Northern Moraine Wastewater Reclamation District

2014 Facility Plan Update

Continuity • Collaboration • Commitment

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LIST OF ABBREVIATIONS

ABBREVIATION	DESCRIPTION
avg	average
BOD_5	5-day biochemical oxygen demand
DMA	Designated Management Agency
DO	dissolved oxygen
FPA	Facility Planning Area
FRSG	Fox River Study Group
gal	gallons
gcd	gallons per capita per day
gpd	gallons per day
gpm	gallons per minute
GIS	Geographical Information System
HP	horsepower
IEPA	Illinois Environmental Protection Agency
I/I	infiltration and inflow
lbs	pounds
l.f.	lineal feet
mg/L	milligrams per liter
MGD	million gallons per day
NH_3-N	ammonia nitrogen
NMWRD	Northern Moraine Wastewater Reclamation District
NPDES	National Pollutant Discharge Elimination System
PE	population equivalent
SSES	Sanitary Sewer Evaluation Study
TSS	total suspended solids
USEPA	United States Environmental Protection Agency
VFD	variable frequency drive
WQBEL	water quality based effluent limit
WQS	water quality standard
WRD	Wastewater Reclamation District
WWTF	wastewater treatment facility
yr	year

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EXECUTIVE SUMMARY

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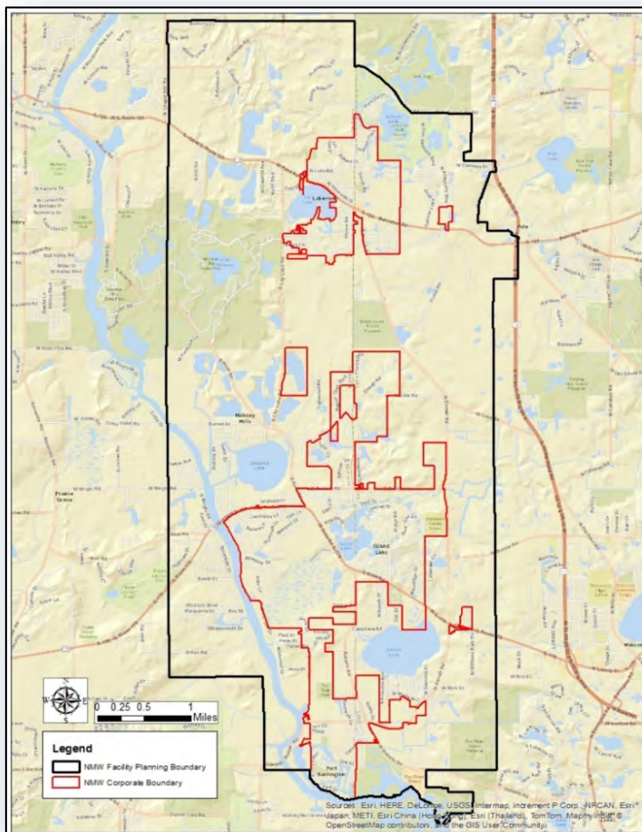
EXECUTIVE SUMMARY

INTRODUCTION AND BACKGROUND

The Northern Moraine Wastewater Reclamation District (NMWRD) is the Designated Management Agency (DMA) for all areas within its Facility Planning Area (FPA). The Northern Moraine FPA is shown on Exhibit 1, and includes approximately 16,700 acres of which 3,700 acres are currently annexed to the District, including 2,400 acres that have been developed.

Exhibit 1: NMWRD Facility Planning Area



The NMWRD currently provides wastewater collection and treatment services to the Villages of Island Lake, Lakemoor, and Port Barrington. Portions of the Village of Lake Barrington and Holiday Hills are also located within the Northern Moraine FPA but are not currently served or connected to the system.

The existing NMWRD wastewater treatment facility was originally constructed in 1978 and operates under the requirements of NPDES Permit No. IL0031933. The treatment facility was originally designed to treat an average flow of 1.20 MGD. Since the facility was constructed it has been expanded and improved several times. Long-range planning has also been conducted on multiple occasions over the past 35 years.

Facility planning was completed in 1998 that projected a service population increase from 10,000 PE in 1998 to approximately 30,000 PE in the year 2020. Construction occurred that increased the capacity of the treatment facility to accommodate up to 20,000 PE, which was designed to facilitate the subsequent expansion to 30,000 PE. The second phase expansion was never constructed.

An update to the previous facility planning reports was prepared in 2004 to reflect the unprecedented level of growth that was occurring within the Northern Moraine FPA. Population projections were prepared based on the then current Comprehensive Plans for each community served by the District, and the updated plan called for phased expansions to the treatment facility up to an ultimate capacity of 10 MGD.

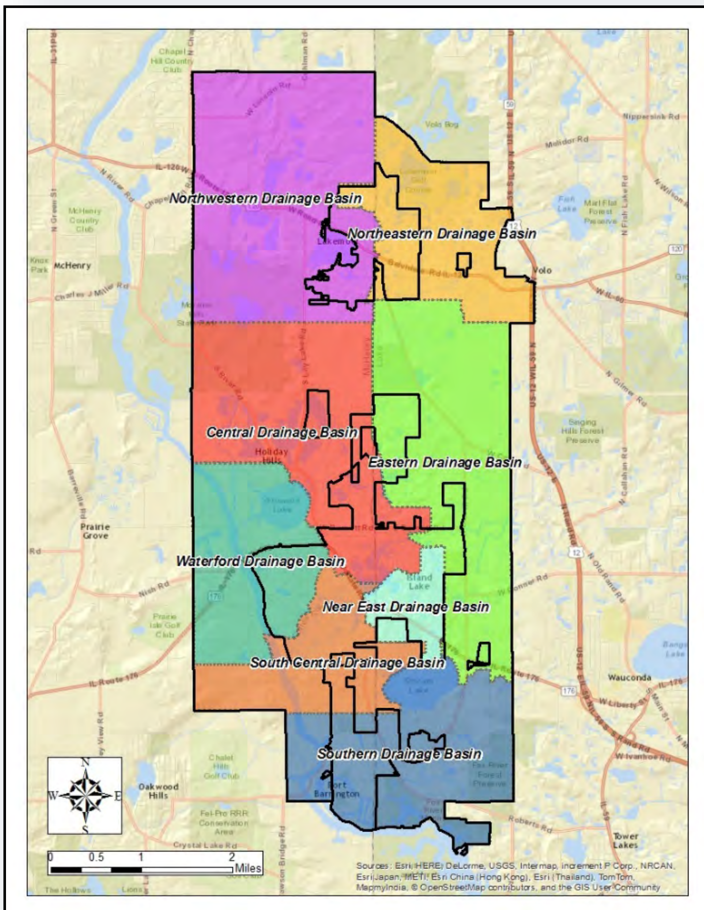
The housing market slowed in 2006 and crashed in 2008. Growth within the Northern Moraine FPA has stagnated over the latter half of the past decade, and in response each community revised their Comprehensive Plans to reflect current economic conditions and scale back long-term projections for growth. The current plans are less aggressive and more consistent with long-term historical growth trends in Lake and McHenry Counties. This report updates the previous facility plans to be consistent with the current Comprehensive Plans of each community, and was prepared using more complete and accurate demographic data contained in the previously unavailable Geographical Information System (GIS).

WASTEWATER DRAINAGE BASINS

The areas within the Northern Moraine FPA were divided into eight wastewater drainage basins for facility planning purposes as shown on Exhibit 2. The exhibit shows the current facility planning area as well as the corporate boundary (shown in black).

These drainage basins were delineated because each is unique in regards to which community they are associated with, current level of development, the potential for future development, topography, and the existence or lack of existing collection system sewers, lift stations, and force mains.

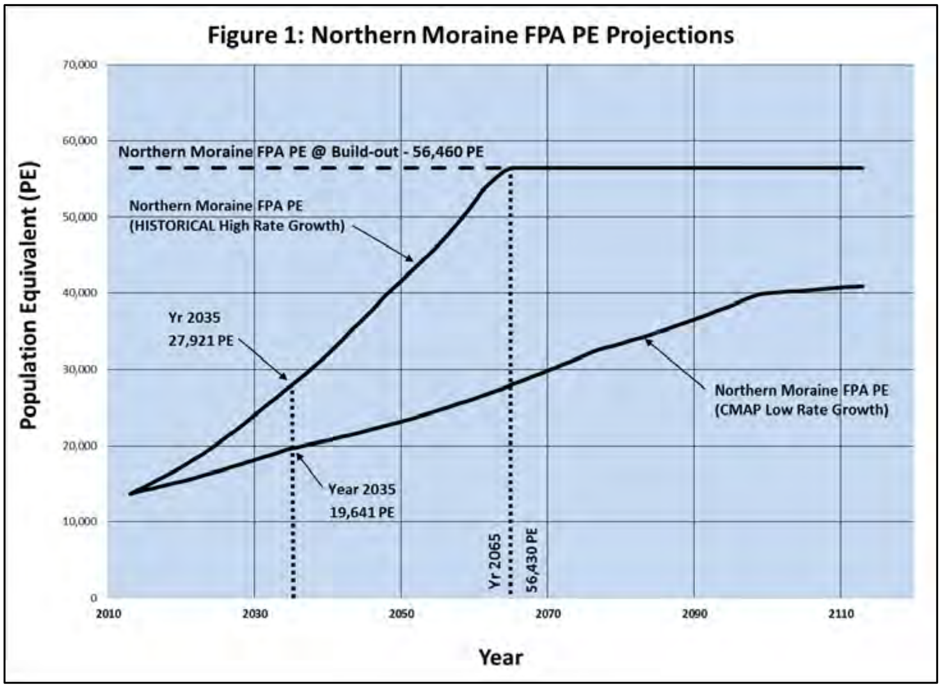
Exhibit 2: NMWRD Drainage Basins



The Northwestern and Northeastern Drainage Basins generally comprise the areas surrounding Lakemoor to the west and east, respectively. The Central and Eastern Drainage Basins comprise those areas generally north of Island Lake and south of Lakemoor. The Waterford Drainage Basin includes mostly developed properties west of Island Lake and along the Fox River, including the Village of Holiday Hills. The Near East Drainage Basin comprises the east side of Island Lake and the South Central Basin is home to the NMWRD wastewater treatment facility. The Southern Drainage Basin includes the Village of Port Barrington and other areas to the east and west of that Village.

POPULATION AND WASTEWATER FLOW PROJECTIONS

The 20-year planning projections performed as part of this facility planning effort indicate that the NMWRD service population will grow from the current 13,695 PE to between 19,641 PE and 27,921 PE based on low rate and high rate growth projections, respectively. It is also projected that the ultimate service population at full build-out of the FPA will reach approximately 56,460 PE. Overall 100-year PE projections are shown on Figure 1.



Current and projected service populations for the 20-year facility planning period and at full build-out of the entire Northern Moraine FPA are summarized by drainage basin in Table 1.

Table 1: Current and Projected PE Breakdown by Basin

Drainage Basin	Existing PE	Year 2035 Low Rate Growth PE	Year 2035 High Rate Growth PE	Ultimate Build-Out PE
Central Drainage Basin	3,155	5,425	5,425	5,425
Eastern Drainage Basin	69	116	441	5,323
Near East Drainage Basin	2,235	2,663	3,487	3,487
Northeastern Drainage Basin	1,912	3,222	4,828	14,587
Northwestern Drainage Basin	1,618	2,726	4,085	7,243
South Central Drainage Basin	378	422	996	2,085
Southern Drainage Basin	1,197	1,336	3,152	11,489
Waterford Drainage Basin	3,131	3,731	5,507	6,821
Total PE	**13,695**	**19,641**	**27,921**	**56,460**

An analysis was completed which reviewed the District's current commitments for areas within the corporate boundary, as well as areas outside the corporate limits but within the facility planning area. A breakdown of projected population equivalents is summarized in Table 2.

Table 2: Population Equivalent Projections

	Incorporated FPA	Unincorporated FPA	Built-Out FPA
Existing Residential PE	12,818	0	
Future Residential PE Growth	1,878	32,910	
Total Projected Residential PE	**14,696**	**32,910**	**47,606**
Existing Non-Residential PE	877	0	
Future Non-Residential PE Growth	757	7,220	
Total Projected Non-Residential PE	**1,634**	**7,220**	**8,854**
Total Projected PE	**16,330**	**40,130**	**56,460**

The current and projected wastewater flows and pollutant loadings are summarized in Table 3.

Table 3: Current and Projected Influent Wastewater Flows and Loadings

Condition	PE	Average Flow (MGD)	Per Capita Flow (gcd)	BOD_5 (lbs/day)	Per Capita BOD_5 (lbs/PE/d)	TSS (lbs/day)	Per Capita TSS (lbs/PE/d)	NH_3-N (lbs/day)
Design	20,000	2.0	100	2,800	0.14	3,370	0.17	417
Current Conditions (2013– 2014)								
Current	13,695	1.05	77	1,793	0.13	1,595	0.12	227
Percent of Design	68%	53%	77%	64%	93%	47%	71%	54%
Projected Conditions at Build-out of NMWRD Incorporated Area								
Projected	16,338	1.3	80	2,242	0.14	2,124	0.13	282
Percent of Design	82%	66%	80%	80%	100%	63%	76%	68%
Projected 20-year Conditions within NMWRD FPA - HIGH RATE GROWTH								
Projected	27,921	2.47	88	4,211	0.15	4,440	0.16	524
Percent of Design	140%	124%	88%	150%	107%	132%	94%	126%
Projected 20-year Conditions within NMWRD FPA - LOW RATE GROWTH								
Projected	19,641	1.64	84	2,804	0.14	2,728	0.14	351
Percent of Design	98%	82%	84%	100%	100%	83%	82%	84%
Projected Conditions at Build-out of NMWRD FPA								
Projected	56,460	5.3	94	9,063	0.16	10,148	0.18	1,125
Percent of Design	282%	266%	94%	324%	114%	301%	106%	270%

The existing NMWRD treatment facility has sufficient capacity to treat the projected wastewater flows and pollutant loads at build-out of the incorporated areas. However, development of unincorporated properties within the FPA will require expansion(s) of the NMWRD facility.

The existing treatment facility is designed for an average flow of 2.0 MGD. When the influent flow reaches 80 percent of design, or 1.6 MGD, the District should begin planning for the expansion of the facility. It is projected that flows will not reach 80 percent of design for at least another 10 to 20 years based on high rate and low rate growth projections, respectively.

COLLECTION SYSTEM

The current and projected population equivalents in each drainage basin were used to estimate peak wastewater flows in the interceptor sewers. The interceptors that were evaluated included:

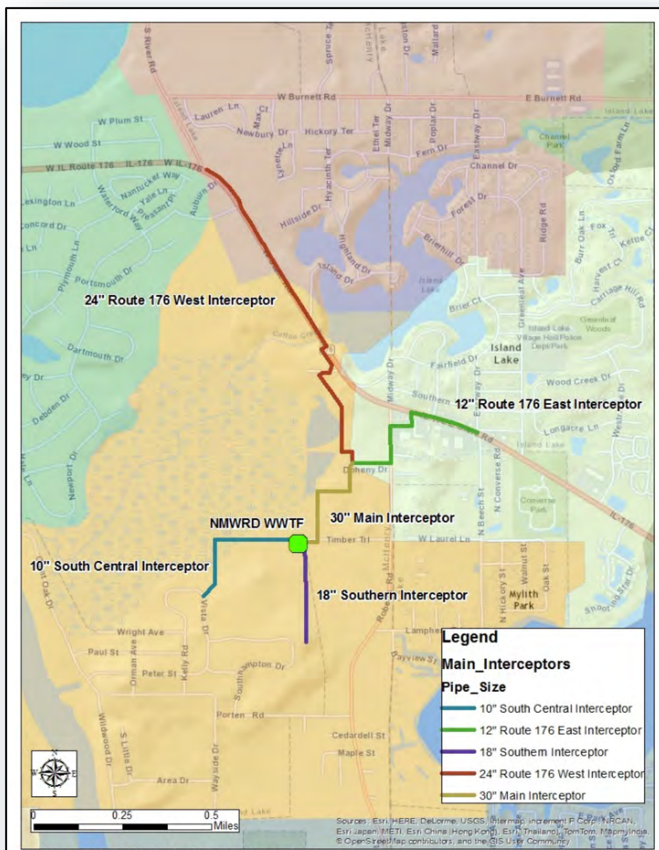
- Main Interceptor (30-inch)
- Route 176 West Interceptor (24-inch)
- Route 176 East Interceptor (12-inch)
- Southern Interceptor (18-inch)
- South Central Interceptor (10-inch)

