EAST BASIN UTILITY SERVICE STUDY

Village of McFarland, Wisconsin

April 2018

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1. INTRODUCTION

The purpose of this study is to evaluate the existing wastewater and public water utility service capabilities within the Village of McFarland's East Basin and quantify and plan for the impacts of projected future community growth on these systems.

For the purposes of this report, McFarland's East Basin generally consists of the current service area of Lift Station #2, the current service area of Lift Station #5, and the undeveloped, unserved lands on the eastern edge of the Village east to Door Creek Road. The eastern edge of the Village has historically and is currently seeing the greatest pressure for new development and this report seeks to aid in proper planning of infrastructure needs as the Village grows.

A similar planning effort was completed in November of 2000 and was titled "McFarland East Basin Capacity Study". That study only explored wastewater service options. This study is intended to be a reevaluation of sanitary sewer capacity recommendations and conclusions from the prior study updated planning data.

Additionally, this study will evaluate the capacity of the Village's municipal water system to serve future developments. The most recent study that contemplated the capacity of the Village's water system was completed in 1997, just prior to the installation of a new elevated storage tank that the study recommended.

2. STUDY METHODOLOGY

2.1. STUDY BOUNDARY DETERMINATION

The initial step in beginning this evaluation was to determine the overall boundaries of the undeveloped and currently unserved areas to be studied. This determination was made by using a combination of existing future land use planning data, topographical and geographic obstacles, and a projection of lands that are most likely to require municipal service in the future. A meeting with Village and MMSD staff was held in February 2018 to review MMSD plans for the study zone and outlying areas to ensure that service options were consistent with existing MMSD plans. A map of the resulting overall study boundary is shown in Figure 2-1, East Basin Study Boundary.

2.2. DEVELOPMENT AND FUTURE LAND USE PROJECTIONS

Future projections for wastewater flows and municipal water demand, require estimates of the expected number of future utility system users. This evaluation will use as the primary population data source the Village's recently adopted, August 2017 Comprehensive Plan. Future land use data was obtained from that plan and the data set was clipped to the overall study boundaries.

The future land use data was then classified as either "developable" or "undevelopable". Developable land is land that can be and is expected to be developed at some future point. Undevelopable land is land that cannot be developed due to environmental or legal reasons or is otherwise particularly unlikely to be developed.

Much of the land within the study boundary cannot be developed due to the presence of existing wetlands or conservation easements. The Wisconsin Department of Natural Resources (WDNR) Wetland Inventory was included in the analysis and it was assumed that lands within this dataset are undevelopable.

The Town of Dunn's Purchase of Development Rights (PDR) program has obtained conservation easements on several of the parcels within the study boundary. These conservation easements restrict, in perpetuity, the use of land to farming or open space. The latest available dataset (April 2017) of PDR properties was used in this analysis and these properties were considered undevelopable.

There are some parcels within the study boundaries that are owned by either Dane County or WDNR. Although these parcels may not have a formal conservation easement, it was assumed that the purpose of the aforementioned parties' ownership was for conservation and that any development on these properties in the foreseeable future is unlikely. These properties were treated as undevelopable lands for the purposes of this study.

Figure 2-2 depicts the resulting developable and undevelopable land within the study boundary. Developable land is shown in solid colors and undevelopable land is shown as hatched.

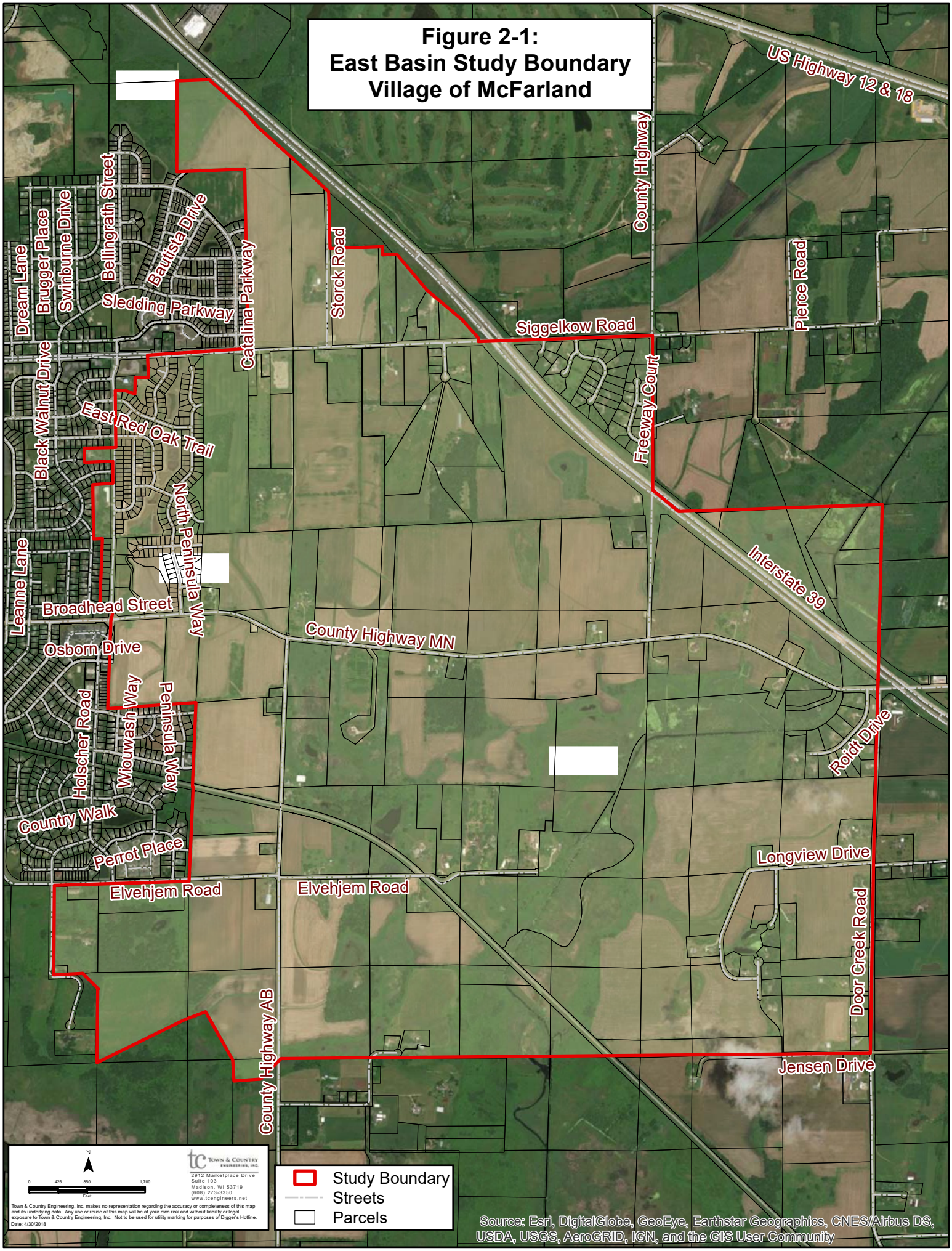
Figure 2-1:
East Basin Study Boundary
Village of McFarland
US Highway 12 & 18
County Highway
Pierce Road
Storck Road
Siggelkow Road
Freeway Court
Interstate 39
Catalina Parkway
Sledding Parkway
Bautista Drive
Bellingrath Street
Swinburne Drive
Brugger Place
Dream Lane
Black Walnut Drive
East Red Oak Trail
North Peninsula Way
Leanne Lane
Broadhead Street
Osborn Drive
County Highway MN
Wiouwash Way
Peninsula Way
Holscher Road
Country Walk
Perrot Place
Roidt Drive
Longview Drive
Elvehjem Road
Elvehjem Road
Door Creek Road
Jensen Drive
County Highway AB
N
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Date: 4/30/2018
Study Boundary
Streets
Parcels
Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

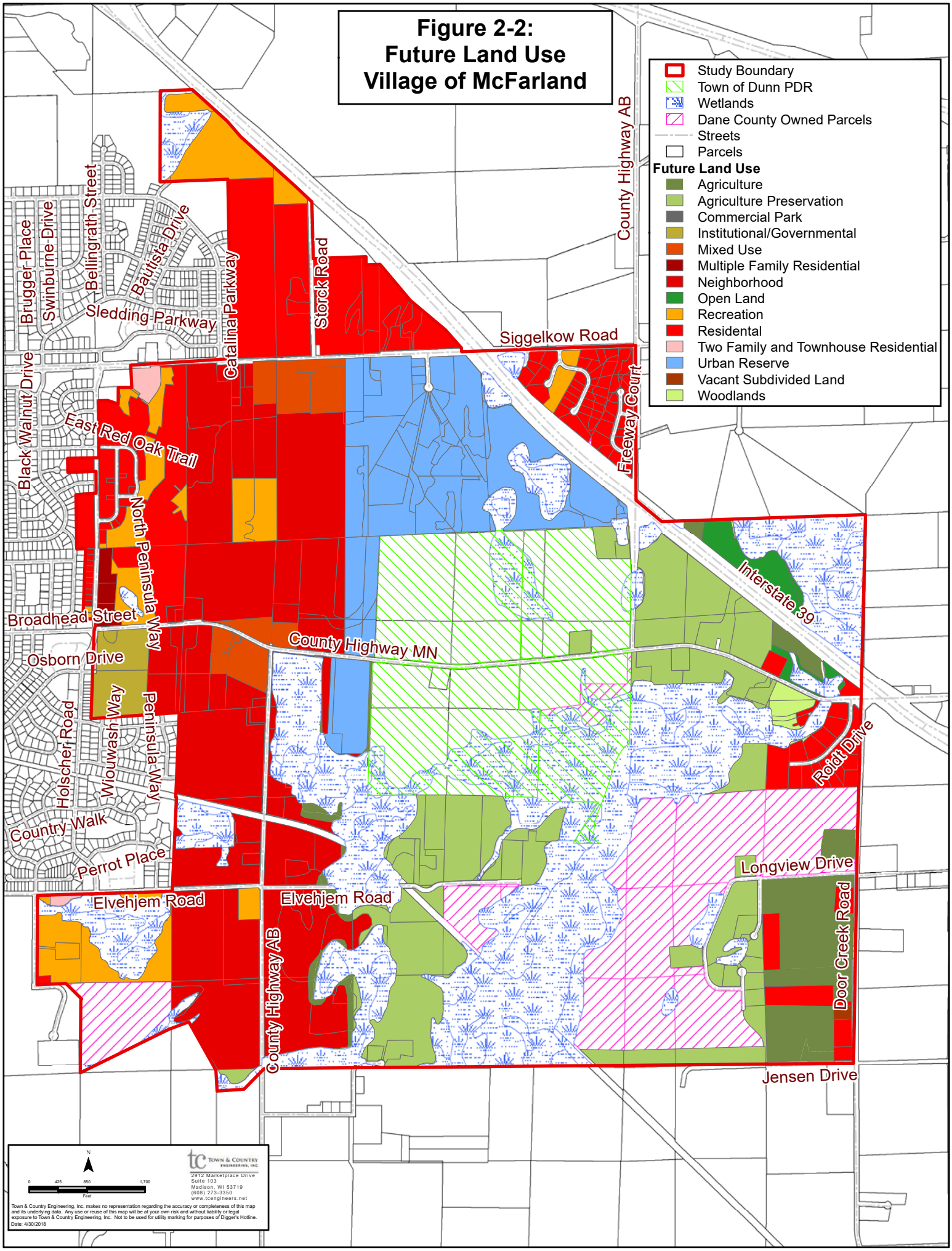
Figure 2-2:
Future Land Use
Village of McFarland
Study Boundary
Town of Dunn PDR
Wetlands
Dane County Owned Parcels
Streets
Parcels
Future Land Use
Agriculture
Agriculture Preservation
Commercial Park
Institutional/Governmental
Mixed Use
Multiple Family Residential
Neighborhood
Open Land
Recreation
Residental
Two Family and Townhouse Residential
Urban Reserve
Vacant Subdivided Land
Woodlands
Brugger Place
Swinburne Drive
Bellingrath Street
Bautista Drive
Sledding Parkway
Catalina Parkway
Storck Road
County Highway AB
Siggelkow Road
Freeway Court
Black Walnut Drive
East Red Oak Trail
North Peninsula Way
Broadhead Street
Osborn Drive
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Date: 4/30/2018

