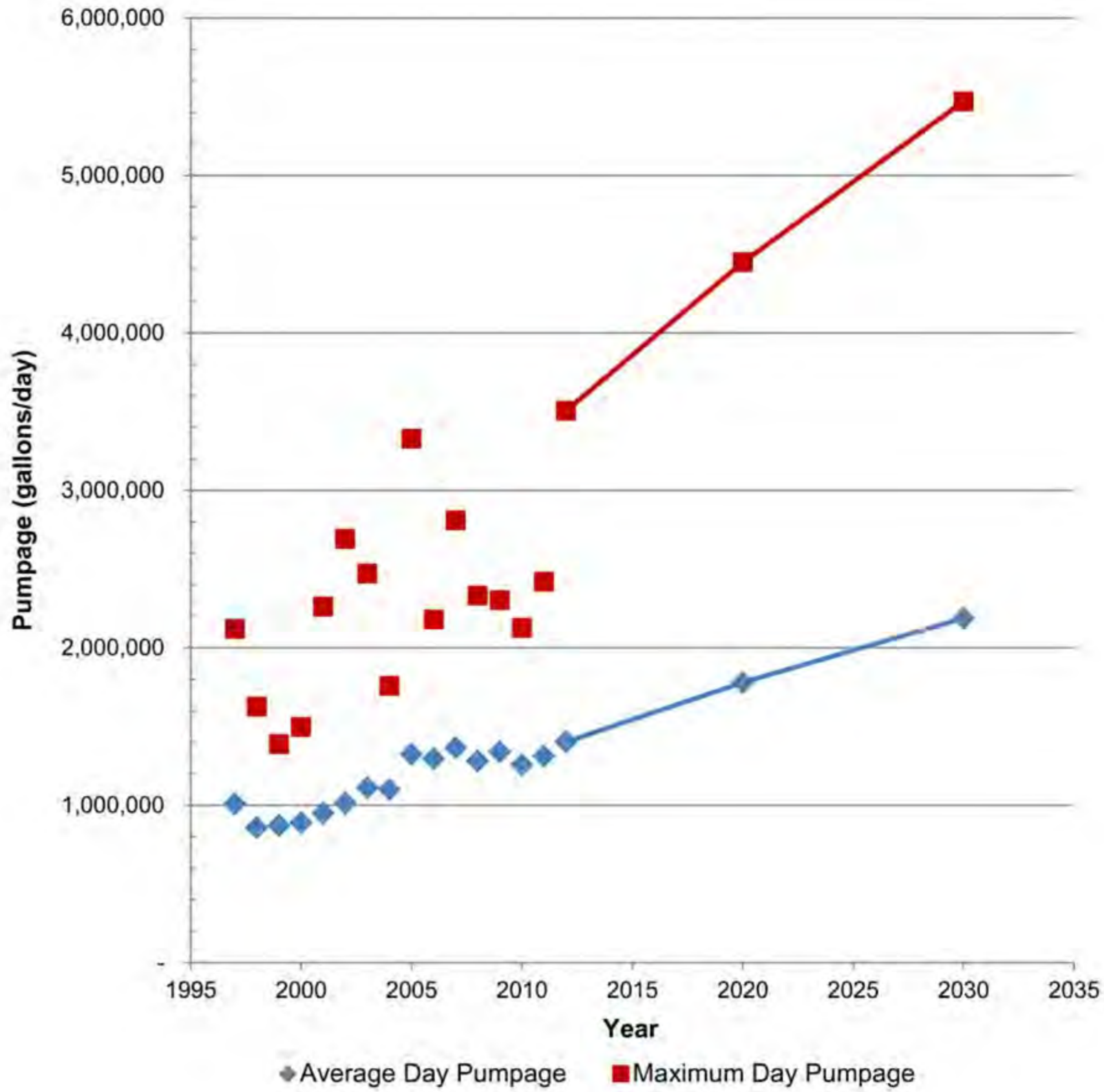

Figure 3.06-1 Projected Average and Maximum Day Demands