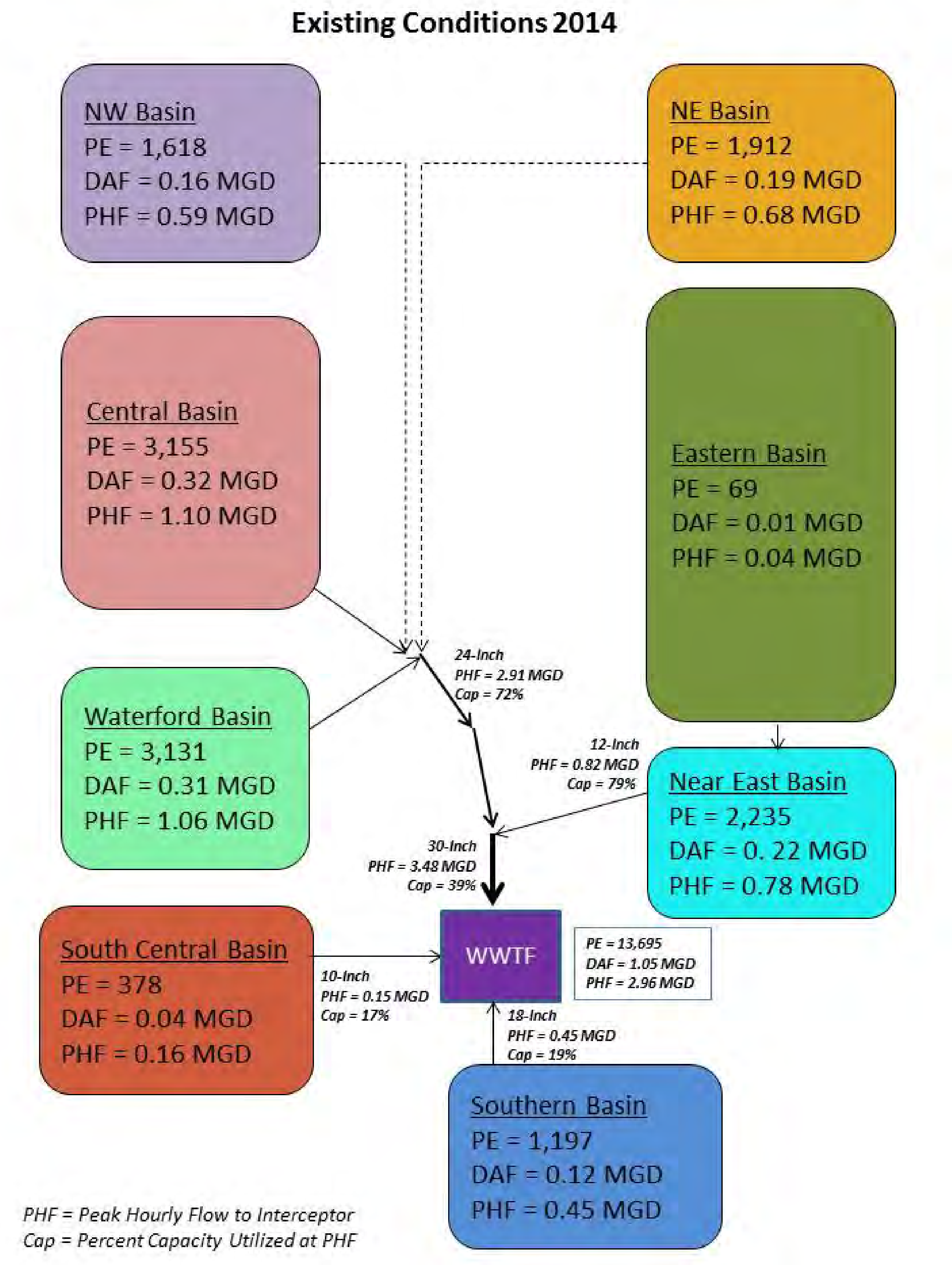
It was found that all of the existing interceptors have sufficient capacity to convey the current peak flows.

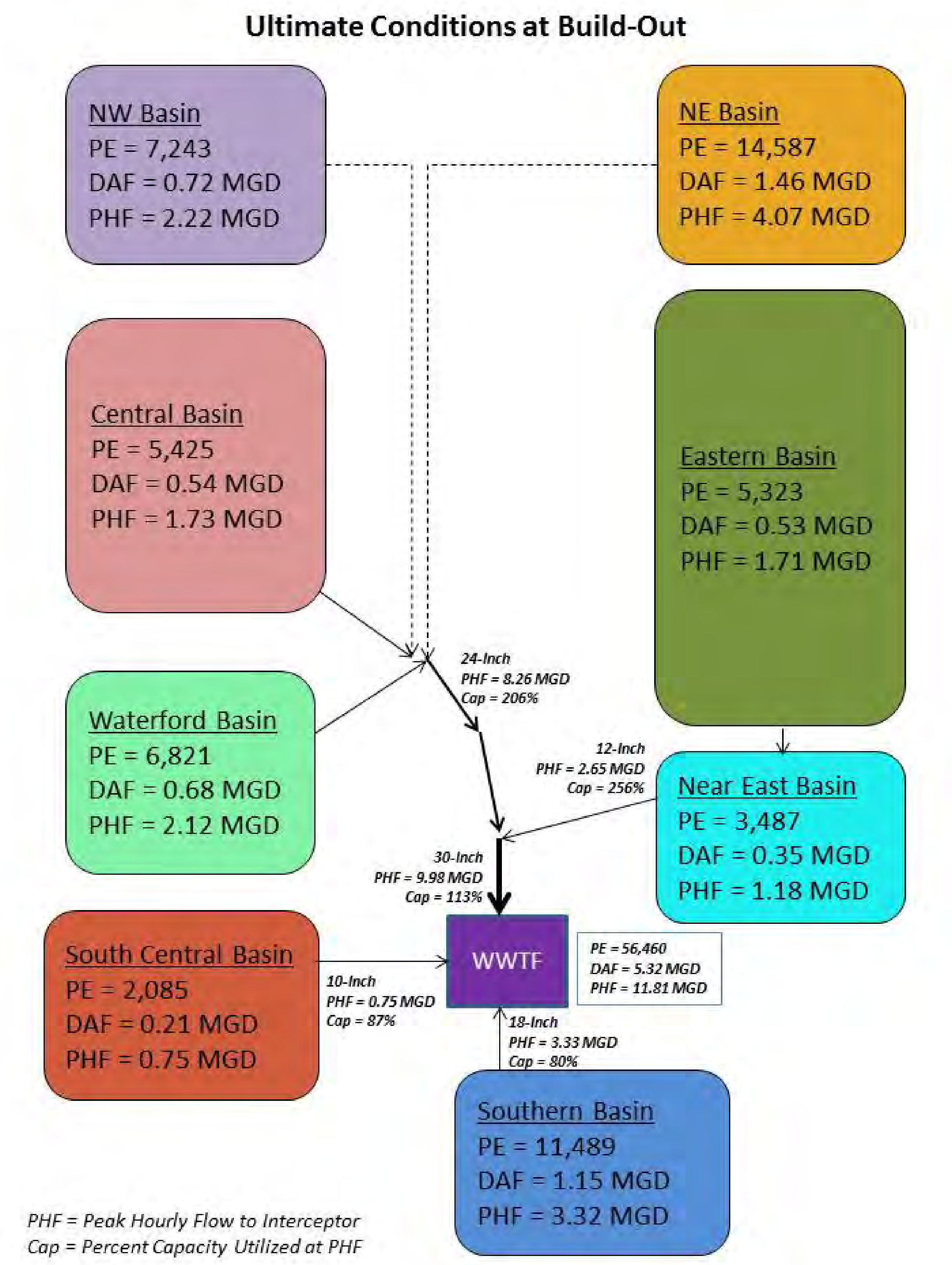
On the following page, a simplified basin flow diagram is shown which illustrates the inter-relationship between the basins and these interceptor sewers under existing conditions.

Page 7 provides a similar flow diagram showing the impacts on the existing infrastructure under build-out conditions within the FPA. Under future conditions the interceptor sewer which serve the Southern and South Central Basins are adequately sized. However, under the current configuration of the collection system, the 30-inch Main Interceptor, and the 24-inch and 12-inch Route 176 Interceptors will all become overloaded in the future. This collection system serves the Waterford, Central, Northwestern, Northeastern, Eastern and Near East Basins.

Exhibit 3: NMWRD Interceptor Sewers




Existing Conditions 2014
NW Basin
PE = 1,618
DAF = 0.16 MGD
PHF = 0.59 MGD
NE Basin
PE = 1,912
DAF = 0.19 MGD
PHF = 0.68 MGD
Central Basin
PE = 3,155
DAF = 0.32 MGD
PHF = 1.10 MGD
Eastern Basin
PE = 69
DAF = 0.01 MGD
PHF = 0.04 MGD
24-Inch
PHF = 2.91 MGD
Cap = 72%
Waterford Basin
PE = 3,131
DAF = 0.31 MGD
PHF = 1.06 MGD
12-Inch
PHF = 0.82 MGD
Cap = 79%
Near East Basin
PE = 2,235
DAF = 0. 22 MGD
PHF = 0.78 MGD
30-Inch
PHF = 3.48 MGD
Cap = 39%
South Central Basin
PE = 378
DAF = 0.04 MGD
PHF = 0.16 MGD
10-Inch
PHF = 0.15 MGD
Cap = 17%
WWTF
PE = 13,695
DAF = 1.05 MGD
PHF = 2.96 MGD
18-Inch
PHF = 0.45 MGD
Cap = 19%
Southern Basin
PE = 1,197
DAF = 0.12 MGD
PHF = 0.45 MGD
PHF = Peak Hourly Flow to Interceptor
Cap = Percent Capacity Utilized at PHF


Ultimate Conditions at Build-Out
NW Basin
PE = 7,243
DAF = 0.72 MGD
PHF = 2.22 MGD
NE Basin
PE = 14,587
DAF = 1.46 MGD
PHF = 4.07 MGD
Central Basin
PE = 5,425
DAF = 0.54 MGD
PHF = 1.73 MGD
Eastern Basin
PE = 5,323
DAF = 0.53 MGD
PHF = 1.71 MGD
24-Inch
PHF = 8.26 MGD
Cap = 206%
Waterford Basin
PE = 6,821
DAF = 0.68 MGD
PHF = 2.12 MGD
12-Inch
PHF = 2.65 MGD
Cap = 256%
Near East Basin
PE = 3,487
DAF = 0.35 MGD
PHF = 1.18 MGD
30-Inch
PHF = 9.98 MGD
Cap = 113%
South Central Basin
PE = 2,085
DAF = 0.21 MGD
PHF = 0.75 MGD
WWTF
PE = 56,460
DAF = 5.32 MGD
PHF = 11.81 MGD
10-Inch
PHF = 0.75 MGD
Cap = 87%
18-Inch
PHF = 3.33 MGD
Cap = 80%
Southern Basin
PE = 11,489
DAF = 1.15 MGD
PHF = 3.32 MGD
PHF = Peak Hourly Flow to Interceptor
Cap = Percent Capacity Utilized at PHF

Main Interceptor (30-inch)

The Main Interceptor currently flows at approximately one-third capacity but will eventually become slightly overloaded in its upper reaches at full build-out. The Main Interceptor could be off-loaded in the future if an Eastside Collection System were constructed. Flow from the Northeastern and Eastern Basin would be diverted into the proposed 42-inch Treatment Plant Interceptor extending from the NMWRD treatment facility to the Water's Edge Lift Station, also allowing the lift station to be retired and abandoned at that time.

Route 176 West Interceptor (24-inch)

The most critical restrictions will exist in the 24-inch Route 176 West Interceptor. This sewer is installed at flatter grades along Route 176 than in its downstream reaches. An in depth analysis of this sewer was performed, including review of field invert data to determine the true capacity in comparison with an estimate based on regulatory minimum pipe slope.

It is estimated that the reach along Route 176 (2,320 l.f.) is currently operating at approximately 70 percent capacity. A more accurate estimate of utilization could be accomplished through flow metering and modeling. It should be noted that one segment of the interceptor (218 foot) is installed at less than minimum slope. This segment should be monitored and cleaned as needed.

The Route 176 West Interceptor has capacity to serve an additional 4,425 PE. This sewer receives flow from four basins (Waterford, Central, Northwestern and Northeastern Basins). In total, these four basins have the potential to contribute an additional 24,260 PE. Clearly this upper reach of the Route 176 West Interceptor is not able to support build-out of all four basins.

The Waterford Basin is expected to expand by 3,690 PE. The Central Basin at build-out will produce an additional 2,270 PE. Combined, these two basins equate to 5,960 PE, which exceeds the currently available capacity of 4,425 PE. Therefore, the Northwestern and Northeastern Basins need to be removed from the upper reach of this interceptor to allow for full build-out of the Waterford and Central Basins.

The Northwestern Basin has potential to contribute an additional 5,625 PE, for a total of 7,243 PE at build-out. A portion of this basin is served by Lift Station 1. Lift Station 1 has adequate capacity to serve its tributary area, which appears to be fully built-out. Section 3 outlines a series of recommendations for development of the remaining sub-basins in the Northwestern Basin. These recommendations include construction of a new lift station and reuse of the existing 8-inch and 12-inch force mains along Lily Lake and River Roads. The 12-inch force main is currently utilized by Lift Station 7 which serves the Northeastern Basin. As indicated above, these force mains from the Northwestern Basin will need to be extended to the lower reaches of the Route 176 West Interceptor to allow full build-out of the Northwestern Basin and not just only the Central and Waterford Basins.

The Northeastern Basin could produce an additional 12,675 PE, which includes 3,210 PE currently served by the Rockwell Development utilities. As previously stated, the Route 176 West Interceptor only has capacity for an additional 4,425 PE. Ultimately, additional capacity must be created to convey future flows from the Northeastern Basin to the treatment facility.

Northeastern Basin - Lakemoor Lift Station 7 Force Main

The 12-inch force main from Lakemoor Lift Station 7 was evaluated. Lakemoor Lift Station 7 currently has surplus capacity to connect an additional 1,480 PE, equivalent to 540 residential units at 2.74 PE/unit.

The capacity of Lakemoor Lift Station 7 can be increased by installing larger pumping units. Wastewater sewage pumps are limited to about 200 feet of head. Any increase in capacity beyond that would require the installation of parallel force mains and/or intermediate booster lift stations, neither of which are desirable.

Lift Station 7 currently has two 800 gpm pumps and was designed for the installation of a third pump. If a third pump were installed, it is estimated that the lift station would be capable of pumping 1,200 gpm, which equates to an additional 3,460 PE. This needs to be verified with a more in-depth hydraulic analysis.

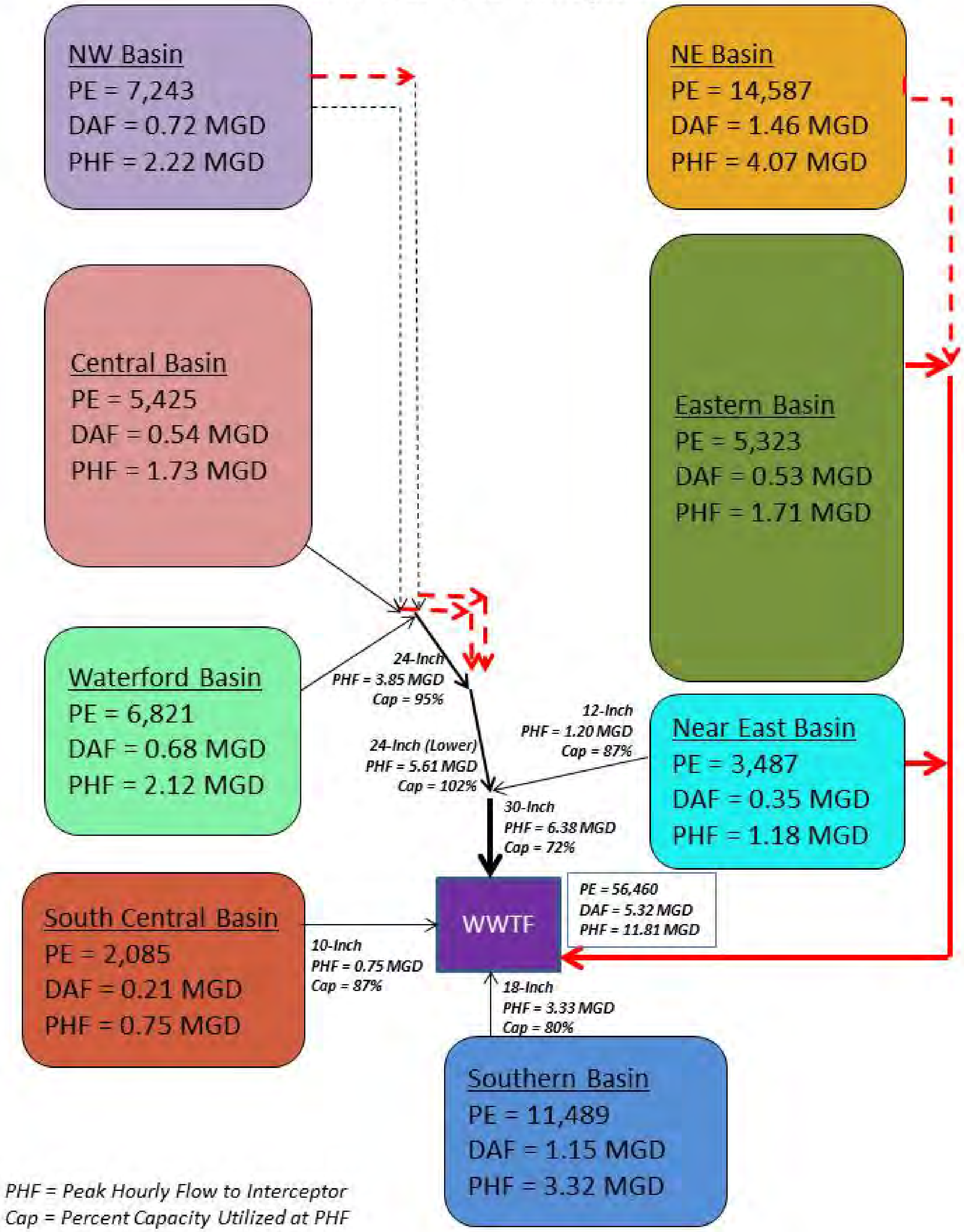
At Lift Station 7, the capacity could be increased to as high as 2,000 gpm without exceeding 200 feet of head. This would increase the pumping capacity by an additional 905 PE for a total of an additional 4,335 PE (1,582 residential units) in the Northeastern Basin. If development were only allowed in the Northeastern Basin, then Lift Station 7 and its force main could be modified to maximize the available capacity in the Route 176 West Interceptor. However, this alternative would prohibit development of the Northwestern, Central, and Waterford Basins.