2.3. IMPACTS OF DEVELOPMENT ON FUTURE WATER USAGE

From the data contained in Figure 2-2, assumptions of expected population density and water usage can be applied to estimate future utility service demand within the study area. Table 2-1 summarizes the design parameters and Table 2-2 summarizes the population and water usage estimates within the study area.

Table 2-1: Design Parameters for Community Growth Forecasts

Population Density		
Housing Density	3.2	housing units/acre
Occupancy Density	2.83	persons/housing unit
Water Usage		
Individual Usage	75	gpd/person
Wastewater Peaking Factor	3.5	
Industrial/Commercial Usage	1000	gpd/acre
Residential Usage	679.2	gpd/acre

Table 2-2: Developable Property within Study Boundary Wastewater Usage Estimates

Future Land Use Classification	Total Area Within Study Boundary (ac.)	g.p.d./ac.	Average Water Useage (gpm)	Peak Wastewater Flow (gpm)
AGRICULTURE	83.4	679	39	137.64
AGRICULTURE PRESERVATION	244.2	679	115	403.16
COMMERCIAL PARK	0.0	1000	0	0.00
INSTITUTIONAL/GOVERNMENTAL	23.0	1000	16	55.84
MIXED USE	42.2	679	20	69.64
MULTIPLE FAMILY RESIDENTIAL	6.6	679	3	10.97
NEIGHBORHOOD	419.9	679	198	693.15
OPEN LAND	19.6	679	9	32.32
RECREATION	104.2	679	49	171.99
RESIDENTIAL	267.5	679	126	441.59
TRANSPORTATION, COMMUNICATIONS AND UTILITIES	4.2	0	0	0.00
TWO FAMILY AND TOWNHOUSE RESIDENTIAL	5.6	679	3	9.30
URBAN RESERVE	194.5	679	92	321.16
VACANT SUBDIVIDED LAND	1.2	679	1	1.95
WATER	0.4	0	0	0.00
WOODLANDS	6.9	679	3	11.43
		TOTAL	674	2360.13

2.4. POPULATION GROWTH ESTIMATES

Another useful set of design data is the Village's forecasted population growth. This data can be used to estimate the point in time at which certain levels of development will be reached. For the purposes of this study, it has been assumed that all future population growth in the Village will be direct result of development within the study boundary.

The population growth estimates used in this study were obtained from the Wisconsin Department of Administration (DOA). The DOA estimates were available to the year 2040. For use in this study, it was assumed that the rate of growth included in the estimates from 2020-2040 would continue steadily out to 2060. The population projections used in this study are given in Table 2-3.

Table 2-3: Population Estimates

Village of McFarland Population Growth Estimates	
Year	Population
2010	7,815
2016	8,320
2020	8,490
2025	8,930
2030	9,335
2035	9,635
2040	9,895
2050	10,457
2060	10,682

3. WASTEWATER COLLECTION SYSTEM EVALUATION

3.1. EXISTING INFRASTRUCTURE

The Village of McFarland has an agreement with Madison Metropolitan Sewerage District (MMSD) to accept sanitary sewer flow for processing at the Nine Springs Wastewater Treatment Facility. As a result of this agreement, the Village of McFarland does not operate its own wastewater treatment facility, but instead oversees a collection sewer system, including lift stations, to convey their flow contributions to the MMSD system.

A detailed map of the existing Village and MMSD infrastructure is included in Appendix C. This map also includes a delineation of future drainage sub-basins and highlights infrastructure components discussed in this section.

The following sub-sections discuss elements of the Village's existing wastewater infrastructure that will be critical to the conveyance of flows from future developments.

3.1.1. LIFT STATION #5

The East Basin area of McFarland is currently served by two such lift stations that convey flow to the MMSD system. The Village's Lift Station #5 (also referred to as "Holscher Road Lift Station") was built in 2016. The service area for Lift Station #5 encompasses approximately 148 acres of which the Juniper Ridge and Prairie Place housing developments are the primary users. The creation of this lift station directs flow to a discharge point in the MMSD system that was not originally designed to or planned for accepting flows from developments in this area. For this reason, MMSD's approval of the lift station included several contingencies.

The MMSD approval contingency most relevant to this report is that Lift Station #5 must be abandoned once gravity flow sanitary sewer is constructed to the south that is able to relieve the lift station and redirect its contributory flow to Lift Station #2. Additionally, the maximum rate of discharge from the station may not exceed 500,000 gallons per day. For further reference, the MMSD Commission Resolution can be found in Appendix A. For the purposes of this report, it is assumed that flows tributary to Lift Station #5 will become flows tributary to Lift Station #2 at some point in the future.

3.1.2. COUNTRYWOOD INTERCEPTOR

The Countrywood Interceptor is a 15" PVC gravity sewer which carries flows to Lift Station # 2 from the east. This interceptor will also serve the majority of the new drainage sub-basins depicted in Figure 3-1. Generally, it will be the first connection point of any new developments to the east into the existing system and as such, the capabilities of this interceptor are critical to this analysis. The estimated maximum capacity of this interceptor is 1,010 gpm or 1.45 MGD.

In the fall of 2014 and spring of 2015, a study of the existing influent flows and existing pumping capabilities was completed to assess the capacity of Lift Station #2. Flow monitoring of the Countrywood Interceptor was conducted as a part of that assessment. At that time, it was found that the peak flow in this interceptor was 190 gpm. No significant amount of development has occurred in this area since the spring of 2015 and this peak flow number will be used for the purposes of this study.

3.1.3. LIFT STATION #2

Lift Station #2 is located near the southeast corner of the Village. A map of the Village's collection system is included in Appendix C. It was constructed in 1979 and is a dual pump dry pit type "can" station. The pumps in Lift Station #2 were upgraded in 2000. There are two 20 HP pumps that alternate pumping wastewater every other cycle into an 8" force main. The pumps were designed to pump at minimum 600 gpm with 56' of total dynamic head (TDH).

Lift Station #2 receives gravity flow from a 10-inch PVC collector and from the 15-inch PVC Countrywood Interceptor. The 10-inch collector serves an area to the north of Lift Station #2 which has already been developed and from which future wastewater flows are unlikely to increase. The highest observed peak flow from this interceptor was recorded during the 2000 East Basin Capacity Study. That peak flow rate will be used for future planning and is 182 gpm. Flow contributions from the Countrywood Interceptor are discussed in Section 3.1.2.

A study of the existing influent flows and existing pumping capabilities was completed in the fall of 2014 and spring of 2015 to assess the capacity of Lift Station #2. Drawdown tests were conducted in both years and it was found that the minimum pumpage rate of the station was consistently above the rated pump capacities. Tracked pumpage rates were as high as 740-860 gpm. The minimum pumpage rate at the station was found to be 680 gpm. Accordingly, 680gpm will be used as the maximum design capacity of the station. The analysis completed in 2014 and 2015 was detailed further in a memo that has been included in Appendix B.

3.1.4. LIFT STATION #2 FORCE MAIN

The Lift Station #2 Force Main is an 8" cast iron force main that was likely constructed with the lift station in 1979. It has one air relief valve and has a total length of approximately 3,200 feet. The force main discharges into the Highland Drive Interceptor at Manhole 301, as shown in Appendix C

The estimated peak capacity of this force main is 950 gpm. This ultimate capacity is dependent upon the specific pump and motor combination used. Additionally, the estimated capacity decreases as the condition of the pipeline deteriorates. The condition of the force main has not been directly assessed. Given that it is an approximately 40 year-old cast iron pipe, it is possible that it has experienced some tuberculation over time. This type of corrosion restricts the interior diameter of the pipe, increases frictional losses, and would reduce the ultimate capacity. Further

inspection of the force main would be required to make an accurate assessment of its ultimate capacity. Cast iron pipelines are known to be relatively brittle and a condition assessment would also yield useful information to inform decisions about potentially increasing flow rates and corresponding pressure surges.