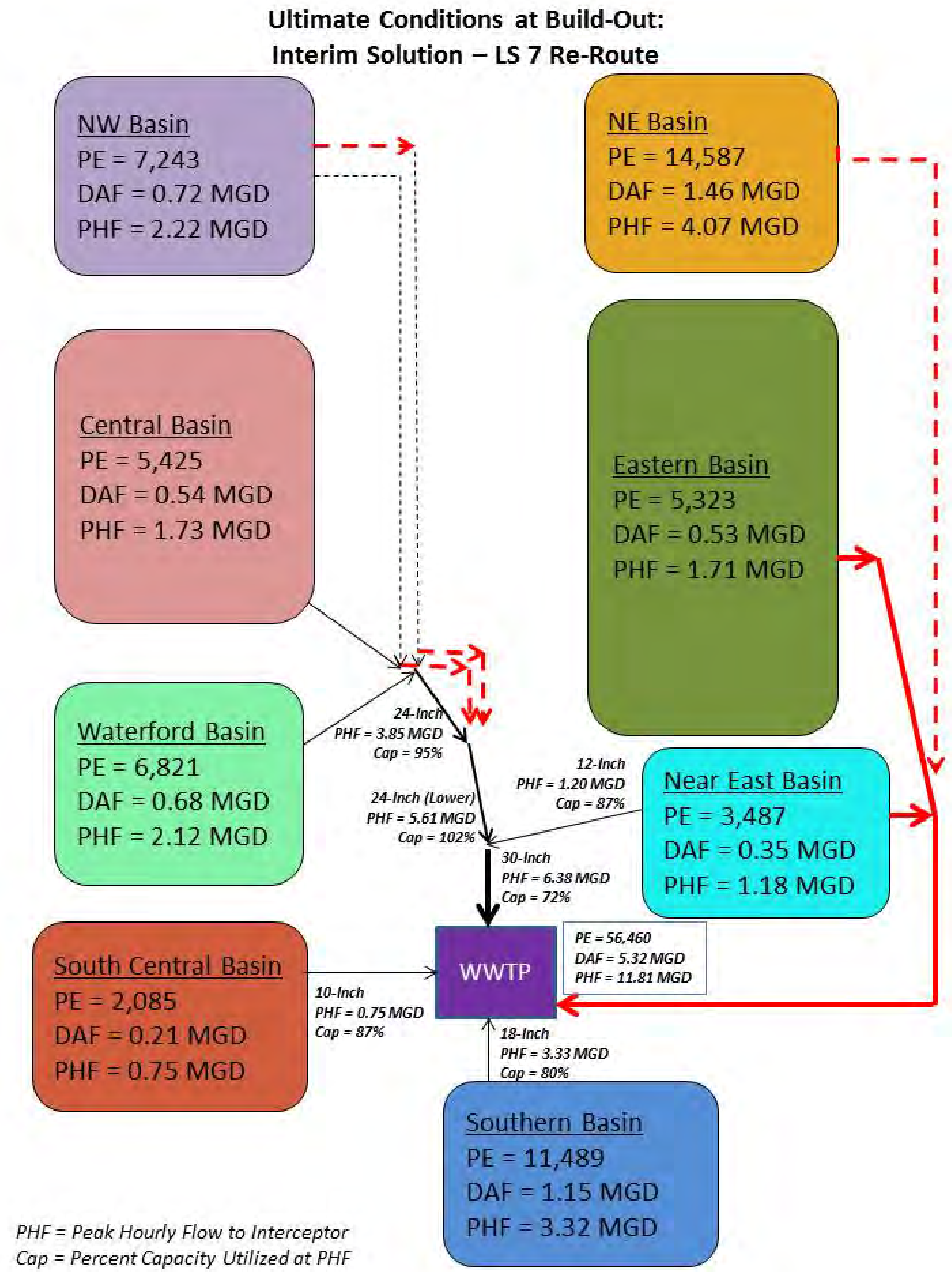
Alternatively, the Northeastern Basin could be allowed to develop up to the current capacity of Lift Station 7, which equates to an additional 1,480 PE (540 residential units). Once Lift Station 7 reaches capacity, improvements to the lift station and force main could be completed to divert this flow through the Eastern Basin. If the Northeastern Basin is diverted, the 12-inch force main on Lily Lake Road would be re-used to convey flow from the Northwestern Basin.

In summary, the existing Route 176 Interceptor sewer has capacity to support 4,425 PE. This capacity can be allocated on a first-come first-serve basis. The Central and Waterford Basins are free to develop as they are directly tributary to the Route 176 West Interceptor. The Northeastern Basin should only be developed up to the capacity of the existing Lift Station 7 or the Route 176 West Interceptor, whichever reaches capacity first. Development within the Northwestern Basin is severely limited until the Northeastern Basin is diverted and the 12-inch force main along Lily Lake Road becomes available. Therefore, ultimately the collection system on the eastern half of the Northern Moraine FPA needs to be constructed to allow for build-out of all the basins.

This issue was recognized in the 2004 Facility Plan Update. Over the last decade, the District has invested into planning, design and easement acquisition for construction of an eastern interceptor sewer system commonly known as the Darrell Road Collection System. On the following page, the ultimate simplified basin flow diagram includes the Darrell Road Collection System. Two alternatives exist, the original 2004 mostly gravity concept shown on Page 10, as well as an alternative Interim Solution Collection System shown on Page 11. The Interim Solution Colletion System includes extension of a new force main from the Northeastern Basin to the intersection of Route 176 and Darrell Road.

**Ultimate Conditions at Build-Out:
Darrell Road Collection System**




Ultimate Conditions at Build-Out:
Interim Solution – LS 7 Re-Route
NW Basin
PE = 7,243
DAF = 0.72 MGD
PHF = 2.22 MGD
NE Basin
PE = 14,587
DAF = 1.46 MGD
PHF = 4.07 MGD
Central Basin
PE = 5,425
DAF = 0.54 MGD
PHF = 1.73 MGD
Eastern Basin
PE = 5,323
DAF = 0.53 MGD
PHF = 1.71 MGD
Waterford Basin
PE = 6,821
DAF = 0.68 MGD
PHF = 2.12 MGD
24-Inch
PHF = 3.85 MGD
Cap = 95%
12-Inch
PHF = 1.20 MGD
Cap = 87%
24-Inch (Lower)
PHF = 5.61 MGD
Cap = 102%
Near East Basin
PE = 3,487
DAF = 0.35 MGD
PHF = 1.18 MGD
30-Inch
PHF = 6.38 MGD
Cap = 72%
South Central Basin
PE = 2,085
DAF = 0.21 MGD
PHF = 0.75 MGD
WWTP
PE = 56,460
DAF = 5.32 MGD
PHF = 11.81 MGD
10-Inch
PHF = 0.75 MGD
Cap = 87%
18-Inch
PHF = 3.33 MGD
Cap = 80%
Southern Basin
PE = 11,489
DAF = 1.15 MGD
PHF = 3.32 MGD
PHF = Peak Hourly Flow to Interceptor
Cap = Percent Capacity Utilized at PHF

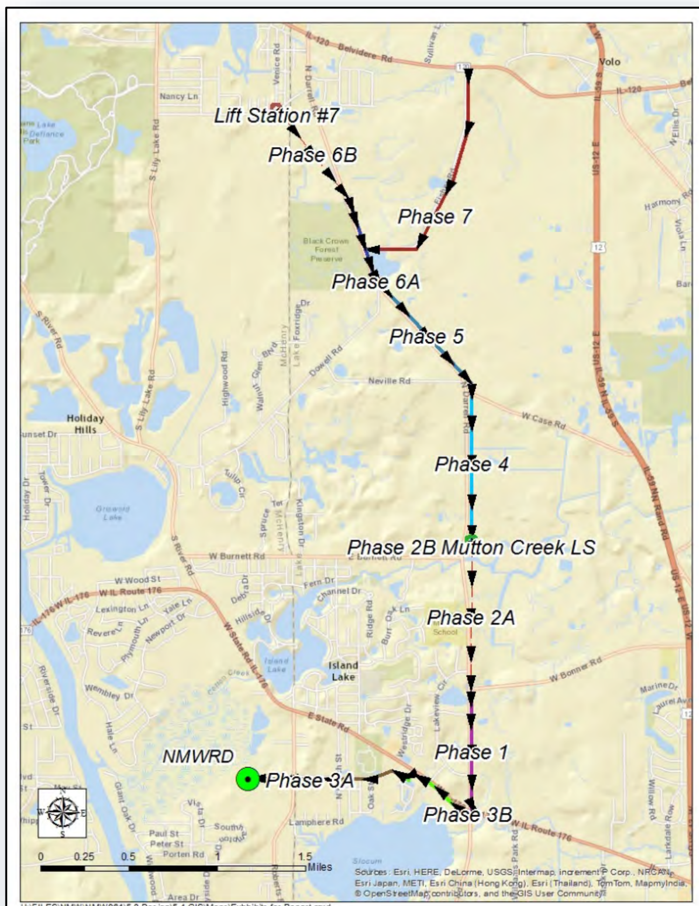
Darrell Road Collection System

The Darrel Road Collection System would be implemented in a phased manner over the course of several years as dictated by development. For facility planning purposes, a potential phased implementation plan is shown on Exhibit 4.

Construction of the proposed Darrell Road Collection System (see Exhibit 4) would provide relief for the 24-inch Route 176 West Interceptor, because flows from the Northeastern Basin would be removed from this sewer. In the flatter uppermost reaches of this sewer, deficiencies are still projected at build-out, but their severity would be greatly diminished and their occurrence would be shifted far into the future. At that time, the easiest method to provide relief would be to extend the Lakemoor force main(s) east along Route 176 to connect to the steeper downstream reaches.

The Darrell Road Collection System would also provide relief to the 12-inch Route 176 East Interceptor, because peak flows that currently are routed through this pipe would be conveyed directly to the NMWRD treatment facility, bypassing the Route 176 East Interceptor and the 30-inch Main Interceptor.

Exhibit 4: Darrell Road Collection System



Construction of the Darrell Road Collection System would also relieve projected overload conditions in the existing 8-inch and 12-inch parallel force mains. Extension of the Darrell Road Interceptor north along Darrell Road to Lakemoor will allow the force main from Lakemoor Lift Station 7 to be redirected to the east, thereby freeing capacity in the existing 12-inch force main along Lily Lake and River Roads to convey the additional flow that is projected to be generated by growth in the Northwestern Drainage Basin.

The Darrell Road Collection System would not only provide relief to the existing interceptor sewers and force mains, but also would provide a means to extend wastewater service to the currently unserved Eastern Drainage Basin.

Probable capital costs for the Darrell Road Collection System are summarized in Table 4.

Table 4: Probable Capital Costs - Darrell Road Collection System

Phase	Description	Probable Cost
1	Darrell Road Interceptor (South)	$ 2,255,000
2A	Mutton Creek Force Main	$ 2,149,000
2B	Mutton Creek Lift Station	$ 3,112,000
3A	Treatment Plant Interceptor	$ 5,056,000
3B	Water's Edge Interceptor Replacement	$ 1,902,000
4	Darrell Road Interceptor (Central)	$ 3,279,000
5	Darrell Road Interceptor (North)	$ 2,065,000
6A	Darrell Road Interceptor (Far North)	$ 1,687,000
6B	Lakemoor Lift Station 7 Force Main	$ 1,615,000
7	Fisher Road Interceptor	$ 3,321,000
TOTAL PROBABLE CAPITAL COSTS		**$ 26,441,000**

Interim Solution Collection System

Funding of the Darrell Road Collection System has always been contingent upon development within the Eastern Basin. An interim solution (see Exhibit 5) is to upgrade Lift Station 7 and construct a new force main from the Northeastern Basin to the intersection of Darrell Road and Route 176, as well as construction of Phase 3A, the completion of the 42-inch interceptor from the treatment facility to the Water's Edge Lift Station. Therefore, the District would be able to construct all other phases if and when the Eastern Basin is developed.

Probable capital costs for the Interim Solution Collection System are summarized in Table 5. The cost of the major upgrades to Lift Station 7 would be deferred to a later date as all that is needed to meet the 20-year projections for the Northeastern Basin is installation of a third pump in the existing wet well, and only if high rate growth warrants.

Exhibit 5: Interim Solution Collection System

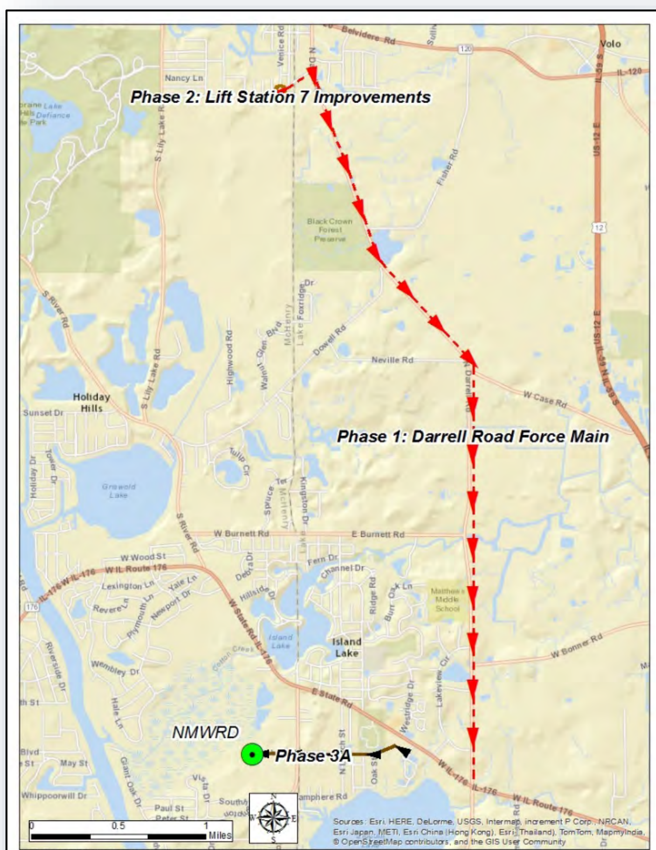


Table 5: Probable Capital Costs – Interim Solution Collection System

Phase	Description	Probable Cost
1	Darrell Road Force Main	$ 8,464,000
2	Lift Station 7 Upgrades - Future	$ 3,112,000
3A	Treatment Plant Interceptor	$ 5,056,000
TOTAL PROBABLE CAPITAL COSTS		**$ 16,632,000**

LIFT STATIONS

Pump run times at each of the twenty-two NMWRD lift stations were reviewed to assess the ability of the existing pumping equipment to meet current flow conditions. The runtime data indicates that all of the existing pump installations are sufficient for the current flow conditions.

Only at Lakemoor Lift Stations 1 and 6 do average runtimes exceed 5 hours per day. The capacities at these lift stations are becoming marginalized under average conditions; this will be exacerbated under wet weather flows. Caution should be exercised and further evaluation performed prior to connecting additional services to these lift stations. Somewhat less severe conditions exist at the Hale 1 and Fern Lift Station, and at Lakemoor Lift Station 3 and 4, where average pump runtimes exceed 4 hours per day. However, increased flows are not expected at these lift stations. The pumps at Lift Station 7 also exceed 4 hours runtime per day. These stations should also be analyzed prior to increasing their connected service population.

Pump drawdown tests were conducted at each lift station. Pump drawdown tests provide accurate and reliable measurements of actual pumping rates, which are then compared to rated capacities to determine whether the pumps are operating as intended by design, or if they are under- or over-performing. Significant deviations could be a result of a worn pump impeller, varying motor speed, partially obstructed pump discharge or force main piping, or an improperly designed installation. Drawdown test results are discussed in Section 4.

WASTEWATER TREATMENT FACILITY

Influent wastewater flow and pollutant loading data at the NMWRD treatment facility was reviewed for the past 3 years. Current flows and loads are well within the treatment capability of the existing treatment facility.

Current and projected wastewater flows and pollutant loadings at the treatment facility were summarized in Table 3. The treatment facility also has sufficient capacity to support planned development throughout those areas currently incorporated into the District. It is only as additional areas in the Northern Moraine FPA become annexed to the District that the treatment capacity of the facility will be exceeded.

Current population and flow projections estimate that the NMWRD treatment facility will reach capacity around the year 2025. The logical expansion of capacity at this facility would be to increase the average treatment capacity from 2.0 MGD to 3.0 MGD as has always been planned. In fact, the existing treatment facility was designed in a manner which facilitates this type of expansion through the construction of a third ring to the existing oxidation ditch.

However, the next major improvement at the NMWRD treatment facility is necessitated by the changing regulatory environment, not by growth. The District's new NPDES permit requires that phosphorus removal capabilities be incorporated, that the construction of these upgrades be completed by May 1, 2018, and that full compliance with an effluent limit of 1.0 mg/L total phosphorus be achieved by May 1, 2019. Alternatives to achieve compliance with the new phosphorus limit are evaluated in Section 6, including chemical phosphorus removal and biological phosphorus removal.

The new Northern Moraine NPDES permit was issued on October 23, 2014, has an effective date of November 1, 2014 and will expire on October 31, 2019. The permit includes special conditions related to the Fox River Study Group (FRSG) implementation plan. The special conditions require NMWRD to complete a phosphorus removal feasibility report within 12 months of the effective date of the permit. The study must evaluate options and costs associated with reducing phosphorus concentrations in the treatment facility effluent to 1.0 mg/L as well as 0.5 mg/L. The compliance schedule contained in the District's NPDES permit is summarized in Table 6.

Table 6: Phosphorus Removal Compliance Schedule

Item	Required date of completion	Completion Date
Interim Report on Phosphorus Removal Feasibility Report	6 months from the effective date of Permit	May 1, 2015
Phosphorus Removal Feasibility Report submitted	12 months from the effective date of Permit	November 1, 2015
Progress Report on FRSG Phosphorus Input Reductions and Implementation Plan	18 months from the effective date of Permit	May 1, 2016
Progress Report on Recommendations of FRSG Implementation Plane	24 months from the effective date of Permit	November 1, 2016
Plans and specifications submitted	30 months from the effective date of Permit	May 1, 2017
Progress Report on Construction	36 months from the effective date of Permit	November 1, 2017
Complete Construction	42 months from the effective date of Permit	May 1, 2018
Progress Report on Optimizing Treatment System	48 months from the effective date of Permit	November 1, 2018
Achieve Annual Concentration and Loading Effluent Limitations for Total Phosphorus	54 months from the effective date of Permit	May 1, 2019

The various improvements at the NMWRD treatment facility were segregated into three categories, including:

- Existing Treatment Facility Rehabilitation
- Regulatory Compliance (Phosphorus Removal)
- Treatment Facility Expansion

Existing Treatment Facility Rehabilitation

It is not anticipated that the existing treatment facility will reach capacity for at least another 10 years. However, there is equipment that will be reaching the end of its useful service life before the treatment facility needs to be expanded. These improvements are discussed in Section 5 and include:

- Replace the raw sewage pumps (although two of the pumps have low hours of use)
- Replace the influent magnetic flow meter and upsize or parallel the influent force main
- Replace the older 40-inch fine screen

This equipment should be replaced to ensure the reliability of the existing treatment facility to provide uninterrupted and effective treatment. Probable costs associated with these improvements are summarized in Table 7.

Table 7: Probable Capital Costs – Treatment Facility Rehabilitation

Description	Probable Capital Cost
GENERAL CONDITIONS	$ 93,750
RAW SEWAGE PUMP REPLACEMENT	250,000
INFLUENT FORCE MAIN AND METER REPLACEMENT	100,000
FINE SCREEN REPLACEMENT	250,000
SUBTOTAL CONSTRUCTION	$ 693,750
CONTINGENCY @ 25%	173,440
CONSTRUCTION TOTAL	$ 867,190
ENGINEERING @ 15%	130,100
PROBABLE CAPITAL COST – TREATMENT FACILITY REHABILITATION	$ 997,290

Regulatory Compliance (Phosphorus Removal)

Two approaches were considered for compliance with the new 1.0 mg/L phosphorus limit, chemical phosphorus removal and biological phosphorus removal. In either case, the ability to chemically remove phosphorus would be provided for polishing as needed to remove that phosphorus not removed biologically to the compliance level.

The alternatives were evaluated on a life-cycle present value cost basis, as each has differing initial capital expenditures and annual operating costs associated with them.

Annual chemical costs associated with chemical phosphorus removal are summarized in Table 8, and are based on the use of ferric chloride ($FeCl_3$).

Table 8: Chemical Costs – Phosphorus Removal

Phase	DAF (MGD)	Phosphorus (lbs/day)	Phosphorus (lbs/year)	$FeCl_3$ (gallons/year)	Estimated Annual Cost
I	2.0	0	-	-	-
2017 P Removal	2.0	100	36,500	54,750	$ 54,750
II	3.0	150	54,750	82,125	$ 82,125
III	4.5	225	82,125	123,188	$ 123,188
IV	6.0	300	109,500	164,250	$ 164,250

Either alternative (chemical or biological P removal) will result in additional sludge production. The aeration costs for digesting this sludge in the aerobic digesters must also be considered in the life cycle cost effective analysis. Table 9 summarizes the estimated sludge production from the secondary process under three conditions – current design, chemical phosphorus removal and biological phosphorus removal.

Table 9: WAS Production – Chemical P Removal versus Biological P Removal

Phase	DAF (MGD)	WAS Production Without P Removal (lbs/day)	WAS Production With Chemical P Removal (lbs/day)	WAS Production Using Biological P Removal (lbs/day)
I	2.0	1,600	-	-
2017 P Removal	2.0	1,600	1,978	1,970
II	3.0	2,400	2,966	2,954
III	4.5	3,601	4,450	4,432
IV	6.0	4,801	5,933	5,909

It can be seen that the amount of sludge produced is essentially equal under either phosphorus removal alternative. However, in either case sludge production will increase by approximately 23 to 24 percent above that produced by the existing processes without phosphorus removal.

Chemical phosphorus removal would require construction of chemical feed facilities which would include a building to house the chemical storage tank and feed pumps. The building must include sufficient containment to prevent spills from escaping the building. Probable capital costs associated with construction of the Chemical Feed Building are presented in detail in Section 6, and are summarized in Table 10.

Biological phosphorus removal would require construction of an anaerobic selector tank upstream of the existing oxidation ditch, sized at 150,000 gallons to provide 1.5 hours detention time at the current design flow of 2.0 MGD. Probable capital costs associated with the selector tank are presented in detail in Section 6, and are also summarized in Table 10.

Table 10: Probable Capital Costs - Phosphorus Removal

Description	Chemical Feed Building	Anaerobic Selector Tank
CONSTRUCTION SUBTOTAL	$ 358,015	$ 589,800
GENERAL CONDITIONS	44,750	73,725
CONTINGENCY @ 15%	60,415	99,530
CONSTRUCTION TOTAL	463,180	763,055
DESIGN ENGINEERING @ 7.5%	34,740	57,230
CONSTRUCTION ENGINEERING @ 7.5%	34,740	57,230
TOTAL CAPITAL COSTS	**$ 532,660**	**$ 877,515**

The life-cycle cost effective analysis comparing chemical phosphorus removal to biological phosphorus removal at the NMWRD treatment facility is summarized in Table 11.

Table 11: Cost Effective Analysis – Phosphorus Removal

Description	Chem-P	Bio-P
Capital Cost	$ 532,600	$ 1,410,175
Annual Costs:		
Chemicals	$ 54,750	$ 11,467
Blower Energy	40,887	50,323
Mixing Energy	-	18,291
Subtotal – Annual Costs	$ 95,637	$ 80,080
Discount Rate	4%	4%
Planning Period (years)	20	20
Present Value – Annual Costs	$ 1,299,740	$ 1,088,317
Total Present Value	$ 1,832,340	$ 2,498,492

The analysis shows that chemical phosphorus removal is the cost-effective solution at the NMWRD treatment facility at this time. When the facility is expanded to a treatment capacity of 3 MGD, the addition of the third ring to the oxidation ditch will provide the flexibility to achieve a degree of biological phosphorus removal through modification to the operation and control of oxygen in the multi-channel ditch. Even then, the chemical feed facility would be used for polishing as needed to comply with the NPDES effluent limits.

The NPDES permit requires that the District assess the method, time frame and costs specific to the NMWRD treatment facility for reducing its loading of phosphorus to not only 1.0 mg/L but also to 0.5 mg/L. A ferric to phosphorus molar ratio of 3:1 was assumed during the evaluation of chemically removing phosphorus to 1.0 mg/L. This is a conservative assumption used for planning purposes; some treatment facilities successfully remove phosphorus chemically at a molar ratio of 1.5:1.

The required molar ratio to remove phosphorus below 1 mg/L to 0.5 mg/L is greater. The typical molar ratio for these low levels increases by 2 to 6 times that for less stringent requirements; for planning purposes a molar ratio 4 times greater was assumed, or 12:1. Instead of assuming 1.5 gallons of ferric chloride per pound phosphorus removed, it was assumed that 6.0 gallons of ferric chloride per pound phosphorus removed would be required. Therefore, the cost per pound of phosphorus removed will increase as the concentration is lowered from 1 mg/L to 0.5 mg/L.

It is recommended that jar testing be conducted on the NMWRD mixed liquor to clarify actual chemical requirements for phosphorus removal at the NMWRD treatment facility.

Treatment Facility Expansion Phasing

The Phase I Expansion completed in 1998 set precedence for the future expansion of the facility. While that particular expansion phasing plan only contemplated one additional phase increasing the capacity from 2.0 to 3.0 MGD, it was concluded that future phases should be designed to parallel the processes as much as practical.

The 2004 Facility Plan Update included four future phases of expansion up to an ultimate treatment capacity of 10 MGD. Future population and flow projections have been revised for this 2014 Facility Plan Update, which now includes upgrades only up to an ultimate capacity of 6.0 MGD as shown in Table 12.

Table 12: Phased NMWRD Treatment Facility Expansion Plan - Design Flows

Construction Phase	2004 Facility Plan Design Average Flow (MGD)	2014 Facility Plan Design Average Flow (MGD)
Phase I (Existing)	2.0	2.0
2017 P Removal	n/a	2.0
Phase II	3.0	3.0
Phase III	4.5	4.5
Phase IV	6.0	6.0
Phase V	10.0	n/a

The treatment facility is currently operating at 53% of its hydraulic design, 64% of its BOD loading design, 47% of its TSS loading design, and 54% of its ammonia loading design. It is recommended that once the facility plan is approved that the District proceed with the 2017 Upgrades (phosphorus removal) design to ensure adequate treatment capabilities are available for continued compliance with the updated NPDES permit. Phases II, III and IV will need to be considered once the facility reaches 80% of its design capacity of the preceding phase (currently 1.6 MGD). It is currently projected that the Phase II Expansion will need to be completed by the year 2025 as shown on Figure 1 and discussed in Section 2. Probable capital costs associated with the Phase II Expansion of the NMWRD treatment facility are summarized in Table 13.

Table 13: Probable Capital Costs – Phase II Expansion

Description	2004 Probable Cost	Probable Cost (2015 Dollars)
CONSTRUCTION SUBTOTAL	$ 9,076,500	$ 13,252,300
GENERAL CONDITIONS	1,134,700	1,600,000
CONTINGENCY (25%)	2,269,200	3,313,100
PROBABLE CONSTRUCTION COST	$ 12,480,400	$ 18,165,400
DESIGN ENGINEERING @ 7.5%	936,100	1,362,500
CONSTRUCTION ENGINEERING @ 7.5%	936,100	1,362,500
TOTAL CAPITAL COST	**$ 14,352,600**	**$ 20,890,400**

IMPLEMENTATION PLAN

The Northern Moraine NPDES permit includes a compliance schedule which includes milestones related to phosphorus removal planning, design, construction and start-up. The schedule requires that designs for phosphorus removal be submitted to the IEPA by April 1, 2017 and that construction April 1, 2018. Full compliance with the 1.0 mg/L phosphorus limit must be achieved by April 1, 2019. Interim progress reports must also be submitted, including an interim report of the Phosphorus Removal Feasibility Study which is due on April 1, 2015. The final Phosphorus Removal Feasibility Study must be submitted prior to November 1, 2015.

The various improvements recommended for rehabilitation of the existing NMWRD treatment facility could be implemented as stand-alone projects or as part of the 2017 phosphorus removal construction project. This would place the replacement pumps online in 2018, at which time they will be 20 years old.

The Phase II Expansion is not anticipated to be required until at least the year 2025 as projected under the growth assumptions discussed in Section 2.

The construction of the Interim Solution Collection System improvements would not be required until either the current capacity of Lift Station 7 is reached, or until the surplus capacity currently available in the 24-inch Route 176 West Interceptor is exhausted (4,425 PE currently available). The growth projections presented for the Central, Waterford, Northwest and Northeastern Basins indicate the interceptor's capacity will not be reached within the 20-year facility planning period except under the most optimistic of growth projections.

In consideration of the remaining service life of the existing facilities, regulatory requirements, and projected growth through the Northern Moraine FPA, the phased Implementation Plan for this 2014 Facility Plan Update is summarized in Table 14.

Table 14: Phased Implementation Plan

Description	Probable Capital Costs ($ millions)						
	2015	2016	2017	2018	2019	2020 to 2029	2030 to 2039
TREATMENT FACILITY							
Rehabilitation / Replacement		$ 0.07	$ 0.93				
Phosphorus Removal		$ 0.04	$ 0.50				
Phase II Expansion						$ 20.90	
COLLECTION SYSTEM							
Interim Solution Eastside System						$ 13.52	$ 3.11
Annual rehabilitation / replacement	$ 0.62	$ 0.62	$ 0.62	$ 0.62	$ 0.62	$ 6.20	$ 6.20
LIFT STATIONS							
Annual rehabilitation / replacement	$ 0.33	$ 0.33	$ 0.33	$ 0.33	$ 0.33	$ 3.30	$ 3.30
TOTAL PROBABLE CAPITAL COSTS	$ 0.95	$ 1.06	$ 2.38	$ 0.95	$ 0.95	$ 43.92	$ 12.61

NOTE: PROBABLE COSTS ARE PRESENTED ABOVE IN 2014 DOLLARS AND DO NOT ACCOUNT FOR FUTURE INFLATION. OVER THE PAST 10 YEARS, ENR COST INDEXES HAVE INFLATED AT AN EQUIVALENT ANNUAL COMPOUND RATE OF 3.16%.

Replacement expenditures for sanitary sewers were estimated at approximately $617,300 annually. Attention should first be given older areas in the collection system and to those areas where I/I has been detected. Replacement costs for lift stations and force mains were estimated to be approximately $324,800 per year.

The scheduling of these improvements are initial estimates employed for planning purposes. The phosphorus removal improvements at the NMWRD treatment facility must be completed by April 2018 as required in the District's NPDES permit. Scheduling for Phase II expansion and Interim Solution collection system will become known with greater certainty over time as actual development throughout the Northern Moraine FPA occurs.

USER RATES AND PROJECT FUNDING

The District recently completed an internal User Rate Study that concluded that the current rate schedule was insufficient to cover current operation, maintenance, and replacement costs. A 5-year rate plan was adopted by the District Board in 2014 to address those issues. The replacements of the raw sewage pumps, along with the fine screen, could be funded out of the District's replacement fund. The phosphorus removal improvements might be funded out of capital project cash reserves. A more detailed user rate and connection fee study will be required to assess how to cover the debt service that would be incurred to implement the Phase II Expansion of the treatment facility (planning to begin when average flows reach 1.6 MGD), as that project will involve borrowing, likely in the form of an IEPA low-interest loan.