3.1.5. HIGHLAND DRIVE INTERCEPTOR

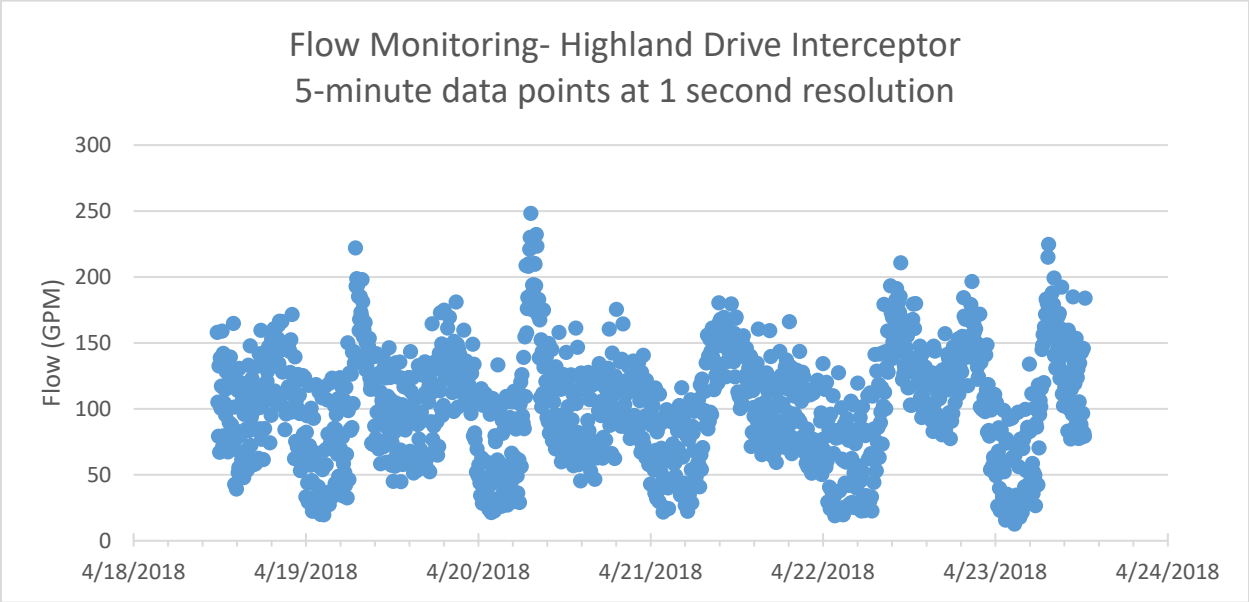
The Lift Station #2 Force Main discharges into the Highland Drive Interceptor. This is a concrete sanitary sewer, with an average slope of approximately 0.27%. The initial 1,300 feet of this sewer is 12 inches and the diameter increases to 18 inches at Jaeger Road. The total length of the interceptor is approximately 3,100 feet. The design capacity of this sewer for the purpose of this study will be set at the capacity of the 12" section of the sewer, which is 702 gpm.

The Highland Drive Interceptor has the lowest amount of remaining design capacity to be allocated to future developments of the existing infrastructure elements examined in this study. For this reason, it is a critical design point. There has not been any studies on this section of sewer to estimate the contribution of local gravity flow (non-force main) into the interceptor. The contribution of local flow can be estimated based on the number of homes that are connected and using the typical design criteria as given in Table 2-1. This estimation approach yields a 37.7 gpm average (no peaking factor) daily local flow contribution.

Some flow monitoring was completed in mid-April of 2018 to verify this estimate of average daily local flow. The monitoring equipment was installed in Manhole SA323, the first manhole east of the intersection of Jaeger Road and Exchange Street. The results of that monitoring are shown in Figure 3-1. Note that Flows throughout the monitoring period remained relatively consistent. Some elevation of flow on the evening of 4/22/18 was measured, which is likely due to a snow melt event. Overall, average daily flow in the interceptor was measured at 102 gpm. Previous flow monitoring studies have indicated that the average daily flow contribution from the Lift Station #2 Force Main is approximately 70 gpm. This would indicate that the average contribution of local flow into the interceptor is 32 gpm, which is consistent with estimates.

The flow monitoring results verify the average daily flow assumptions. However, the peak flows as measured during this monitoring period in this manhole are well below the force main discharge rate of 680 gpm. One possible reason for this difference could be that the data collected simply did not record a time when the peak flow was passing through the monitoring equipment. During the studies completed in 2014/2015 (Appendix B), wet well drawdown times were less than 2 minutes. The data points collected for this monitoring are in 5 minute intervals at 1 second resolution and for this reason may have not recorded during the time of peak flow.

Figure 3-1:



Another possibility is that the wastewater flow is experiencing significant amounts of natural flow equalization prior to reaching the monitoring point. The pump runtimes at Lift Station #2 are relatively short and the approximate distance from manhole SA323 to Lift Station #2 is 4,200-feet. Further analysis and flow monitoring could be completed to verify both average and peak flow rates and flow equalization within the interceptor and force main.

The Highland Drive Interceptor carries flows to a discharge point in a MMSD Manhole located near the intersection of Yahara Drive and USH 51. The MMSD interceptor then carries this flow to MMSD Pumping Station 9 and further on to the Nine Springs Wastewater Treatment Plant.

3.2. WASTEWATER SUB-BASINS

3.2.1. DRAINAGE CHARACTERISTICS AND PROJECTED FLOWS

The sub-basins delineated for this study are shown in the map included in Appendix C. Basins A, B, C, D, E, and F all appear to be capable of conveying flow to Lift Station #2 by gravity alone. Basin A also currently represents the service area of Lift Station #5. Basins J1, J2, J3, J4, J5, J6, and J7 will all require an additional lift station to convey future wastewater flows to the existing system.

Sub-Basin K1 could be served by gravity sewer to Lift Station #2. Sub-Basin K2 would require a lift station to convey flow to Lift Station #2. However, both basins have been identified by the Capitol Area Regional Planning Commission (CARPC) and MMSD to be served by an existing MMSD interceptor which lies approximately 200 feet to the north of the northern edge of Basin K2. For these reasons, the projected flows resulting from development of these basins should not be included in the future needs assessment for Lift Station #2.

By applying the development wastewater use data detailed in Section 2.3 to the land use data within the sub-basin boundaries, it is possible to project future wastewater use within each sub-basin. Table 3-2 (following page) summarizes various sub-basin characteristics

Table 3-2: Projected Flows in Developed Sub-Basins

DEVELOPED DRAINAGE SUB-BASIN CHARACTERISTICS					
Basin	**Total Area in Basin (ac.)**	**Developable Percentage**	**Fully Developed Peak Wastewater Flow (gpm)**	**Estimated Population**	**Estimated Number of Homes**
A	147.21	92.1%	224.04	1226	433
B	28.85	94.2%	46.61	226	80
C	79.46	96.3%	142.30	507	179
D	131.04	57.3%	123.99	680	240
E	113.91	98.1%	184.40	1012	358
F	163.27	96.9%	261.21	1433	506
J1	68.42	34.9%	39.41	216	76
J2	43.19	45.2%	32.22	177	62
J3	294.88	38.7%	188.18	1033	365
J4	297.71	25.8%	126.76	696	246
J5	270.90	58.3%	260.76	1431	506
J6	379.30	33.0%	206.62	1134	401
J7	372.19	36.8%	226.32	1242	439
J8	64.34	62.3%	66.18	363	128
K1	107.15	94.7%	167.51	919	325
K2	49.89	77.2%	63.57	349	123
Sub Total A-F			982.55	5084	1796
Sub Total J1-J8			1146.45	6292	2223
Sub Total K1-K2			231.08	1268	448
Sub Total A-J8			2129.00	11376	4019
Overall Total			2360.09	12644	4467

3.2.2. GRAVITY COLLECTOR SEWER ROUTING, SUB-BASIN A

A relief gravity sewer route from Lift Station #5 to Lift Station #2 is depicted in Figure 3-2. It is shown in green on the map and labeled as "Lift Station #5 Relief Sewer". As shown, this relief sewer is 4,000 lineal feet and 12-inches in diameter. The southernmost point of this sewer is a connection to an existing manhole in the intersection of Peninsula Way and Tuscobia Trail, referred to as Manhole #30A. The invert elevation in this manhole is 856.14. The invert elevation of Lift Station #5 is 866.00. The proposed sewer route has 10 manholes and allowing for a 0.1' invert elevation drop in each manhole, the required slope is approximately 0.22%.

The bury depth of this route varies from approximately 12-28 feet. The portion of the route that traverses Sub-Basins B and C is the deepest and any major deviations from the route shown would likely require bury depths beyond 28 feet.

It should also be noted that this interceptor would be capable of serving portions of Sub-Basins B, C, E and F. It would, however, require that significant additional sewer be constructed that is beyond 20 feet in depth of bury. Additionally, it would not be possible to serve significant portions within the southernmost part of Sub-Basin F with this sewer via gravity. As a result, a separate branch of the collector sewer would need to be constructed, shown on the map as "Northeast Interceptor" in cyan.

3.2.3. GRAVITY COLLECTOR SEWER ROUTING CORRIDOR, SUB-BASINS B-F

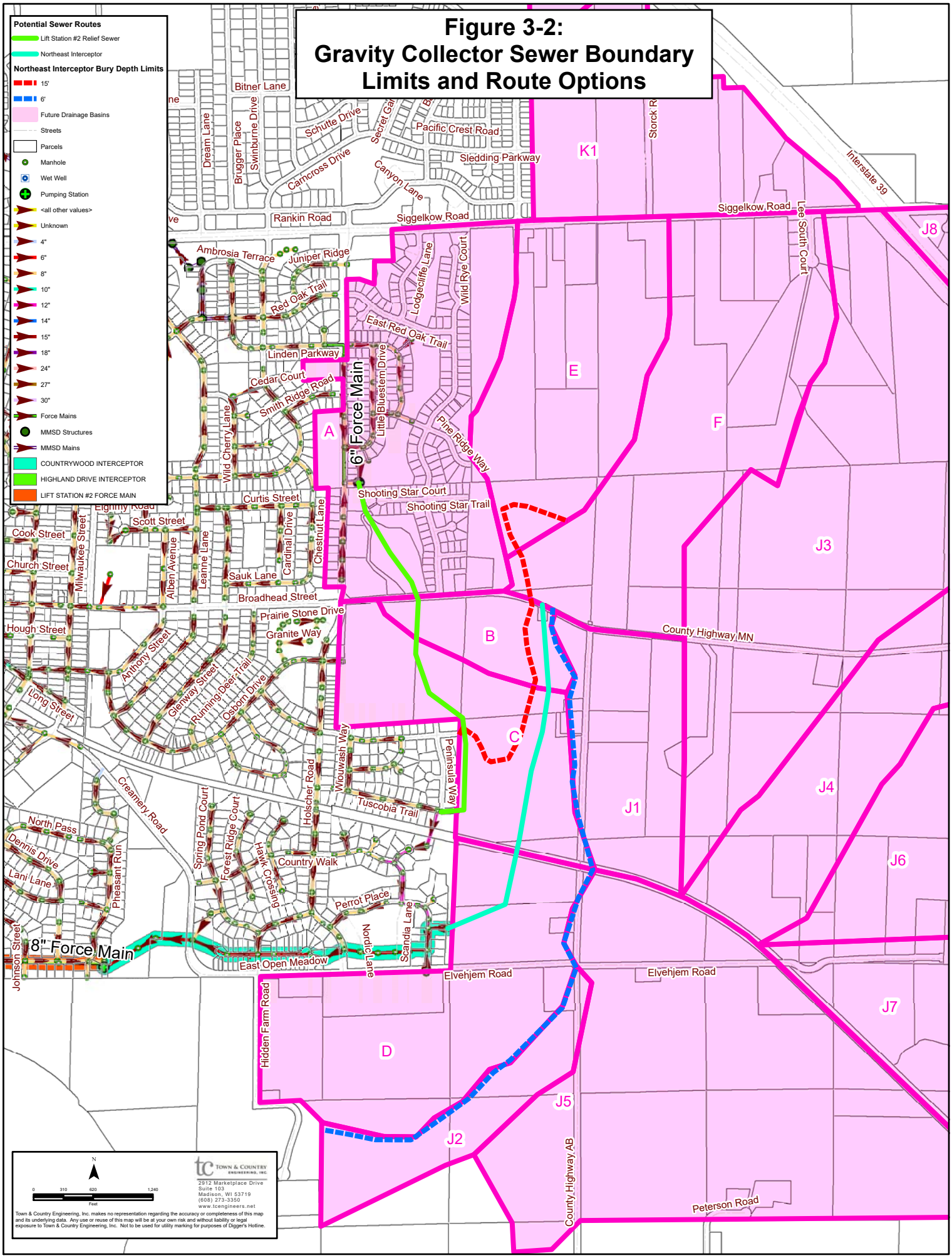
In order for flows from future developments to be conveyed via gravity from Sub-Basins B, C, E and F, a gravity sewer will have to be carefully planned and constructed. The sewer will need to be constructed at minimum grades to be able to be connected to the existing system and there is a relatively narrow corridor within which the sewer can be constructed and still be capable of conveying flows from basins B-F.