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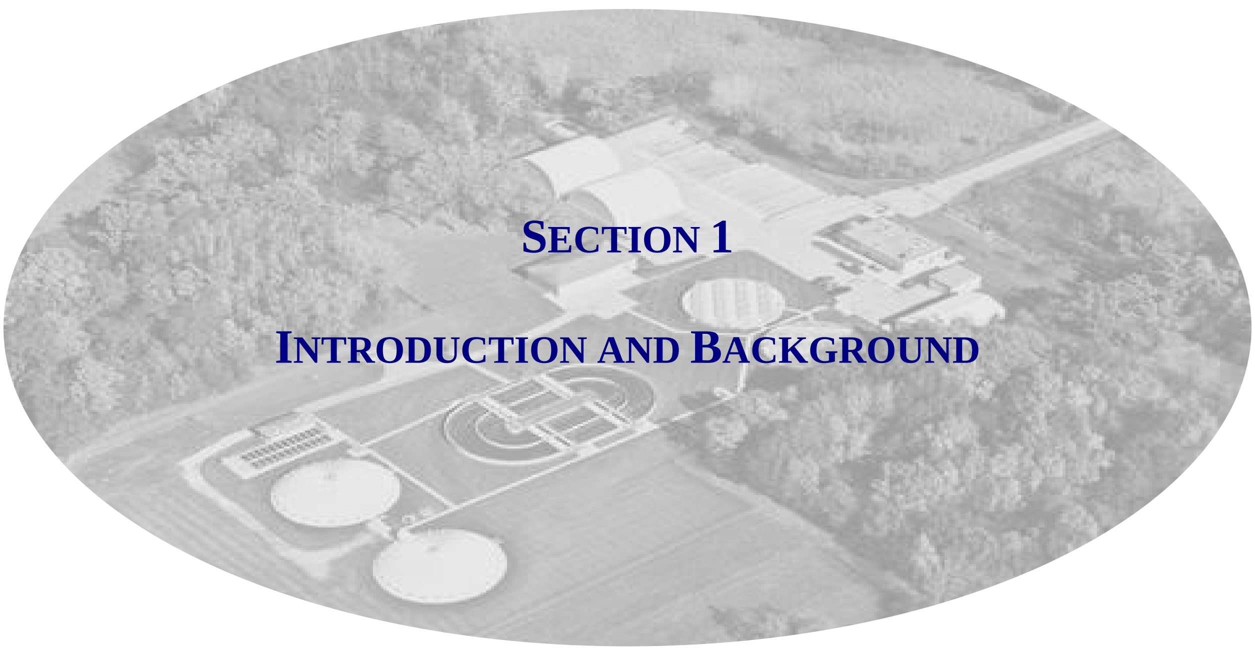

SECTION 1

INTRODUCTION AND BACKGROUND

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1. INTRODUCTION AND BACKGROUND

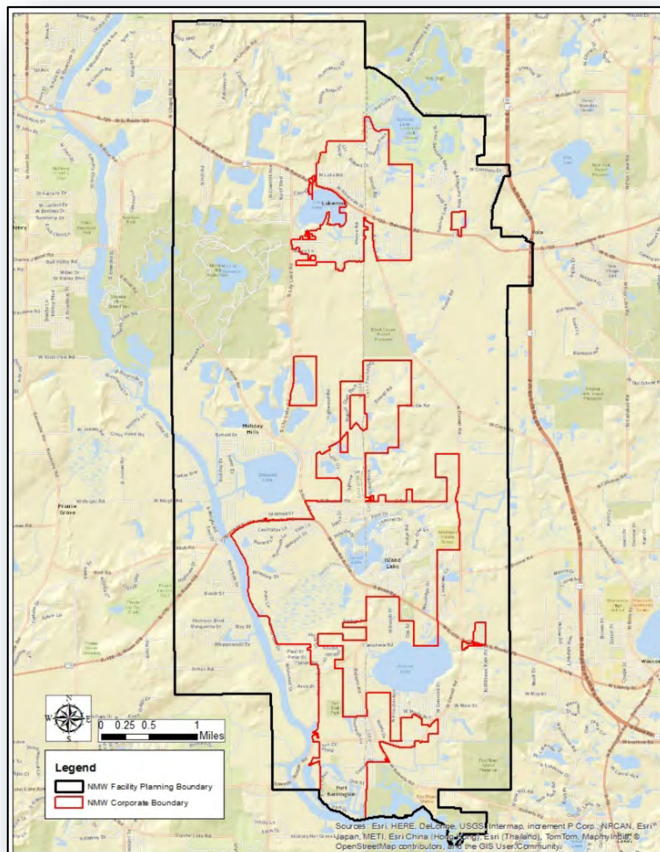
1.1 GENERAL BACKGROUND

The Northern Moraine Wastewater Reclamation District (NMWRD) is located in Lake and McHenry Counties, approximately 40 miles northwest of Chicago. It is the regional wastewater Designated Management Agency (DMA) for the Facility Planning Area shown on Exhibit 1-1.

The District's Facility Planning Area (FPA) includes 16,700 acres, of which 3,700 acres are annexed to the District, including 2,400 acres already developed.

The District currently serves the Villages of Island Lake, Lakemoor, and Port Barrington. Portions of the Villages of Lake Barrington and Holiday Hills also lie within the NMWRD Facility Planning Area but are not currently served or connected to the collection system.

Exhibit 1-1: Facility Planning Area



The District owns and operates a wastewater treatment facility, located in unincorporated Island Lake, which is currently rated to treat an average flow of 2.0 million gallons per day (MGD). The treated effluent is discharged to the Fox River under NPDES Permit No. IL0031933.

Most of the communities within the Facility Planning Area have recently updated their Comprehensive Plans to designate current and planned land-use and provide direction for continued development.

The wastewater treatment and collection facilities are vital assets of community infrastructure. Recognizing the benefit of and need for coordination, the District has elected to update its Facility Plan to address current and long-range infrastructure needs consistent with the current land planning documents. The goal of this report is to evaluate the existing facilities and develop a plan for the extension of sanitary services to meet current and future needs while promoting environmental interests within the Facility Planning Area.

This updated Facility Plan was also prepared using the more complete and accurate demographic and logistical data contained in the Geographical Information System (GIS) system that was unavailable during previous facility planning efforts.

1.2 CAPACITY NEEDS

Wastewater treatment capacity is usually rated in flow (MGD), pollutant load (mg/L and lbs/day), and/or population equivalents (PE). In the absence of current flow and/or load data, and for future planning purposes, the IEPA allows for each population equivalent to contribute up to 100 gallons per capita per day (gcd); 0.17 lbs/day of biochemical oxygen demand (BOD_5); and 0.22 lbs/day of total suspended solids (TSS). The loading of other pollutants such as ammonia-nitrogen and phosphorus are typically projected based on their concentration. For example, an influent concentration of phosphorus of 6 mg/L might be assumed. Influent ammonia concentrations are typically in the range of 20 to 30 mg/L. The IEPA rates each facility on its capacity to adequately treat each of these parameters.

The NMWRD currently serves a residential population of approximately 12,826 PE. In addition, the District serves commercial customers equating to 877 PE, for a total population equivalent served of 13,703 PE. The derivation of these figures are discussed in Section 2.

Historically, the communities within the Northern Moraine FPA have grown at a moderate pace. In recent years growth has leveled consistent with decreased activity in the housing market. Approximately 2,400 acres of the 16,700 acres within the Northern Moraine WRD's FPA are developed. This equates to an average density of approximately 5.7 PE per acre in developed areas. The updated comprehensive land use plans discussed in Section 2 provide a guide as to how future growth should be planned.

1.3 COLLECTION SYSTEM

Northern Moraine WRD's collection system includes eight major wastewater drainage basins. The District owns and maintains approximately 57 miles of sanitary sewer and 15 miles of sanitary force main. Generally, the sewers are in good condition and the collection system does not experience extensive levels of infiltration and inflow (I/I) during wet weather conditions. Section 3 will address future expansion of the collection system to serve future development, most notably in the eastern, northeastern, and northwestern extents of the Northern Moraine FPA, focusing on the installation of the Darrel Road Collection System.

1.4 LIFT STATIONS

There are 22 lift stations within the District's collection system that are of varying age and style. Throughout the years, the District has conducted several upgrades to the lift stations utilizing both in-house manpower and contracted services. These upgrades have included the addition of modern level monitoring devices, variable speed pumping capabilities, flow metering devices, electrical rehabilitation, and emergency autodialers.

Treatment Plant Lift Station

The District requires proposed development to pay for the construction of any new lift stations it may require. The District has thoroughly examined its construction review process and developed a formal checklist of essential requirements to be included in the design and construction of all new lift stations. Section 4 of this report provides analysis of the existing lift stations and discusses capacity requirements and locations for future lift stations.

1.5 EXISTING WASTEWATER TREATMENT FACILITY

The NMWRD wastewater treatment facility discharges to the Fox River through a 30-inch outfall sewer that extends from the facility through the Cotton Creek Marsh to the Fox River. Constructed in 1978, the initial treatment facility included a raw sewage pump station, control building, garage, two package treatment plants, and drying beds. The biological processes included contact stabilization in conjunction with aerobic digestion. Biosolids were dewatered utilizing the drying beds and applied to agricultural lands.

In 1991, new chlorination/dechlorination facilities were constructed to comply with revised NPDES permit disinfection requirements. In 1992, additional upgrades included raw sewage flow metering and replacement of aeration diffusers within the biological process and aerobic digesters. In 1993, the District replaced the original comminuter (grinder) with a small mechanical fine screen. In 1997, the blowers for the contact stabilization package plants were replaced.

Treatment Plant Building

1.5.1 1998 Facility Plan Amendment

The District completed a Facility Plan Amendment in April 1998. At that time, the District was serving roughly 10,000 PE. The recommendations of the plan were based on a projected 20-year service population of 30,000 PE. The recommended plan included a phased expansion of the treatment facility. The Phase I improvements expanded the facility's design capacity from 1.2 MGD to 2.0 MGD to serve up to 20,000 PE.

Phase I improvements was completed in 1999 and included installation of a second mechanical fine screen, replacement of the raw sewage pumps, construction of a two-ring oxidation ditch, two new final clarifiers, sludge dewatering building, and a chemical feed building. The existing package treatment plants were converted to provide aerobic digestion and sludge storage.

1.5.2 2004 Facility Plan Update

In 2004, the communities within the facility planning area were developing at unprecedented rates. In response to the growing need for wastewater service throughout the Northern Moraine

FPA, the District completed a Facility Plan Update which included a thorough review of each community's comprehensive plan and land use plan. Based on the needs demonstrated in these comprehensive plans, the District developed a phased expansion plan to increase the treatment facility's capacity from 2.0 MGD to 10.0 MGD.

Since the completion of the 2004 Facility Plan, the District has successfully acquired the Village of Lakemoor collection system. In addition, several improvements have been made to the wastewater treatment facility.

In 2013, the aerobic digesters were rehabilitated. The project included removal of the interior steel walls, replacement of the diffused aeration system, and minor modifications to waste activated sludge, decant, and digested sludge piping. The improvements also included aluminum covers to maintain temperature during winter operation. Under a separate project, a centrifuge was added to the dewatering facility, and covers were installed over two of the five sludge drying beds to provide for dewatered sludge storage.

In 2014, a new influent screen and a new aerobic digester blower were installed. Additional improvements included implementation of variable speed aerator control via variable frequency drives (VFD) to control dissolved oxygen (DO) levels in the biological treatment process.

1.5.3 Current Treatment Capacity

The existing facility has capacity to treat an average flow of a 2.0 MGD and consists of influent screening, raw sewage pumping, extended aeration/biological treatment, tertiary clarification, and chlorine disinfection. The waste activated sludge is aerobically digested, mechanically thickened, dewatered, and land applied as fertilizer.

The average influent wastewater flow from July 2013 to June 2014 was 1.05 MGD, which equals 53 percent of the facility's rated capacity. Similarly, the BOD_5 and TSS loads received during this period were only 64 and 47 percent of design capacity, respectively.

The IEPA places facilities that receive over 80 percent of their rated hydraulic capacity under Critical Review and facilities that are operating at or above full design capacity on Restricted Status. Status is determined using the 3-month low average flow. The NMWRD treatment facility currently operates well below these thresholds, the 3-month low average influent flow for the 12-month period from July 2013 through June 2014 was only 0.92 MGD (46 percent of design capacity).

Current and projected annual average wastewater flows and pollutant loadings are compared to the current design capabilities at the NMWRD treatment facility throughout this report. However, it is important to note that it is the 3-month low flow that would ultimately trigger action by the IEPA should the District choose to wait until enforcement action is taken to begin planning and design of future treatment facility expansion.

The existing NMWRD wastewater treatment facility is discussed in Section 5, which provides an analysis of the performance, condition, and adequacy of each of the existing unit processes along with their ability to meet current and future conditions.

SECTION 2

COMMUNITY NEEDS

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2. COMMUNITY NEEDS

2.1 INTRODUCTION

This section of the Facility Plan Update is intended to outline the NMWRD's facility planning area, existing and future populations, wastewater flows, pollutant loadings, and regulatory considerations in order to provide a complete evaluation of the District's wastewater collection, conveyance, and treatment facility needs.

2.2 GENERAL BACKGROUND

The Northern Moraine Wastewater Reclamation District is located in northeastern Illinois, within southwestern Lake County and southeastern McHenry County. The District serves a facility planning area consisting of 16,700 acres. Within the facility planning boundary, 3,700 acres are incorporated into the NMWRD. Approximately 2,400 acres within the District's corporate boundary are currently developed.

The District currently provides service to the communities of Lakemoor, Island Lake and Port Barrington. The populations of these communities are 8,080, 6,000, and 1,500, respectively. Exhibit 2-1 outlines the NMWRD corporate and facility planning area boundaries, along with the extents of the surrounding communities. As can be seen, sections of the Villages of Volo, Holiday Hills, and Lake Barrington also lie within the extents of the Northern Moraine FPA, but are not currently incorporated within the District and do not currently receive wastewater service.

For planning purposes, the NMWRD FPA was divided into eight distinct wastewater drainage basins, including the Central, Eastern, Near East, Northeastern, Northwestern, South Central, Southern, and Waterford Drainage Basins. The wastewater drainage basins are shown on Exhibit 2-2.

Exhibit 2-1: NMWRD FPA and Corporate Boundaries

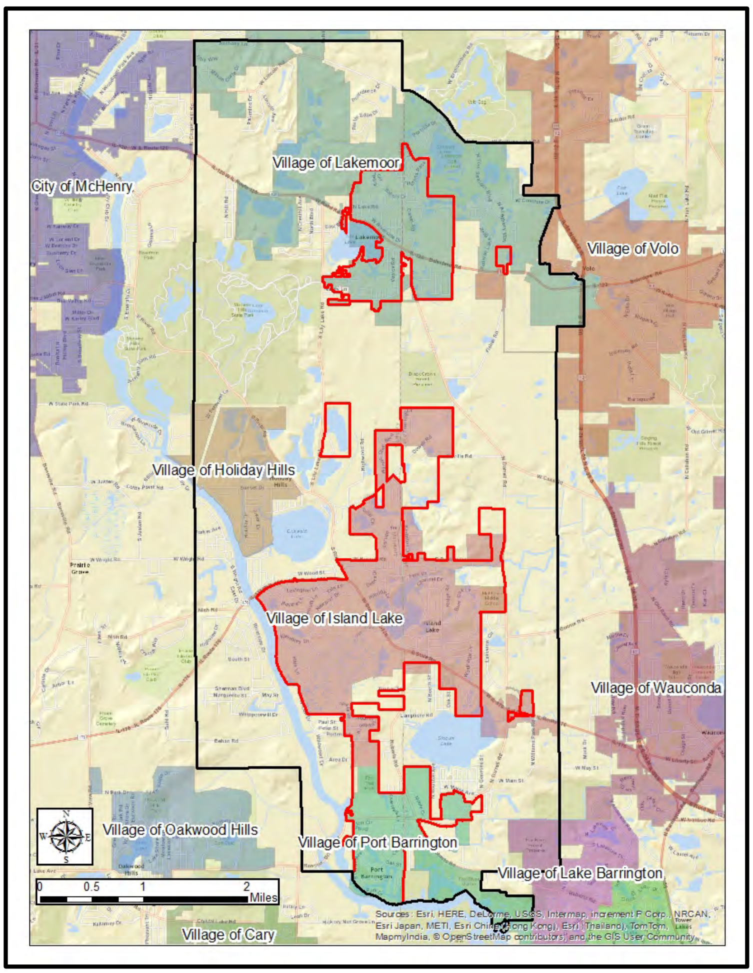
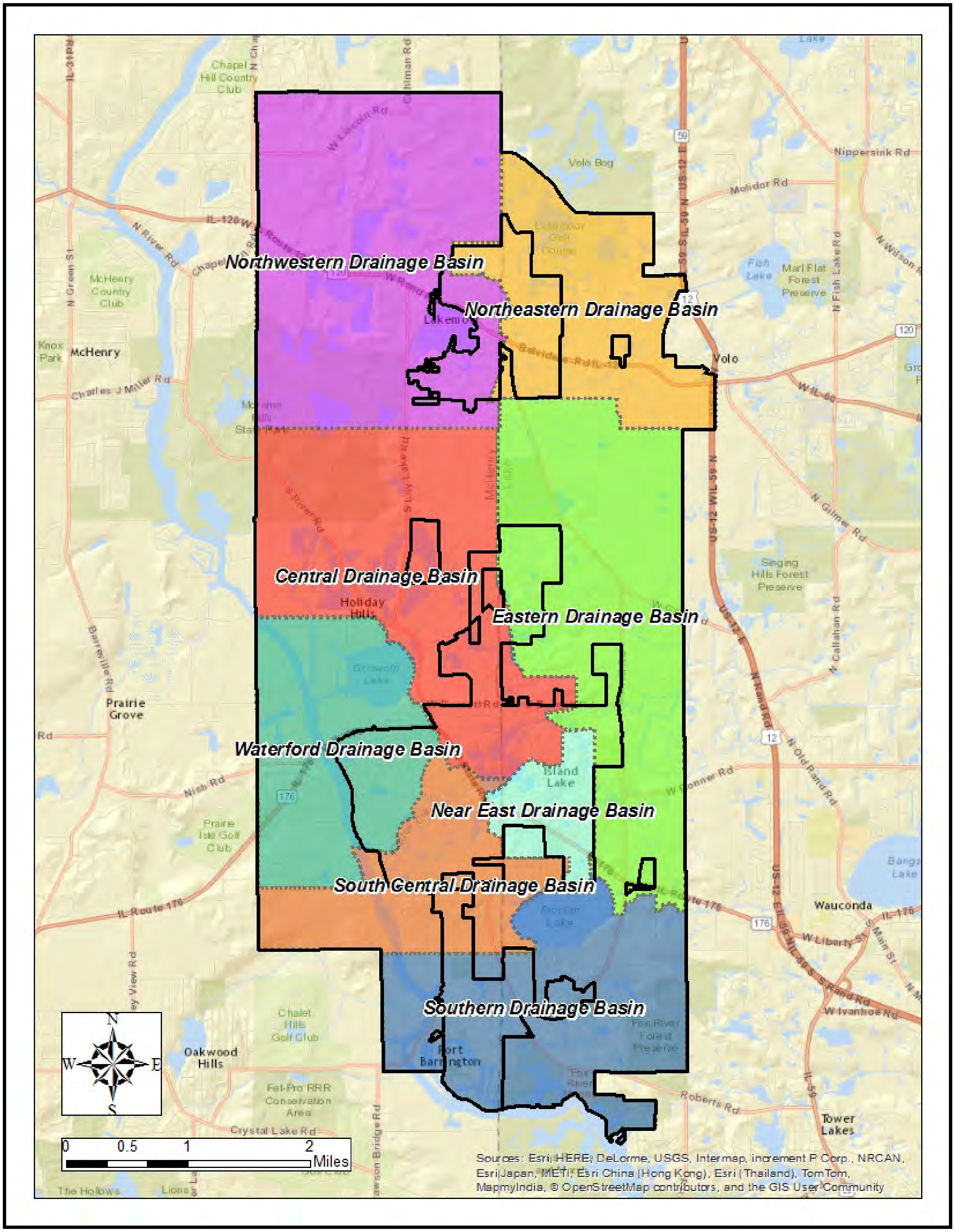