Using the existing invert elevation at the Village's manhole on Devil's Lake Way (849.76) and a minimum pipe slope of 0.3%, it is possible to determine the required ground elevation at a given point that would be required to maintain a required depth of cover over the pipe. Figure 3-2 is a representation of this analysis.

Shown in Figure 3-2 are the boundary lines (in red and blue) at which a new interceptor sewer would reach the shallow bury limits of 6 feet and the deep bury limit of 15 feet. The space between these limits is representative of a corridor within which a new sewer could be constructed that would be capable of serving basins B-F. The bury limit of 6 feet directly corresponds with the eastern boundaries of sub-basins F, B, C, and D. South and east of that limit, lift station service will be required to provide sanitary service.

The southern portion of Sub-Basin D is relatively flat and will also require minimum grade sewer connected to the Village's manhole on Devil's Lake Way to provide service. The routing of a collector sewer through this area of the sub-basin will be highly dependent on how the area develops.

Figure 3-2: Gravity Collector Sewer Boundary Limits and Route Options



3.3. WASTEWATER COLLECTION SYSTEM CAPACITY CONSTRAINTS

Having compiled the population, land use, and drainage information from the previous sections, it is possible to contemplate the impacts of development on the wastewater infrastructure. The following sub-sections contemplate these impacts and what improvements may meet future needs for additional system capacity.

3.3.1. EXISTING SYSTEM CAPACITY

The first scenario to consider is the point at which the existing system reaches ultimate capacity without any improvements being made. Table 3-3 summarizes the existing system capacities.

Table 3-3:

Existing Collection System Infrastructure Constraints				
Existing Component	**Design Capacity (gpm)**	**Existing Base Peak Flow (gpm)**	**Remaining Flow Capacity (gpm)**	**Number of Additional Homes to Reach Capacity**
Lift Station #2	680	372	308	597
Countrywood Interceptor	1010	190	820	1590
Lift Station #2 Force Main	950	680	578	1120
Highland Drive Interceptor	702	810	-108	N/A

Table 3-3 shows the Highland Drive Interceptor has no remaining capacity and may be surcharged at points during peak flow events. However, the capacity of this portion of the interceptor has likely been in this state for nearly 8 years now without known incident. Since the peak local flow contribution to the inceptor has an assumed amount of I&I added, it is also possible that the amount of local I&I in the area is less than estimated and that the interceptor does not surcharge. Additional discussion of existing flow characteristics in the Highland Drive Interceptor can be found in Section 3.1.5

Regardless of the actual flow conditions within the Highland Drive Interceptor during a peak event, it is recommended to make capacity improvements to the interceptor prior to upgrades in the capacity of Lift Station #2.

The capacity of the Highland Drive Interceptor could be increased by the installation of a parallel sewer or by replacement of the existing sewer with a larger diameter sewer. If a parallel sewer option is pursued, then lining of the existing sewer should be considered in order to maintain structural integrity and original design capacity. Lining of the existing sewer may increase the flow capacity in the sewer by about 2-3%.

3.3.2. MAXIMIZING CAPACITY OF LIFT STATION #2 AND FORCE MAIN

If improvements are implemented to increase the capacity of the Highland Drive Interceptor, Lift Station #2 would be the next constraint in system capacity. Additionally, with the pump replacements that were made in 2000, the existing capacity of the station is not far below the capacity of the receiving force main and thus these two infrastructure elements would likely reach capacity around the same time.

Currently the lift station has separate wet well and dry well with dry pit centrifugal pumps. Multiple options exist that could add capacity to the existing lift station. The lift station could be converted to a submersible lift station. Larger pumps paired with the correct variable speed drives and a larger force main may allow for significant capacity to be added while keeping the station in a wet well/dry pit configuration. Alternatively, a larger new station could be constructed that accommodates larger pumps. Additional options may exist and a detailed evaluation of all available options should be completed prior to implementation of any improvements project.

A significant increase in lift station capacity would also require additional capacity in the force main. The options for force main capacity improvement would be constructing a parallel relief force main or a full replacement of the existing force main with a new larger diameter force main.

3.3.3. MAXIMIZING COUNTRYWOOD INTERCEPTOR CAPACITY

The final element of infrastructure that will require increased capacity is the Countrywood Interceptor. This interceptor is expected to have enough capacity to accommodate flow from basins A-E as well as approximately 35% of a fully developed basin F. The amount of additional flow that can be accommodated in the interceptor represents a population growth of approximately 4,500 people, which would be a 54% increase in the Village's current population. A straight-line projection of the current forecasted population growth rate would suggest that this population level would not be reached before 2100. However, this calculation would change if population increases are not as forecasted.

When increased capacity is required at this point in the system, it could be achieved with either replacement of the existing sewer with a new, larger diameter sewer or by constructing a parallel sewer to convey the additional flow to Lift Station #2.

3.4. RECCOMENDATIONS

3.4.1. PLANNING FOR SYSTEM IMPROVEMENTS

Table 3-2 displays the ultimate number of homes that could theoretically be added to reach the maximum capacity of a given element of infrastructure. However, improvements to the collection system should be implemented well before that limit is reached. Since the trigger for all improvements in this system is the capacity at Lift Station #2, the recommended point in time for

the implementation of infrastructure improvements will hinge on the influent peak flow to the station.

It is recommended that new improvements be constructed when influent peak flows at Lift Station #2 reach a level 100 gpm below station capacity. This corresponds with an influent peak flow at the station of 580 gpm and the capacity for approximately 400 additional homes to be added to the station. If development and population growth occur at the rates projected by the DOA and outlined in Section 2.4, then this level of flow may be reached as early as 2032.

Capacity improvements in the Highland Drive Interceptor must be constructed before improvements at the lift station are implemented. It would likely be prudent to complete those improvements a few years prior to the lift station upgrade. It is recommended that planning for the interceptor improvements should begin when the peak influent flows at Lift Station #2 reach approximately 530 gpm or when replacement of the pumps at the lift station becomes necessary, whichever occurs first. If the Village's growth is as forecasted, this would result in construction of the interceptor improvements 3 years prior to construction of lift station upgrades.

Further analysis and design will need to be completed to determine the required improvements to the force main when planning for improvements at Lift Station #2. Additionally, the impacts of improving the force main capacity prior to lift station upgrades should be investigated. By increasing the size of the lift station's force main, the total amount of pumping head that the pumps are required to handle should drop. This may increase the overall capacity of the station and could delay the need to pump upgrades and lift station improvements.

3.4.2. EXISTING INFRASTRUCTURE CONDITION ASSESSMENTS

It is recommended that the Village actively monitor the condition of the critical infrastructure elements discussed in this report. This is especially true for the older and thus potentially more vulnerable infrastructure. The Lift Station #2 Force Main and Highland Drive Interceptor would be prime candidates for investment of resources in inspection and preventative maintenance.

The inspection and maintenance of the Highland Drive Interceptor can be completed with typical pipeline televising and cleaning equipment and performed by many local contractors. The cleaning of the interceptor can help to maintain its peak capacity. Inspection data can be used to make a condition assessment and may advise replacement or lining well before any capacity improvements are expected to be made at Lift Station #2.

Further monitoring of the Highland Drive Interceptor to verify peak flow rates would also be advisable over time and as additional users are connected to Lift Station #2. As shown in Table 3-3, the interceptor is likely operating at capacity during peak flow events. To date this has not caused any issues in the area, but as discharged average flows begin to increase it is recommended that this point in the system is closely monitored. Additionally, as the interceptor ages, it is possible that the

amount of inflow and infiltration into the system could increase. Periodical monitoring of peak flows would aid in tracking the status of this critical portion of infrastructure.

The Lift Station #2 Force Main is approaching 40 years of age and is a cast iron force main. This type of force main is prone to internal corrosion and reduction in effective capacity. The existing condition of this pipe is unknown. Force main inspections are less typical in the industry than gravity sewer inspections and the methods do vary. Various emerging and older technologies are available to perform these inspections. Given the length of this force main, it may be possible to conduct an inspection with typical televising equipment. Further analysis would need to be completed to determine the feasibility of inspection and the best and most cost effective method for force main inspection in this case.

3.4.3. IMPACTS OF LIFT STATION #5 ABANDONMENT

Special consideration should be given to monitoring the trigger points that may cause the need to abandon Lift Station #5 and redirect its flows to Lift Station #2. The projected peak flow resulting from a fully developed Sub-basin A alone would require that improvements be made to Lift Station #2 and the Highland Drive Interceptor. The actual build out of developments on the east side of the Village and the corresponding availability of gravity sewer to convey its tributary flow to Lift Station #2 will likely be the governing factor.

If gravity sewer is not available to carry flow from Sub-Basin A to Lift Station #2 and if Sub-Basin A is fully developed, it is possible, given the rate of current growth projected for the Village, improvements to Lift Station #2 could be delayed for another 15-25 years beyond 2030. This relationship between the two lift stations complicates planning efforts. The current status of the potential for Lift Station #5 abandonment should always be taken into consideration prior to making any infrastructure improvement decisions in the study area. Additionally, communication and coordination of planning efforts with MMSD should be ongoing so that any operational or design changes can be properly accounted for.

4. MUNICIPAL WATER SYSTEM EVALUATION

4.1. EXISTING INFRASTRUCTURE

4.1.1. MUNICIPAL WELLS

McFarland's water system serves the entire Village, with an average demand of around 500,000 gallons per day (gpd). The demand stays fairly consistent throughout the year, with a slight increase during the summer months. The Village currently operates 3 wells, Well #1, Well #3, and Well #4. Well #2 was removed from service in 1987 and permanently abandoned in 2000 due to the presence of contaminated groundwater at the site.

A future well, "Well #5", has been contemplated and discussed in multiple studies conducted in the past. Future Well #5 will serve to meet future development capacity needs of the Village. The Village has selected a future site and purchased the property for this planned addition to the system. A report was produced in 2012 (Well Site Investigation Report- Well No. 5) to document the evaluation and suitably of the selected site. The locations of the existing wells and site for the planned well are shown in Figure 4-1. The characteristics of the Village's existing wells are detailed in Table 4-1.

Table 4-1: Existing Well Statistics and Capacities

Well #	Year Installed	Total Depth (ft.)	Well Diameter (ft.)	Pump Size (HP)	Design Capacity (gpm)
1	1971	560	10	40	500
3	1996	700	18	125	1000
4	1990	800	20	100	1000

4.1.2. MUNICIPAL STORAGE

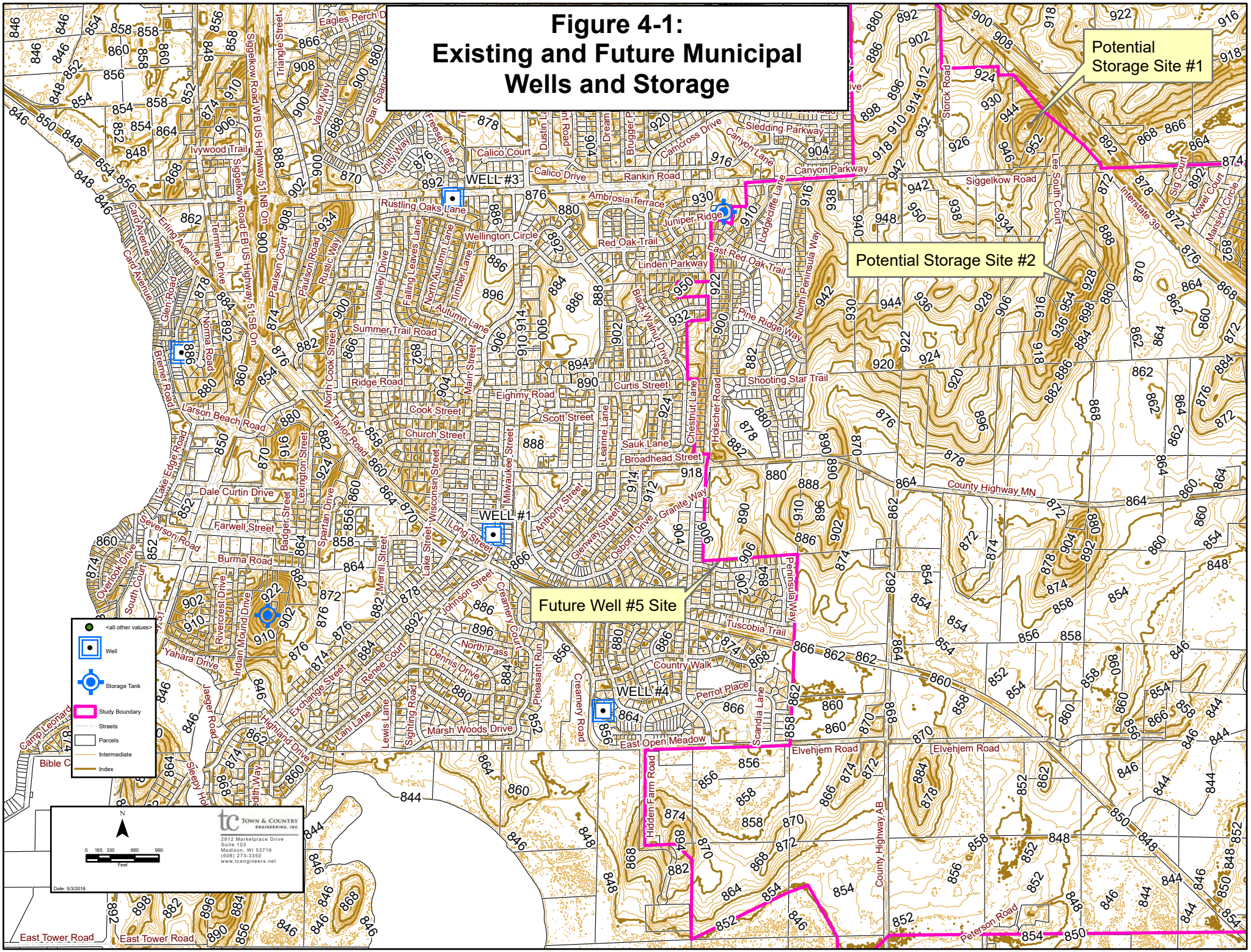
The Village currently has 2 elevated storage tanks. The locations of these storage tanks are as indicated in Figure 4-1.

The Burma Reservoir was constructed in 1975 and is the older of the two storage tanks. It is the southernmost storage tank in the Village. It has a capacity of 500,000 gallons. The tank has an overflow elevation of 1048 feet and an operating range of approximately 37 feet. The ground elevation at the tank is approximately 957 feet.

The Holscher Reservoir was constructed in 2000. It is the northernmost storage tank in the Village. It has a capacity of 750,000 gallons. The tank has an overflow elevation of 1047 feet and an operating range of approximately 40 feet. The ground elevation at the tank is approximately 934 feet.

The maximum service elevation of the existing storage tanks is approximately 965 feet.

Figure 4-1: Existing and Future Municipal Wells and Storage



4.1.3. WATER DISTRIBUTION SYSTEM

The Village of McFarland's water system is comprised of mains ranging in size from 1 inch to 12 inches in diameter. The 2016 water main size inventory is summarized in Table 4-2. There are approximately 231,555 feet, or about 44 miles of water main. The majority of the system is 6-inch and 8-inch in diameter and those sizes make up 29% and 42% of the system, respectively. A listing of Village water main statistics is included in Table 4-2. A map of the existing water distribution system is included in Appendix D.

Table 4-2: Existing Water Main Statistics

Existing Water Main Statistics		
Diameter (inches)	**Total System Length (ft.)**	**Percentage of Overall System**
1	155	0.07%
2	33	0.01%
4	356	0.15%
6	66253	28.61%
8	97664	42.18%
10	43569	18.82%
12	23525	10.16%

4.2. WATER SYSTEM ANALYSIS

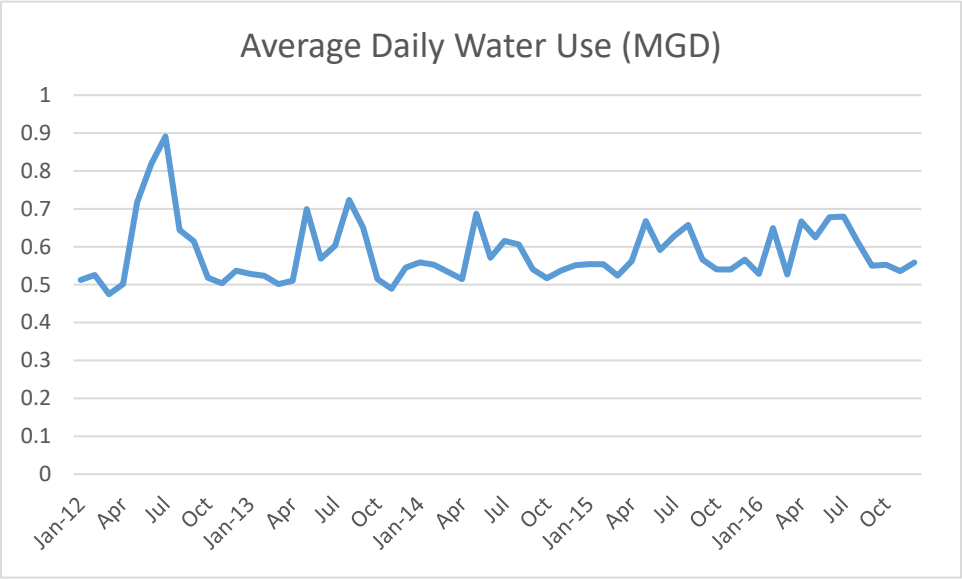
4.2.1. EXISTING WATER USAGE DATA

Existing water usage data was compiled from Public Service Commission (PSC) statistics from 2012-2016. From these datasets, it was possible to obtain both average daily water usage, and maximum daily water usage. The Village's overall average daily water use from 2012-2016 was approximately 584,000 gallons. The peak daily use within this time period was 1.47 MGD. These statistics are further summarized in Figure 4-3 and Table 4-3.

Table 4-3: Maximum Daily Water Usage

Maximum Daily Water Usage 2012-2016			
Year	**Maximum Daily Usage (MGD)**	**Date of maximum**	**Cause of maximum**
2016	1.275	4/22/2016	Flushing hydrants
2015	1.092	5/19/2015	Flushing hydrants
2014	1.427	5/5/2014	Flushing hydrants
2013	1.444	5/20/2013	Flushing hydrants
2012	1.471	5/24/2012	Flushing hydrants

Figure 4-3: Average Daily Water Usage



4.2.2. FIRM CAPACITY ANALYSIS

A firm capacity analysis of the need for future storage and well capacity was completed and is shown in Table 4-4. Firm well capacity is the combined source water supply capacity of a system of municipal wells with the largest unit out of service. Accordingly, all analysis included in this section and in Table 4-4 assumes that the largest well in McFarland is out of service. McFarlands's firm well capacity is 1,500 gpm based on the current operating conditions.

It is necessary to include available water in storage to get an accurate picture of water system performance. In Table 4-4, the combined capacity of wells and storage is referred to as "system capacity". To calculate a total system capacity for this analysis, the effective water storage capacity was used.

Effective water storage capacity is the minimum amount of water in storage under normal operating conditions. This capacity represents all storage facilities being at the low end of their normal operating range, just before calling for the wells to start replenishing storage. For evaluation purposes, effective storage was estimated as 70% (350,000 gallons) of the total storage for the Burma Reservoir and 70% (525,000 gallons) of the total storage for the Holscher Reservoir. Total effective storage volume is 875,000 gallons.