Exhibit 2-2: NMWRD Wastewater Drainage Basins



2.3 EXISTING CONDITIONS

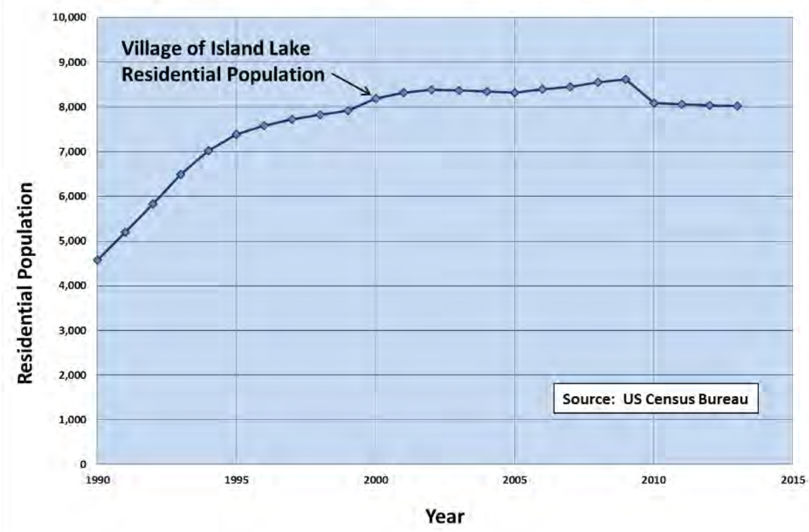
2.3.1 Residential Populations

Historical growth rates of the residential population within the corporate boundaries of the Villages of Island Lake, Lakemoor, and Port Barrington have varied over the past 25 years.

Island Lake Population Growth

The population of the Village of Island Lake from 1990 to 2013 is shown on Figure 2-1 and totaled 8,017 in 2013.

Figure 2-1: Island Lake Residential Population

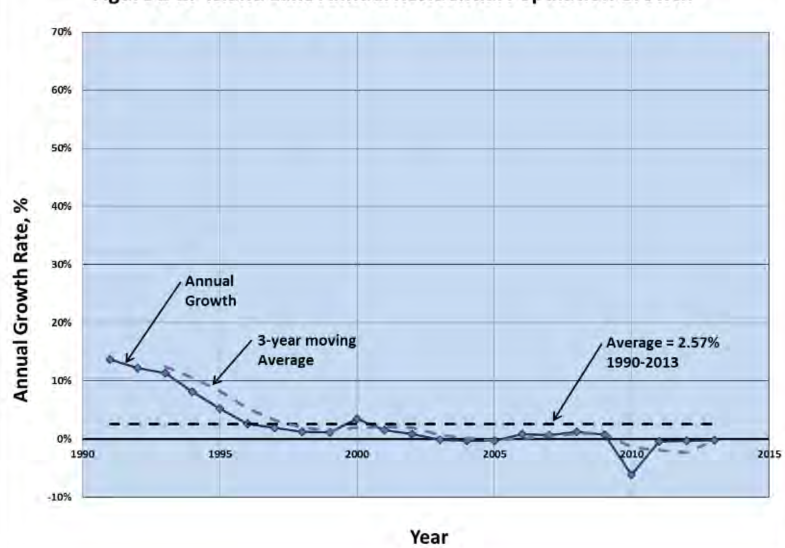


Island Lake experienced moderate and steady growth through the 1990's which has since slowed.

Annual growth in population during this time period is shown on Figure 2-2. Over the 23 year period the average growth rate was 2.57 percent.

Over the past 10 years (2003-2013) Island Lake has experienced negative growth, averaging a negative 0.42 percent which was highly influenced by the decrease in population that occurred in 2010. In general, growth has not occurred over the past decade.

Figure 2-2: Island Lake Annual Residential Population Growth



Lakemoor Population Growth

The population of the Village of Lakemoor from 1990 to 2013 is shown on Figure 2-3 and totaled 6,007 in 2013.

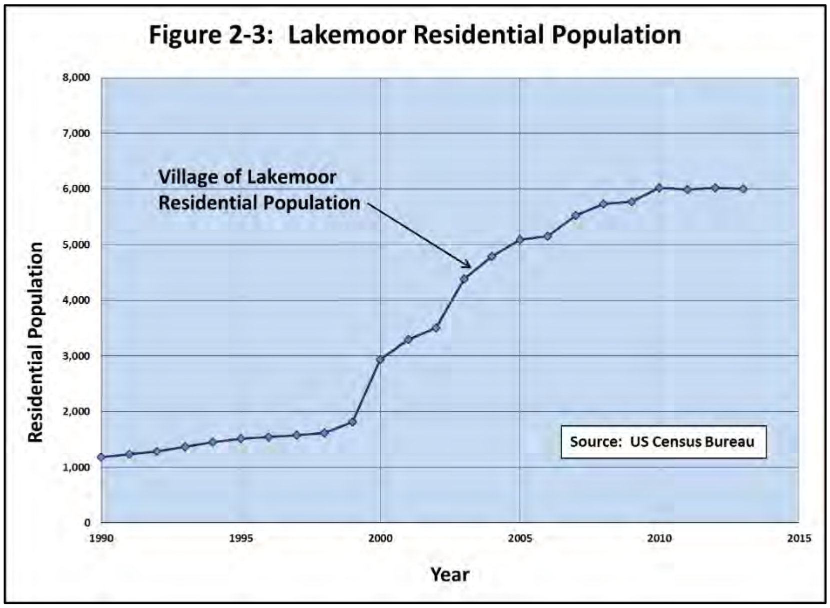


Figure 2-3: Lakemoor Residential Population

Residential construction in Lakemoor boomed in the late 1990's and throughout most of the 2000's up until the slowing of the housing market in 2008. Growth of the residential population has been stagnant since 2010.

Annual growth in population during this time period is shown on Figure 2-4. Over the 23 year period the average growth rate was 7.93 percent. If the years 2000 and 2003 are neglected, the average growth rate in Lakemoor was 4.32 percent.

Over the past 10 years (2003-2013) the Village of Lakemoor has experienced a reduced average growth rate of 3.70 percent.

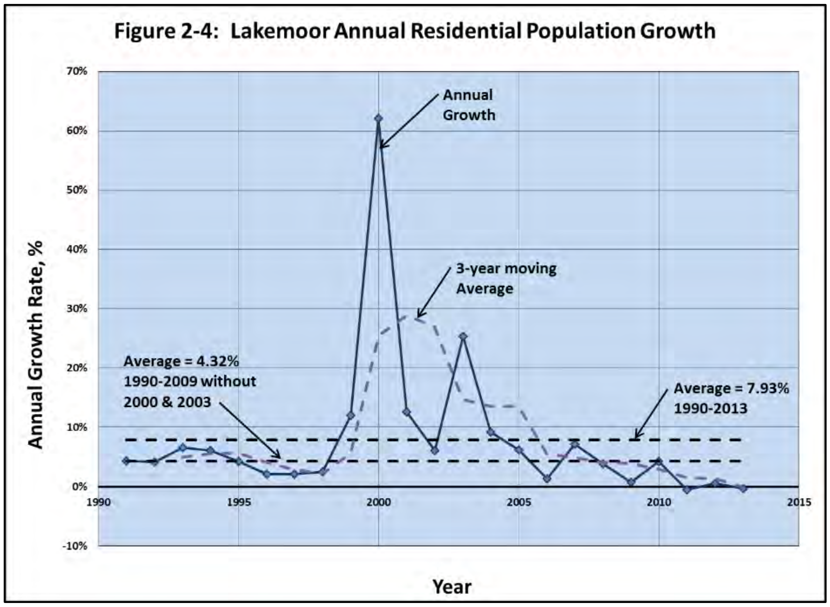
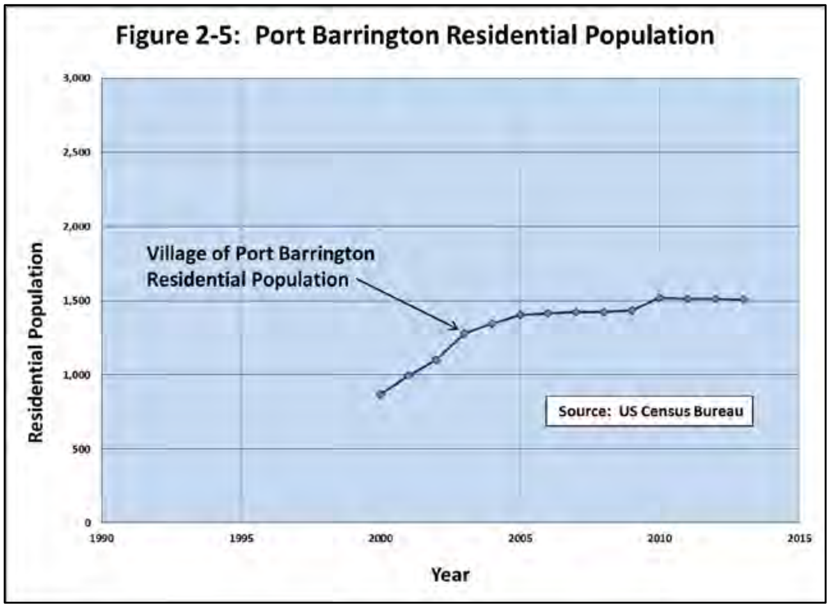


Figure 2-4: Lakemoor Annual Residential Population Growth

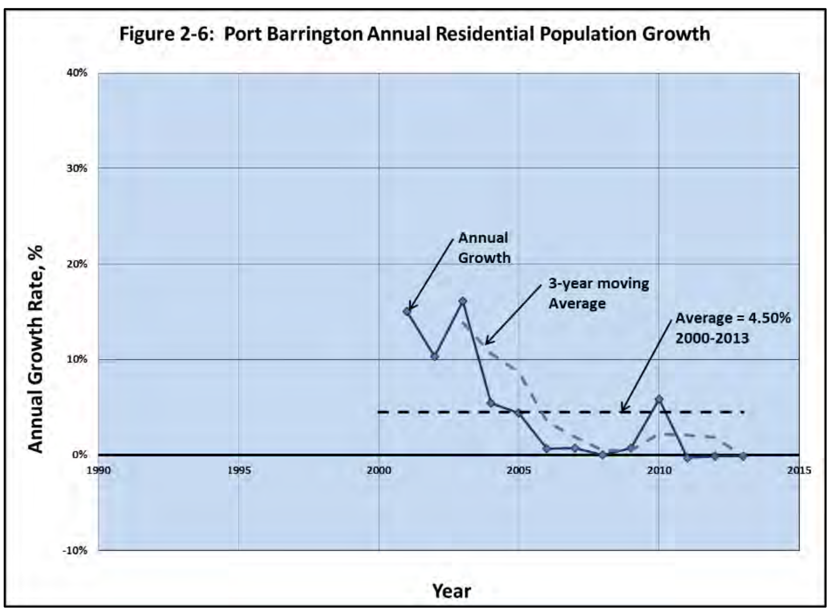
Port Barrington Population Growth

The population of the Village of Port Barrington from 2000 to 2013 is shown on Figure 2-5 and totaled 1,509 in 2013.



Residential construction in Port Barrington was strong early in the 2000's and slowed later in the decade. Growth of the residential population has been fairly stagnant since 2005.

Annual growth in the residential population of Port Barrington during this time period is shown on Figure 2-6. Since the year 2000 the average growth rate was 4.50 percent.



Over the past 10 years (2003-2013) the Village of Port Barrington has experienced a reduced average growth rate of 1.91 percent, which was largely influenced by growth which occurred from 2003 through 2005 and also 2010. With the exception of 2010, growth in Port Barrington has been flat since 2005.

2.3.2 Population Equivalents (PE) & Water Demands

Most communities contain both residential and non-residential land uses. Analysis of current and future water usage and wastewater generation is often done on the basis of "population equivalents", or PE, which provides a common basis for residential and non-residential use to be analyzed. One PE is equivalent to the water consumed or wastewater generated by one resident.

Mapping data was provided by the District that included parcels billed for water usage. This data was separated into categories of residential, commercial metered, commercial low-user, and commercial regular.

The residential PE was calculated by taking the residential parcels multiplied by a value of PE per unit. This PE per unit value was calculated by dividing the population of each town by total number of residential units in that town. The value calculated for Lakemoor was applied throughout the Northeastern and Northwestern Basins. Island Lake's value was applied to the Central, Eastern, Near East, and Waterford Basins, while Port Barrington's was applied to the South Central and Southern Basins.

It is recognized that all residential units are not currently served by sewer. The population associated with these units was excluded from the calculation by comparing total houses to the District's billing records.

Based on this analysis, the existing residential PE was calculated to be 12,818 PE as summarized in Table 2-1.

Table 2-1: Residential PE Breakdown by Basin

Drainage Basin	Residential - Inside NMWRD Corporate Boundary		
	Residential Units	PE/Unit	Residential PE
Central Drainage Basin	1,131	2.74	3,099
Eastern Drainage Basin	14	2.74	38
Near East Drainage Basin	734	2.74	2,011
Northeastern Drainage Basin	644	2.74	1,765
Northwestern Drainage Basin	540	2.74	1,480
South Central Drainage Basin	140	2.22	311
Southern Drainage Basin	535	2.22	1,188
Waterford Drainage Basin	1,068	2.74	2,926
Total	**4,806**	**2.67**	**12,818**

The commercial low-user and commercial regular PE were calculated by assuming that low-users are equivalent to one residential unit, and that regular commercial users are equivalent to two residential units. Therefore, the same PE/unit value that was used to calculate residential PE was again used to calculate the commercial low-user PE (142 PE). It was doubled to calculate a total regular commercial PE (75 PE). Table 2-2 lists the commercial low-user PE by drainage basin and Table 2-3 summarizes the regular commercial PE by basin.