For each design condition, the effective storage is assumed to be fully utilized over the demand period and the system capacity is then the sum of firm well capacity plus the effective storage capacity. The surplus (or deficient) capacity is the difference between the design condition demand and the system capacity. The goal is to have surplus capacity in meeting each of the design

Table 4-4
Firm Capacity Analysis Summary

Year	Village Population	Total Annual Pumpage (1000 gallons)	Average Day Demand (gpm)	Spare Well Capacity during Max Day Demand		Max Day Demand Plus Fire Flow (gpm)	Spare System Capacity during Max Day Demand Plus Fire Flow		Total Pump Running Hours Required to Meet Demand	
				Maximum Day Demand (gpm)	Spare Well Capacity Available (gpm)		Spare Capacity (gpm)	Provided by Largest Well	Day Demand (hrs)	Day Demand (hrs)
2016	8,320	218,304	415	1,022	478	4,022	2,340	43%	7	15.3
2020	8,490	309,885	590	1,356	144	4,356	2,005	50%	9	21.7
2025	8,930	325,945	620	1,426	74	4,426	1,935	52%	10	22.8
2030	9,335	340,728	648	1,491	9	4,491	1,870	53%	10	23.9
2035	9,635	351,678	669	1,539	-39	4,539	1,822	55%	11	24.6
2040	9,895	361,168	687	1,580	-80	4,580	1,781	56%	11	25.3
2050	10,457	381,681	726	1,670	-170	4,670	1,691	59%	12	26.7
2060	10,682	389,886	742	1,706	-206	4,706	1,655	60%	12	27.3

Average Daily Water Pumpage Per Person (gal):	**100.00**
Ratio of Maximum Day to Avergage Day:	**2.30**
ISO Fire Protection Recommendation (gpm):	**3000**
Fire Duration (hours):	**3**
Current Firm Well Capacity (gpm):	**1,500**
Largest Available Well (gpm) (SE):	**1,000**

Burma Reservoir Capacity (gallons):	**500,000**
Burma Reservoir Effective Storage Ratio:	**0.70**
Burma Reservoir Effective Capacity (gallons):	**350,000**
Holscher Reservoir Capacity (gallons):	**750,000**
Holscher Reservoir Effective Storage Ratio:	**0.70**
Holscher Reservoir Effective Capacity (gallons):	**525,000**
Current Total Effective Storage (gallons):	**875,000**

[1] Spare capacity calculated during maximum day demand plus fire event with largest well out of service using all effective storage over the fire demand period.

[2] Spare capacity calculated during maximum day demand plus fire event with largest well out of service and largest reservoir out of service; using all remaining effective storage over the fire demand period.

[3]Data included for the year 2016 is taken directly from actual PSC report data. Forecasts for future years use peak average daily water pumpage per person rates from

conditions, including the worst-case scenario of a fire demand occurring during the period of a maximum daily demand when the largest well is out of service.

The Insurance Services Office (ISO) sets the basic fire flow for each community depending on their individual make-up. The basic fire flow for a municipality is determined as the fifth highest needed fire flow for any location in the municipality. ISO has determined that the basic fire flow for the Village is 3000 gpm at a duration of 3 hours.

The firm capacity analysis shows that if population growth rates are as forecasted, the Village's existing water distribution system has robust storage capacity. The columns most relevant to this conclusion in Table 4-4 have the heading "Space System Capacity during Max Day Demand Plus Fire Flow". There is not a point in the analysis that additional storage in the system will be required on the basis of maximum day demand plus fire flow under firm well capacity constraints.

Included in the analysis as well is the number of total pump runtime hours needed to meet the demands of an average day and the demand on a maximum day under firm well capacity constraints. Generally, it is not recommended to run well pumps in excess of 12 hours per day on a regular basis. This part of the analysis shows that on an average day the wells are not forecasted to pump in excess of 12 hours, but on a day of maximum demand, this constraint could be reached by 2030.

The analysis does show the potential need for the installation of the new Well #5 by the year 2030. The columns most relevant to this conclusion in Table 4-4 have the heading "Spare Well Capacity during Max Day Demand ". This need is on the basis of maximum daily demand without fire flow conditions under firm well capacity constraints. Although the "Maximum Day Demand" values are given in gpm (gallons per minute), they are derived from a peak 24hr-based daily peak value. For this reason it is prudent to compare the maximum daily demand directly to the firm well capacity. By 2030, the maximum day demand is approximately equal to the firm well capacity and there is essentially no remaining spare well capacity.

4.3. RECCOMENDATIONS

4.3.1. CONSTRUCTION OF WELL #5

The firm capacity analysis projects the potential requirement for a new well #5 by 2030. However, this projection is highly dependent on actual development rates and actual peak water usage by the Village. Presently, there appears to be approximately 500 gpm of maximum day pumpage capacity remaining in the system. This is equivalent to the addition of approximately 360 additional homes to the Village.

It takes approximately 3 years to plan and construct a new well. Therefore, it is not imminently necessary that the Village begin detailed planning for the construction of a new well. It is recommended that the Village closely monitor the maximum daily peak water usage on an annual

basis as it rises and begins to approach 1,500 gpm. It is also recommended that the Village plan for this infrastructure improvement in long-term capital infrastructure improvements funding and planning.

4.3.2. POTENTIAL THIRD STORAGE TANK

The firm capacity analysis provides a "snapshot" in time look at the system and assumes that there are no restrictions within the water systems that will slow the delivery of water to a given location. If the analysis shows that there is enough capacity in the system, but a fire occurs in an area with no water main looping, it is likely that demand for fire-fighting will cause the area fed by the singular water main to have inadequate water supply while the rest of the system will have an adequate water supply. For this reason, hydraulic modeling should be used to analyze all future development planning.

Depending on how development progresses on the east side of the Village, it may be advisable to construct a new water tower before elements of the firm capacity analysis would suggest the need for improvements. The distance or disconnectedness of new developments to the existing system may call for an additional storage tank for the sake of system redundancy or maintain system pressures.

Two potential sites for a third storage reservoir have been identified on Figure 4-1. Site #1 is located north of Siggelkow Road, near the intersection with I-39. It should also be noted that this location is the only area within the study boundaries that is unserviceable by the existing water storage reservoirs on the basis of elevation. Site #2 is south of Siggelkow Road, approximately 1,400 feet due south of the end of the cul-du-sac at Lee South Court. It is recommended that the Village take these potential locations into account when planning for new developments in the area of these sites.

APPENDIX A

Attachment: 2015-08-13-R1

RESOLUTION NO: 1

WHEREAS, plans for sanitary sewer extensions were submitted to the Madison Metropolitan Sewerage District (MMSD) for Holscher Road Lift Station and Force Main, in the Village of McFarland, on August 4, 2015, by Gregory Droessler, Town & Country Engineering, Inc., and

WHEREAS, the project consists of installing a lift station and non-sewered storage building on Outlot 1 of the plat of Juniper Ridge, 122 feet of 8-inch gravity sewer within Outlot 1 of the plat of Juniper Ridge, and 1,780 feet of 6-inch force main on Holscher Road and Linden Parkway to connect to the Village of McFarland's sewerage facilities, and

WHEREAS, the proposed lift station is designed to provide service to the developments of Juniper Ridge and Prairie Place, and

WHEREAS, the proposed developments of Juniper Ridge and Prairie Place naturally drain to the south, and

WHEREAS, the proposed sanitary sewer extension will convey flows to the north to connect to the Village of McFarland's sewerage facilities and to MMSD's Southeast Interceptor/Siggelkow extension, and

WHEREAS, MMSD wrote a letter to the Village of McFarland dated July 8, 2008, stating that MMSD would allow the subject lift station to pump to MMSD's Siggelkow Interceptor provided that the maximum rate of discharge not exceed 500,000 gallons per day and that the lift station be relieved when gravity sewer service is available from the south, and

WHEREAS, plan review fees are due, and

WHEREAS, interceptor and treatment plant connection charges for the lands to be served will be determined with future phases of development, and

WHEREAS, the MMSD has received a sewer extension review letter #6-15-44 from the Capital Area Regional Planning Commission (CARPC) dated July 27, 2015 stating the CARPC staff has reviewed the proposed extension and determined that it is consistent with the urban service area provisions of the Dane County Water Quality Plan and the Dane County Land Use and Transportation Plan, and

WHEREAS, the Chief Engineer and Director of MMSD has recommended the approval of said plans,

THEREFORE, BE IT RESOLVED, that the plans for sanitary sewer extensions for Holscher Road Lift Station and Force Main, in the Village of McFarland, submitted on August 4, 2015, by Gregory Droessler, Town & Country Engineering, Inc., be approved subject to the following conditions:

1. That the Village of McFarland shall pay the plan review fee of $1,250.00 to the MMSD within 30 days of receiving an invoice for the fee.

2. That the maximum rate of discharge from the lift station shall not exceed 500,000 gallons per day. The Village shall install such equipment and controls necessary to ensure that said flowrate cannot be exceeded.

3. That the Village shall provide to MMSD copies of the certified pump curves and the results of any factory or field testing performed prior to start-up of the lift station.

4. That the Village shall provide MMSD staff access to the flow metering equipment at the lift station to verify the pumping flowrate.

5. That the Village of McFarland shall consider this facility to be a temporary lift station, which will be relieved when gravity sewer service is available from the south. Future planning performed by the Village of McFarland shall include strategies to establish a gravity sanitary sewer line to relieve this lift station.

6. That if sewer service is established to any buildings constructed on Outlot 1 of the plat of Juniper Ridge, the Village of McFarland shall be responsible for payment of all appropriate MMSD connection fees.

7. That the Village of McFarland shall require the presence of an engineer or an inspector on the construction of said sewer extensions who shall make a record showing in detail the construction as built and shall furnish the MMSD with as-built plans in the case of any significant deviations from the original design.

8. That the construction, operation, maintenance and use of said sewer extensions shall be in accordance with the lawful rules and regulations of the MMSD and with the applicable State of Wisconsin plumbing and sewerage codes.

9. That if construction of the sewer extensions covered by said plans has not commenced within four years following the approval date of this resolution, this approval shall become void and new plans shall be submitted. If construction has commenced but has not been completed within the four-year approval period, the unbuilt portion must be

submitted for reapproval if it is a significant portion of the project and would require DNR approval.

The above resolution was adopted by the
Commissioners of the Madison Metropolitan
Sewerage District at their meeting held in
the District office on August 13, 2015.

APPENDIX B

MEMORANDUM

Date: May 29th, 2015

To: Mr. Allan Coville
Director of Public Works/Utilities
Village of McFarland, WI

From: Adam Groshek
Town and Country Engineering, Inc.

Subject: Lift Station No. 2 Remaining Capacity study

The investigation of the incoming flows to lift station No. 2 (LS No. 2) during the fall of 2014 and the spring of 2015 were valuable in determining the approximate remaining capacity of LS No. 2. The monitoring verified that previous estimates of available capacity were conservative and the lift station still has the ability to accept more sanitary sewer flows.

LS No. 2 has two pumps that alternate pumping wastewater every other cycle into an 8″ forcemain. They were designed to pump at minimum 600 gpm with 56′ of total dynamic head (TDH). The drawdown tests that were performed in 2014 and 2015 showed that the pumps were able to exceed this design flow during all the tests. On average one of the pumps pumped 860 gpm and the other pumped 740 gpm. The minimum pumpage rate tracked in fall 2014 and spring 2015 was 680 gpm. These drawdown tests were also checked by estimating the flow being discharged from the forcemain in its discharge manhole. Peak velocities recorded were approximately 5.2 feet per second, which results in a flow of about 790 gpm in the 8″ ductile iron forcemain flowing full. For the purpose of this evaluation, 680 gpm will be used as the capacity of LS No. 2.

If the pumps do indeed pump at a higher flow of 740-860 gpm on a regular basis, that only provides more capacity. It is not known exactly why the measured flows differed so much between the two pumps, but it is expected that impeller wear or differences within the internals of the pumps is the leading cause. It is recommended that replacement SCADA and a level sensor in the LS No. 2 wetwell could be used to further refine the pump rates of the lift station pumps during all conditions.

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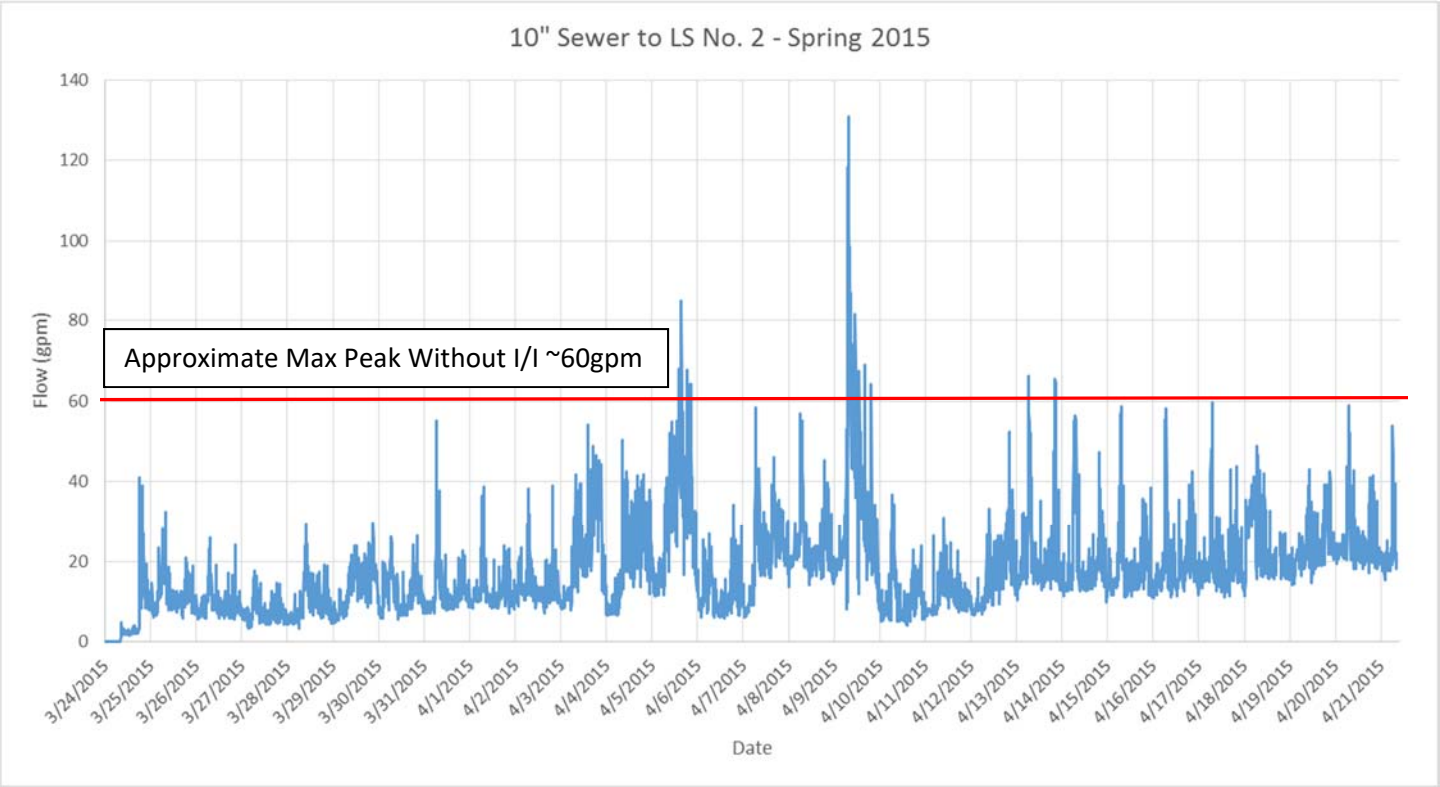
The last time a study was done on the influent wastewater to LS No. 2 was in 2000. It was found that influent wastewater peak flows from the western 10″ sewer were relatively high because of infiltration/inflow (I/I) issues. Flows from the eastern 15″ sewer were relatively low because of the limited amount of development to the east at the time.

In this study, peak incoming flows were measured in the two gravity sewers flowing into the wetwell in fall 2014 and spring 2015. The period during fall 2014 was relatively dry and limited I/I was observed. It was decided to continue the study in the spring of 2015 in order to capture some to the spring runoff flows. The peak flow data for these two lines, 15″ and 10″ in diameter, are shown in Table 1.

Table 1
LS No. 2 Influent Peak Flows

	Previous Studies	**Fall 2014 Monitoring**	**Spring 2015 Monitoring**	**Recorded Peak Flow**
15" San. Sewer E of LS No. 2	68 gpm	190 gpm	135 gpm	**190 gpm**
10" San. Sewer N of LS No. 2	182 gpm	20 gpm	131 gpm	**182 gpm**
Remaining LS No. 2 Capacity	430 gpm	470 gpm	414 gpm	**308 gpm**

The 2015 spring monitoring showed peak flows occurring during rain events that had I/I clearwater add to the sanitary flow. A peak flow of 135 gpm from the 15″ sewer was measured of which 1-20 gpm is estimated to be from I/I. The sanitary sewer coming into the lift station from the north is considerably older, and therefore it is expected to have more I/I. A peak of 131 gpm from the 10″ sewer was measured of which 60-80 gpm is estimated to have occurred due to I/I. These I/I estimates were found by looking at the gathered flow data from the monitoring that occurred on the two influent sewer lines to LS No. 2. The graphs in Figure 1 show this flow data relative to the major rain event of 1.8 inches that occurred on 4/9/15. Daily maximum flow in the sewers tended to follow the twice daily “diurnal” pattern of sanitary sewers with peak flows around 7am (before work) and 7pm (after work). The 7am peak each day tends to be the higher of the two peaks and therefore a rain event that occurs during the morning hours tends to result in the overall peak flow. Figure 1 also shows this peak in the 10″ sewer as most of the 1.8 inches on 4/9/15 fell in the morning hours.



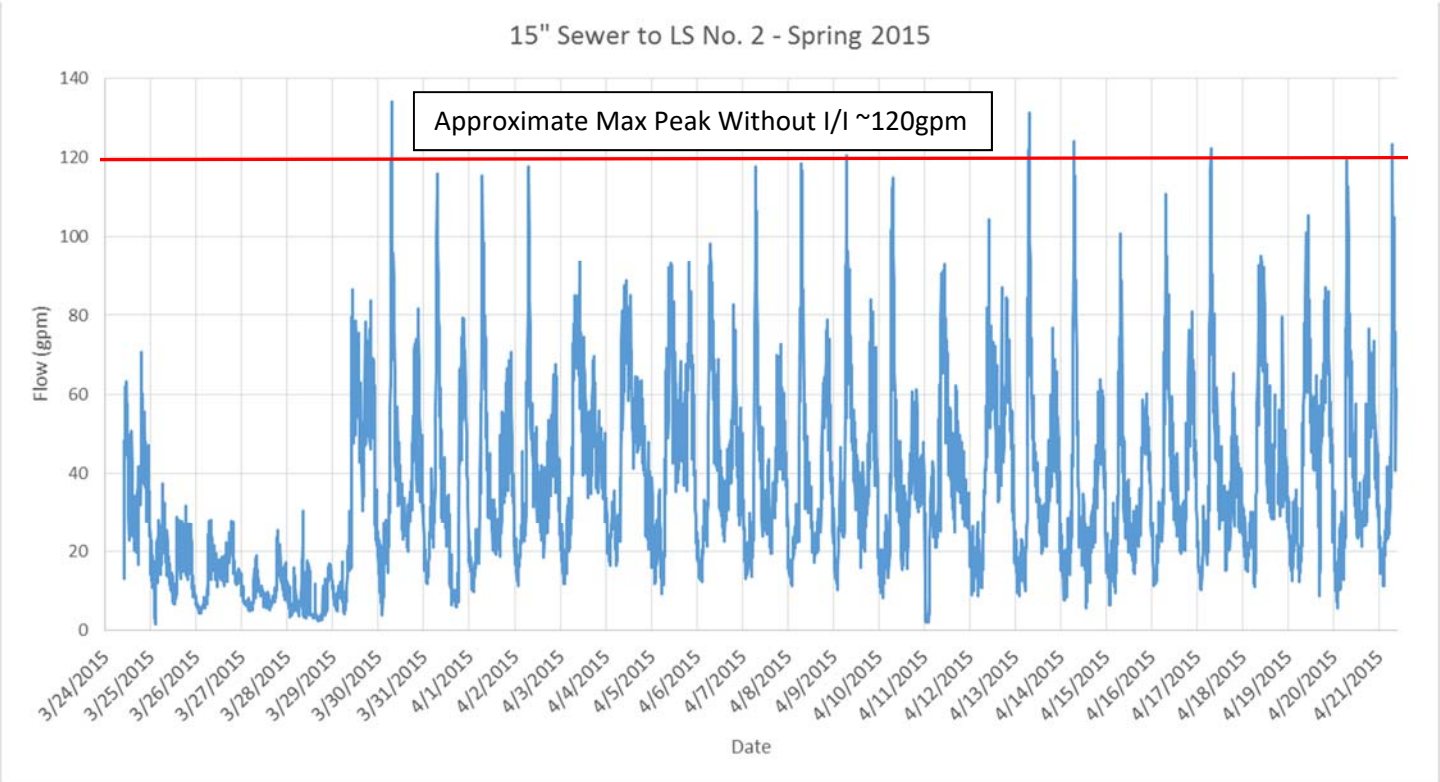


Figure 1: Graphs of the LS No. 2 Influent Flows

I/I is not visibly present in the graph of the 15″ sewer to LS No. 2. It is expected that because the sewers draining to the 15″ interceptor to LS No. 2 are relatively new, little I/I clearwater exists for the time being from the subdivisions east of LS No. 2. I/I flows from the subdivisions east of LS No. 2 may increase as the sewer infrastructure ages.

The sewers and manholes observed in the study appeared to be in relatively good condition but it should be noted that all the manholes that were opened had water stains around the inside barrel of the manholes due to inflow of rainwater under the unsealed manhole castings. Future I/I may be avoidable with better sealing of the sanitary sewer system, especially underneath the manhole cover castings. It is expected that there will always be at least a small amount of I/I entering the sanitary sewer, which is why the factors of safety are built in.