Table 2-2: Commercial Low-User PE Breakdown by Basin

Drainage Basin	Commercial Low - Inside NMWRD Corporate Boundary		
	Low User Units	PE/Unit	PE
Central Drainage Basin	0	2.74	0
Eastern Drainage Basin	2	2.74	5
Near East Drainage Basin	13	2.74	36
Northeastern Drainage Basin	1	2.74	3
Northwestern Drainage Basin	29	2.74	79
South Central Drainage Basin	7	2.22	16
Southern Drainage Basin	2	2.22	4
Waterford Drainage Basin	0	2.74	0
Total	**54**	**2.63**	**142**

Table 2-3: Commercial Regular PE Breakdown by Basin

Drainage Basin	Commercial Regular - Inside NMWRD Corporate Boundary		
	Regular Units	PE/Unit	PE
Central Drainage Basin	0	5.48	0
Eastern Drainage Basin	0	5.48	0
Near East Drainage Basin	4	5.48	22
Northeastern Drainage Basin	0	5.48	0
Northwestern Drainage Basin	9	5.48	49
South Central Drainage Basin	0	4.44	0
Southern Drainage Basin	1	4.44	4
Waterford Drainage Basin	0	5.48	0
Total	**14**	**5.36**	**75**

Water usage data was available for the metered commercial users. This data was separated by basin to get a total usage in thousands of gallons per month in each basin. The monthly volume was then divided by 30 to compute gallons per day. Daily usage was divided by an average per capita daily use of 70 gcd to determine a contribution of 659 PE due to commercial metered users. Table 2-4 summarizes the commercial metered PE for each basin.

Table 2-4: Commercial Metered PE Breakdown by Basin

Drainage Basin	Commercial Metered- Inside NMWRD Corporate Boundary		
	Metered Units	Usage (1000 gal/month)	PE [(1)]
Central Drainage Basin	6	117	56
Eastern Drainage Basin	2	55	26
Near East Drainage Basin	38	349	166
Northeastern Drainage Basin	5	302	144
Northwestern Drainage Basin	11	20	10
South Central Drainage Basin	9	107	51
Southern Drainage Basin	1	1	0
Waterford Drainage Basin	2	430	205
Total	**74**	**1,381**	**659**

(1) Based on assumed per capita wastewater generation rate of 70 gcd.

Table 2-5 summarizes the overall PE within the NMWRD corporate boundary for both residential and non-residential use resulting in a total current population equivalent of 13,695 PE within the incorporated area of the District.

Table 2-5: Total Residential & Commercial PE Breakdown by Basin

Within NMWRD Corporate Boundary			
Drainage Basin	Existing Residential PE	Existing Non-Residential PE	Total Existing PE
Central Drainage Basin	3,099	56	3,155
Eastern Drainage Basin	38	31	69
Near East Drainage Basin	2,011	224	2,235
Northeastern Drainage Basin	1,765	147	1,912
Northwestern Drainage Basin	1,480	138	1,618
South Central Drainage Basin	311	67	378
Southern Drainage Basin	1,188	9	1,197
Waterford Drainage Basin	2,926	205	3,131
Total	**12,818**	**877**	**13,695**

2.3.3 Wastewater Flows and Loadings

Table 2-6 compares the current (July 2013 through June 2014) influent wastewater flows and pollutant loads to the existing facility's design ratings. Future PE projections will be analyzed in the following pages to determine capacity requirements of the treatment facility at build-out throughout the Northern Moraine FPA.

Table 2-6: Current Flows and Loadings versus Design

Parameter	Average Flow (MGD)	BOD_5 (lbs/day)	TSS (lbs/day)	NH_3-N (lbs/day)
Design Loading	2.0	2,800	3,370	417
Current Loading	1.05	1,793	1,595	227
Current Percent of Design	**53%**	**64%**	**47%**	**54%**

2.4 FUTURE CONDITIONS

2.4.1 General

Wastewater facilities planning typically addresses projected needs over a 20-year planning period. Each of the communities within NMWRD's planning area has adopted or created a Comprehensive Plan. These plans, in conjunction with tax parcel data provided by Lake and McHenry Counties, were used to determine the future population projections within the Northern Moraine FPA.

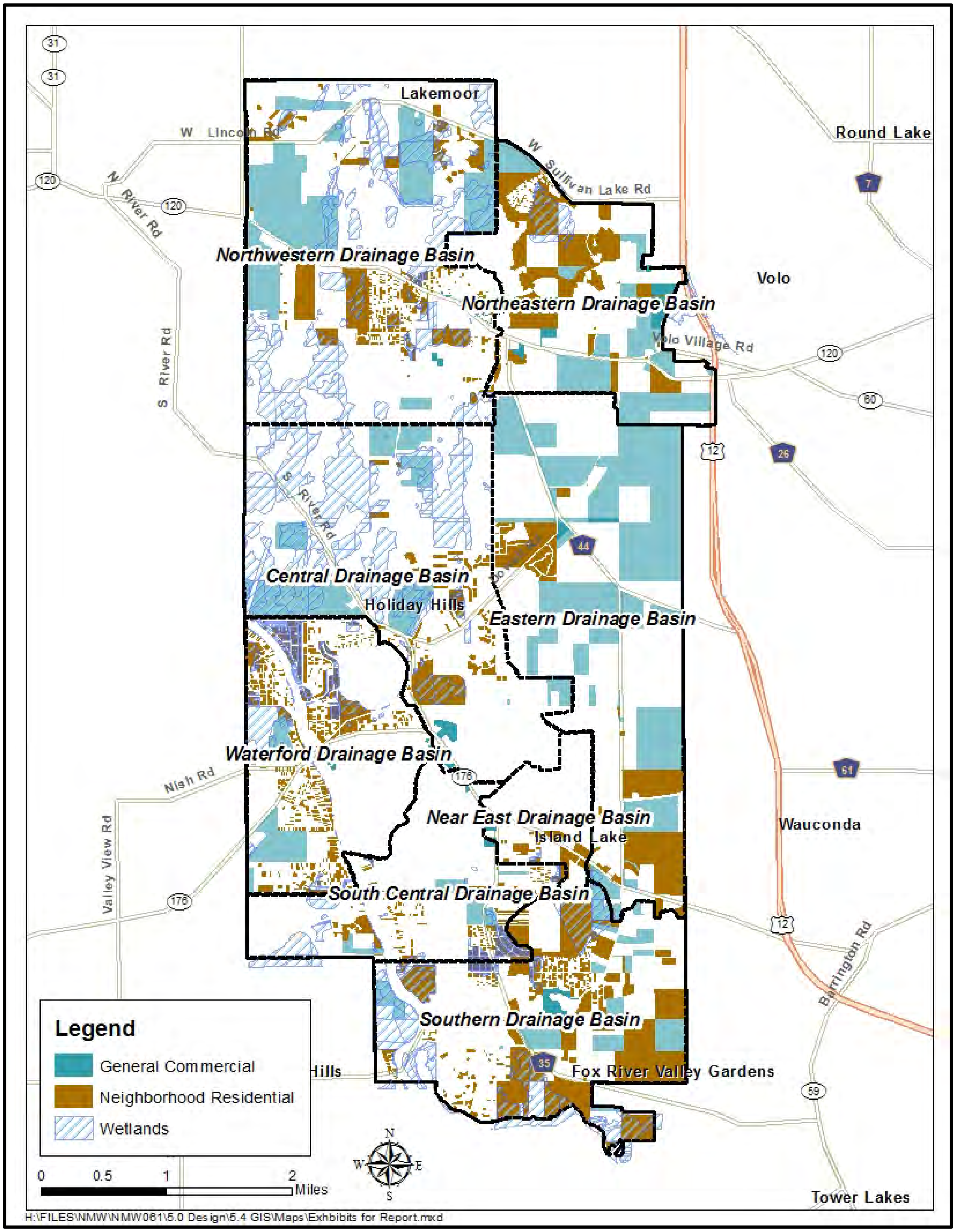
The Village of Lakemoor adopted a Comprehensive Plan in February of 2013 as an update to their 2003 Comprehensive Plan. The Lakemoor plan discusses future land use and population projection within the Village. Throughout the document, the importance of preserving agricultural lands and natural resources was emphasized. Therefore, future development was not considered in these specified areas.

The Village of Island Lake Comprehensive Plan was completed in December of 2012. Various goals and objectives were outlined within the plan to assist Island Lake in enhancing the Route 176 Corridor. Future land use and population projections along this corridor are discussed in the Island Lake planning document.

The Village of Port Barrington adopted a Comprehensive Plan in September of 2013. Much of Port Barrington has been developed with only a few remaining tracts of land available, and the Village does not actively seek to annex more land. If these areas are annexed to the Village in the future, it is proposed they will be developed as residential property.

The future PE to be used for wastewater facilities planning was calculated with consideration given to the comprehensive plans of each community, as well as using the county tax parcel data and gaining input from District staff. A map of planned land use within the Northern Moraine FPA was constructed using these sources and is shown on Exhibit 2-3.

Exhibit 2-3: Future Land Use



2.4.2 Population Equivalent Projections

Future growth within the District can be categorized into two types: development within the corporate boundary and development outside the corporate boundary, but within the facility planning area boundary. The area between the two boundaries is referred to as either the expansion area or the unincorporated area.

Future population equivalents for each drainage basin were calculated using parcel data provided by the District, McHenry County, and Lake County. Existing wetlands acreage were first subtracted from the total expansion acreage. A general use multiplier was applied to account for right-of-way, easements, open space, etc. (0.80 for residential, 0.85 for commercial).

For residential land use, the net acreage was then multiplied by the housing density in units/acre based on either existing parcel data or county zoning ordinances. County ordinances for both Lake and McHenry County specify a minimum residential lot size from which a maximum unit/acre value can be calculated.

In McHenry County, the minimum lot size is 0.5 acres, equating to a maximum units/net acre value of 2. In Lake County, the minimum lot size for a residential parcel is 8,500 square feet, or 0.2 acres. This equates to a maximum units/net acre value of 5.

If the parcel data resulted in a units/acre value greater than the maximum units/acre dictated by each county, the county zoning ordinance was used. This value was then multiplied by future values of 3.5 PE/unit for residential use and 12 PE/unit for commercial use to compute the total PE within each basin.

Based on these methods of calculating future PE, a summary of the existing and future projected PE is provided in Table 2-7. The total future PE calculated for each drainage basin at build-out of the unincorporated areas is summarized in Table 2-8.

Table 2-7: Population Equivalent Projections

	Incorporated FPA	Unincorporated FPA	Built-Out FPA
Existing Residential PE	12,818	0	
Future Residential PE Growth	1,878	32,910	
Total Projected Residential PE	**14,696**	**32,910**	**47,606**
Existing Non-Residential PE	877	0	
Future Non-Residential PE Growth	757	7,220	
Total Projected Non-Residential PE	**1,634**	**7,220**	**8,854**
Total Projected PE	**16,330**	**40,130**	**56,460**

Table 2-8: Current and Projected PE Breakdown by Basin

Drainage Basin	Existing PE	Additional Future PE	Total Build-Out PE
Central Drainage Basin	3,155	2,270	5,425
Eastern Drainage Basin	69	5,254	5,323
Near East Drainage Basin	2,235	1,252	3,487
Northeastern Drainage Basin	1,912	12,675	14,587
Northwestern Drainage Basin	1,618	5,625	7,243
South Central Drainage Basin	378	1,707	2,085
Southern Drainage Basin	1,197	10,292	11,489
Waterford Drainage Basin	3,131	3,690	6,821
Total PE	**13,695**	**42,765**	**56,460**

Based on the previous discussion, the existing residential population within the District was estimated to be 12,818 PE. The residential population is projected to increase by 1,878 PE within the corporate boundary and 32,909 PE within the unincorporated area. The non-residential PE (commercial, manufacturing, institutional, etc.) is expected to increase within the corporate and facility planning boundaries by 757 PE and 7,220 PE, respectively, in addition to the current 877 commercial PE. The total projected PE at build-out of the Northern Moraine FPA is 56,460 PE.

Northwest and Northeast Basin Population Projections

The residential population of the Village of Lakemoor historically averaged 4.3 percent growth per year from 1990 to 2013, ignoring the years 2000 and 2003 because of atypical growth in those years. The Northwest and Northeast Basin border Lakemoor.

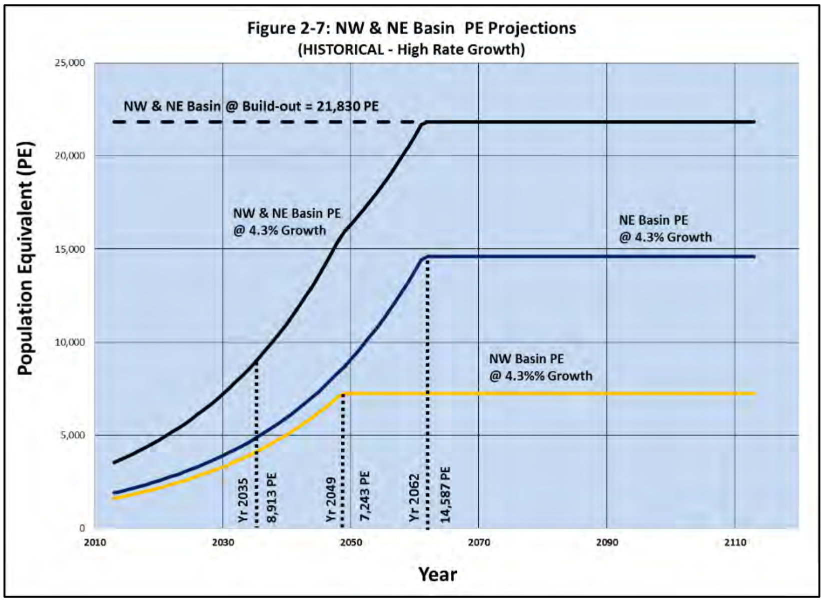


Figure 2-7 shows that if this historical growth rate is projected forward, build-out of the Northwest and Northeast Basins is projected will occur in the years 2049 and 2062, respectively. The combined 20-year projected PE within these basins would be equal to 8,913 PE.

However, as discussed earlier, development in Lakemoor slowed over the past 10 years and has been flat since 2010. Chicago Metropolitan Planning Commission (CMAP) population and employment projections for the year 2040 were reviewed to determine alternative, more modest growth rates throughout the Northern Moraine FPA. For the Village of Lakemoor and its surrounding areas, CMAP projects an annual growth rate of approximately 2.4 percent between the years 2010 and 2040.

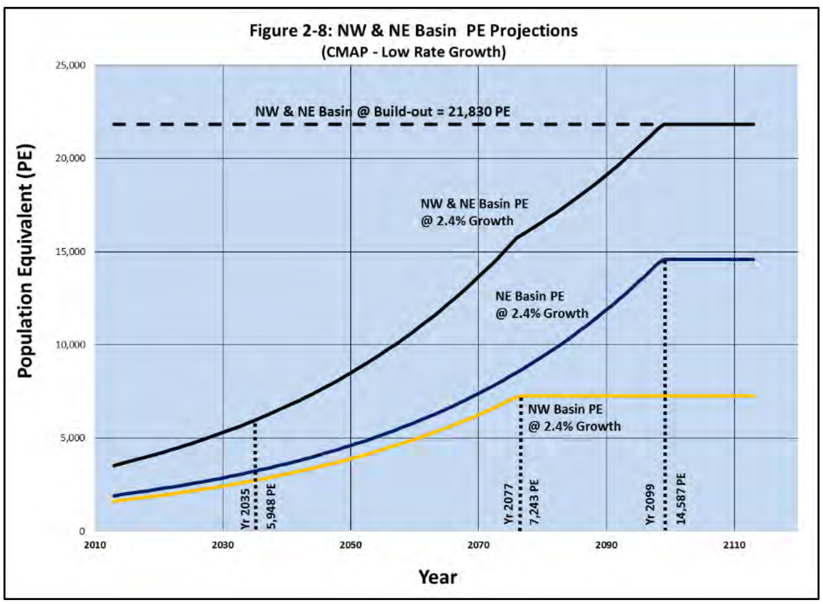


Figure 2-8 shows the revised growth projections for the Northwest and Northeast Basins using CMAP growth rates. Based on these reduced growth expectations, build-out of the Northwest and Northeast Basins is not projected to occur until the years 2077 and 2099, respectively. The combined 20-year PE (year 2035) within these basins would be equal to 5,948 PE.

Near East, Central and Waterford Drainage Basins Population Projections

The residential population of the Village of Island Lake historically averaged 2.6 percent growth per year from 1990 to 2013. The Near East, Central and Waterford Drainage Basins border Island Lake.