There are about 125 household units with wastewater discharging to the 10″ sewer into LS No. 2. This service area has no future growth expected unless the land use of it changes. As of May 2015, there are also about 300 household units discharging to LS No. 2 through the 15″ sewer. Assuming 300 gallons per household per day (which includes some I/I flow) and a peaking factor of 3.5, it is expected that each additional household connected to the sewer and discharging to LS No. 2 could contribute 0.73 gpm to the LS No. 2 wet well. The present flow rate per household based on recent spring 2015 monitoring is 0.63 gpm. This results in the ability to connect at least 400 additional households to discharge to LS No. 2. While this is a conservative estimate based on current conditions of the sanitary sewer leading up to LS No. 2, I/I impacts on any future sewer will always have to be taken into consideration. An illicit connection or a faulty manhole may greatly affect these flows.

Using all of the above, the data indicates LS No. 2 can handle another 308 gpm. It is important to keep in mind that the peak data collected for the two sewer lines coming into LS No. 2 show one-time events. Future peak flows may be higher depending on conditions. The monitoring events are limited so a factor of safety should be applied for the unknown events. For the purpose of this study, it is recommended to limit future additional capacity to 300 gpm to minimize the risk of overloading LS No. 2.

It is recommended that the pump controls be upgraded to gallons flow values (calculated in PLC programming) in order to easily pick out the peak I/I events. It is also recommended that a pressure gauge be installed on the discharge of the LS No. 2 pumps to be able to track the head that the lift station pumps

are pumping against. There is the option to either install one gauge on the main discharge header or one gauge on each pump discharge. A gauge on the main header would involve tapping into the pipe under pressure while the discharge piping from each pump could be separately isolated with the valves to avoid sewage from flowing back. Current flows from the pumps are higher than design most likely because the current friction and elevation head losses are less than the design TDH of 56'. A pressure gauge would allow one to be able to see the head that each pump is pumping against, and using a pump curve, predict the expected flow in the forcemain.

Continuous and future monitoring may support the suggested 400 additional household units and it is likely that future monitoring could show that additional future capacity exists beyond the 400 additional household units. The following recommendations should be taken into consideration as they will allow the capacity of LS No. 2 to be more accurately tracked. They do not need to happen immediately but only as necessary to track the capacity of LS No. 2:

1. Upgrade SCADA to add a pressure sensor to verify the pump head pressure during all times of operation. This will be important to verify the pump capacity during all pumping conditions.
2. Add logic and level sensor to calculate influent flows and pump capacity.
3. Have the qualified pump technician perform a maintenance checkup on the LS No. 2 pumps to identify wear of the impellers and to try to determine the cause of the large difference between flows between the two LS No. 2 pumps.

It is hard to predict exactly what will happen in the future as the Village of McFarland expands eastward. It is known that LS No. 2 has adequate capacity for developments to be added onto the sanitary sewer network discharging to LS No. 2. With proper planning and construction practices, LS No. 2 may be able to handle 400 additional household units being added onto its sewer drainage basin before a forcemain and/or pump upgrade will be needed.

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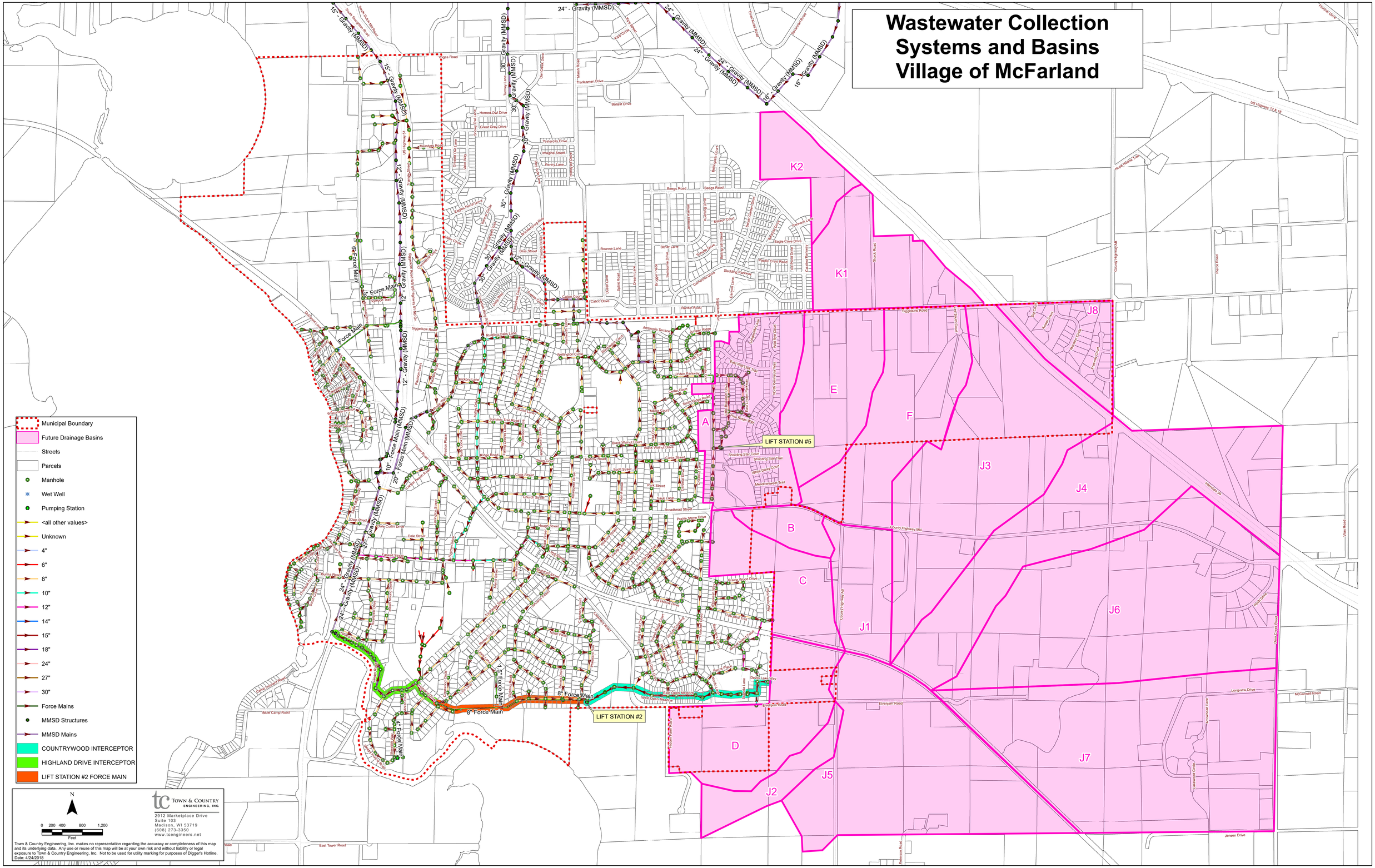
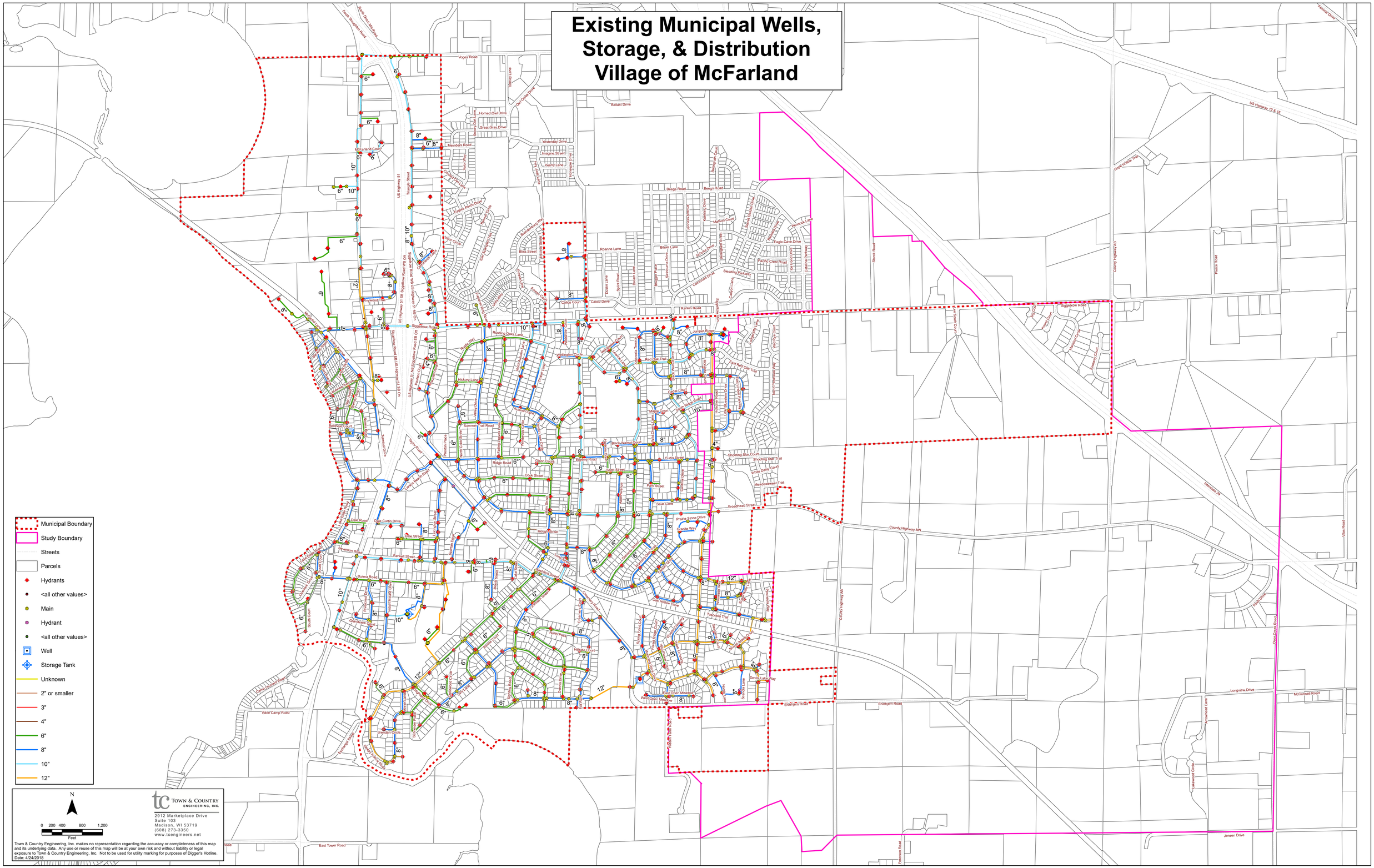
APPENDIX C

Wastewater Collection Systems and Basins Village of McFarland



APPENDIX D

Existing Municipal Wells, Storage, & Distribution Village of McFarland