Figure 2-9: Near East, Central and Waterford Basin PE Projections
(HISTORICAL - High Rate Growth)

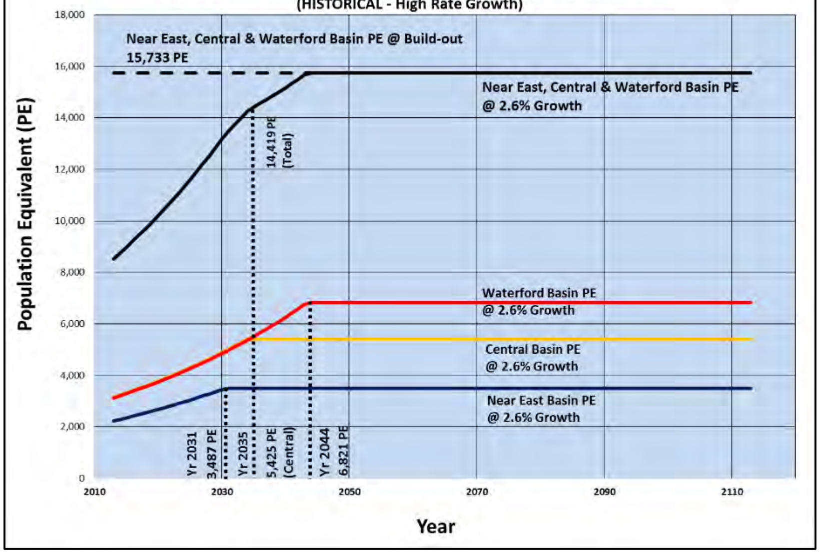


Figure 2-9 shows that if this historical growth rate is projected forward, build-out of the Near East Basin would reach build-out in the year 2031, the Central Basin would reach build-out in the year 2035, and the Waterford Basin would reach build-out in the year 2044. The combined 20-year PE in these basins would be equal to 14,419 PE.

Similar to but even more extreme than in Lakemoor, growth in Island Lake has generally been nonexistent over the past 10 years, and the population actually decreased significantly in 2010.

CMAP population and employment projections for the year 2040 were reviewed to determine an alternative, more modest growth rate which for Island Lake is projected by CMAP to be approximately 0.8 percent per year from the year 2010 through the year 2040.

Figure 2-10: Near East, Central and Waterford Basin PE Projections
(CMAP - Low Rate Growth)

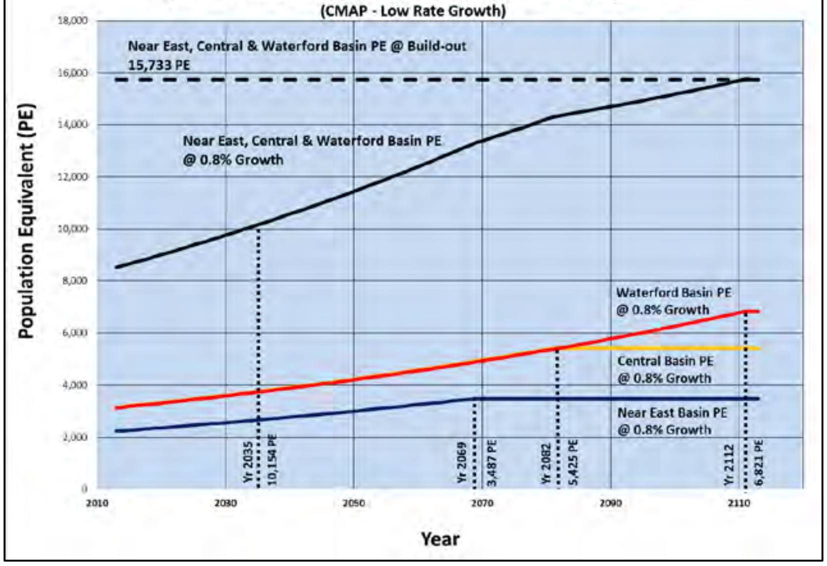
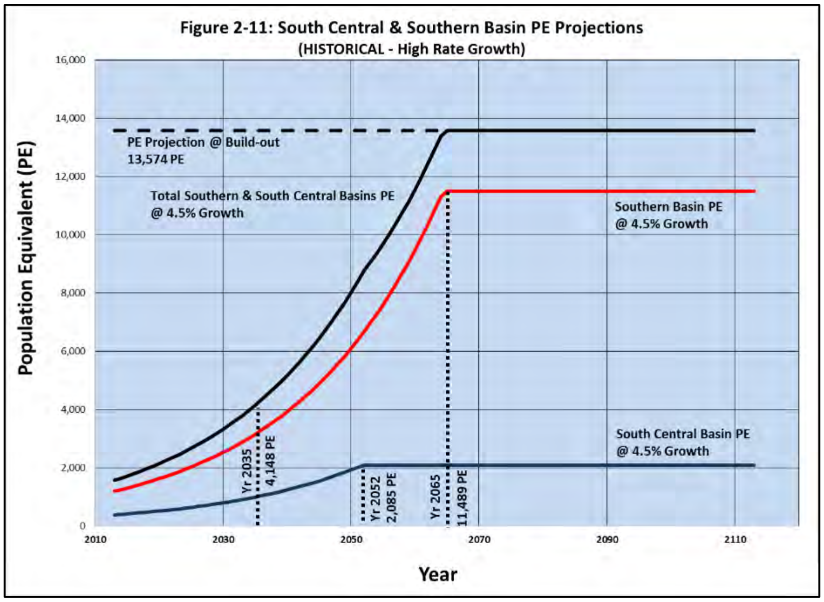


Figure 2-10 shows the revised growth projections for the Near East, Central, and Waterford Basins using CMAP growth rates. Based on these reduced growth expectations, build-out of the Near East, Central, and Waterford Basins is not projected to occur until the years 2069, 2082, and 2112, respectively. The 20-year (year 2035) projected PE would be equal to 10,154 PE.

South Central and Southern Basin Population Projections

The residential population of the Village of Port Barrington historically averaged 4.5 percent growth per year from 2000 to 2013. The South Central and Southern Basins are located in the southern reaches of the Northern Moraine FPA between Island Lake and Port Barrington.

Figure 2-11 shows that if this historical growth rate is projected forward, build-out of the South Central Basin would reach build-out in the year 2052, and the Southern Basin would reach build-out in the year 2065. The combined 20-year PE in these basins would be equal to 4,148 PE.



With the exception of 2010, little to no growth has occurred in Port Barrington since 2005.

CMAP population and employment projections for the year 2040 were reviewed to determine an alternative, more modest growth rate which for Port Barrington is projected by CMAP to be approximately 0.5 percent per year from the year 2010 through the year 2040.

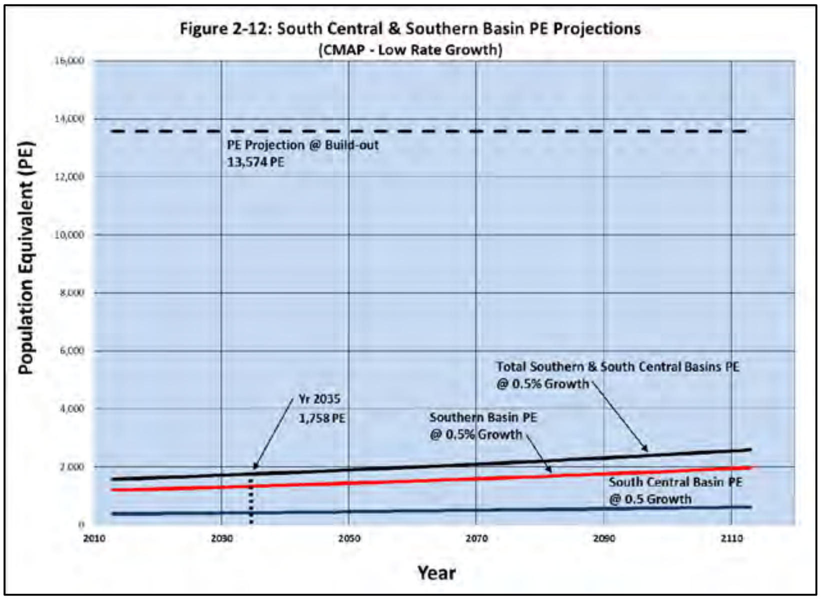


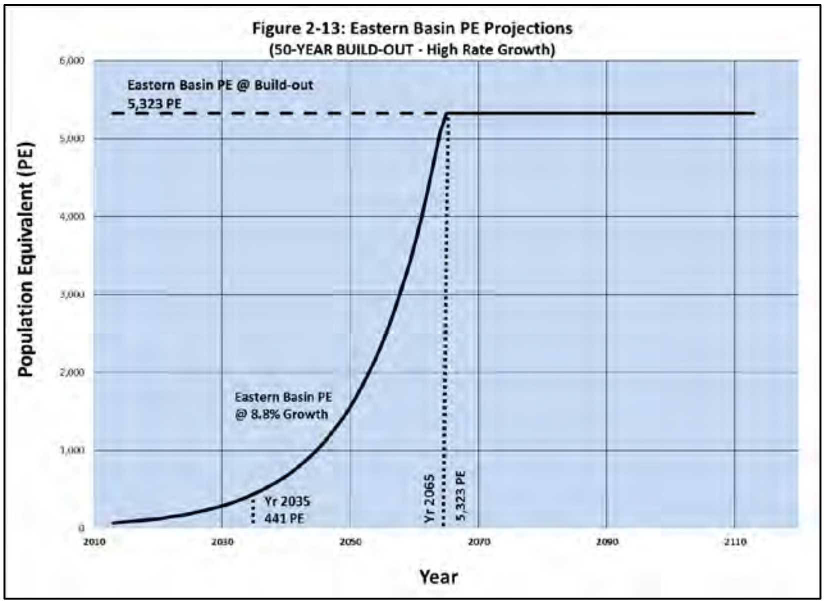
Figure 2-12 shows the revised growth projections for the South Central and Southern Basins using CMAP growth rates. Based on these reduced growth expectations, build-out of the South Central Basin would not occur for 340 years, or the year 2355. Similarly, build-out of the Southern Basin would be pushed out for over 450 years to the 2467.

The 20-year (year 2035) projected PE would be equal to 1,758 PE.

Eastern Basin Population Projections

The currently connected population equivalent is only 69 PE, which is projected to grow to 5,323 PE at build-out. Because of the large percentage of growth at build-out, the Eastern Basin was projected separately from the remainder of the drainage basins. Growth was projected at that rate necessary to reach build-out within a 50-year planning period.

For the build-out population equivalent of the Eastern Basin to reach 5,323 PE by the year 2065 (50 years), annual growth in the Eastern Basin would need to average 8.8 percent, as shown on Figure 2-10.



The Eastern Basin has significant potential for growth. Interest in developing the land has varied over the past few decades, and has been waned in recent years due to the economic downturn.

CMAP 2040 growth rate projections for Lakemoor and its surrounding areas (2.4%) were applied to the current customer base in the Eastern Basin to determine more modest growth projections.

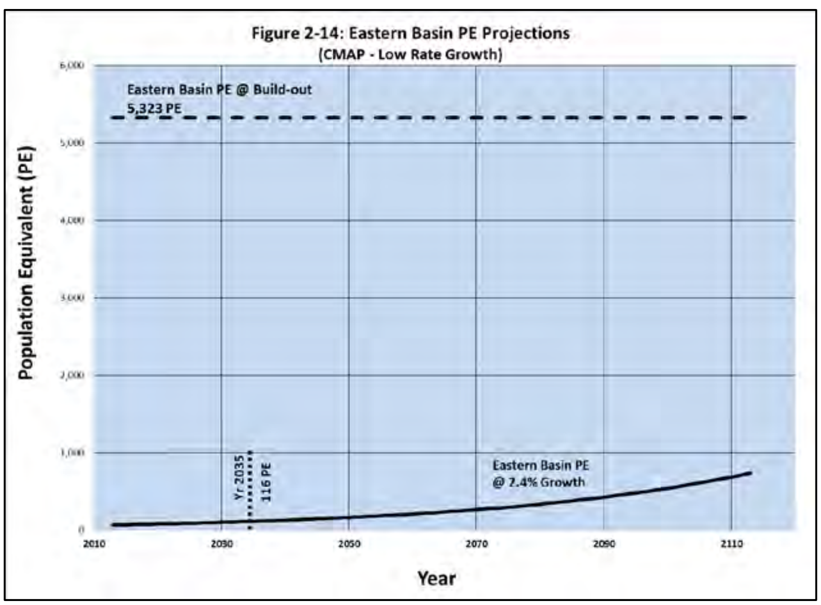


Figure 2-14 shows the revised growth projections for the Eastern Basin using the CMAP growth rate for Lakemoor. Based on this reduced growth expectation, build-out of the Eastern Basin would not occur for 182 years, or the year 2197.

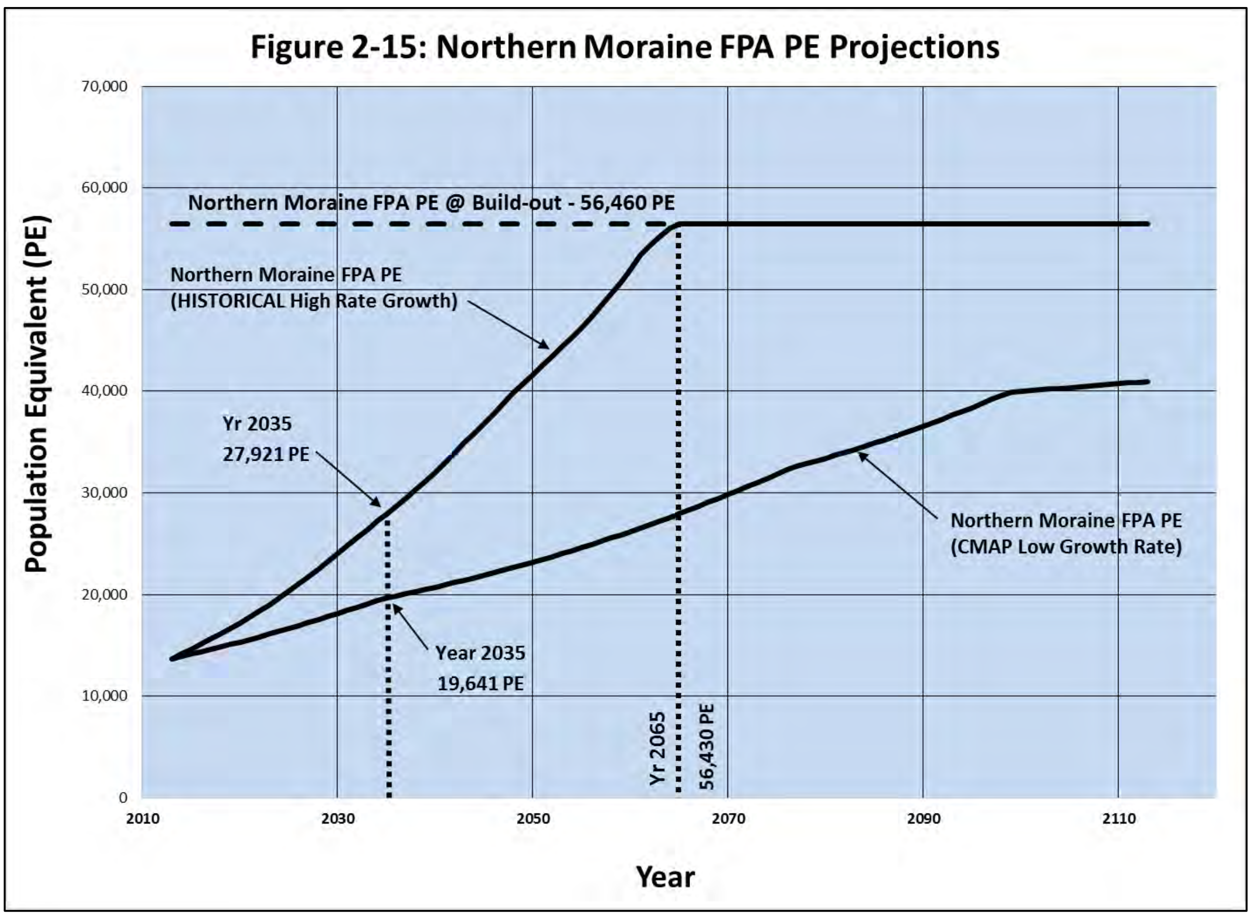
The 20-year (year 2035) projected PE would be equal to 116 PE. This equates to construction of only 2 residential units per year for the next two decades.

Full Northern Moraine FPA Population Projections

The summation of the population equivalent projections shown on Figures 2-7 through 2-14 is shown on Figure 2-15. The composite annual growth rate throughout the Northern Moraine FPA based on historical (high rate growth) data would be 3.29%. The more modest CMAP (low rate growth) projections results in a District-wide weighted average growth rate of 1.65%.

Planning for expansion of the Northern Moraine WTF should commence when the average daily flow reaches 1.6 MGD, equal to 80 percent of the design flow (2.0 MGD). Based on the 2013 average daily flow of 1.05 MGD, the current population equivalent of 13,695 PE, and an assumed value of 100 gcd for additional PE, the current design capacity of 2.0 MGD corresponds to a population equivalent of 23,195 PE. Under the most optimistic, high rate growth projections, the treatment facility would reach capacity in the year 2029. Based on the more modest CMAP projections, facility's capacity would be reached in the year 2050.

Planning for the Phase 2 expansion should begin when the influent wastewater flow reaches 80 percent of the design capacity (1.6 MGD), would corresponds to a population equivalent of 19,195 PE. Under the most optimistic, high rate growth projections, 80 percent capacity would be reached in the year 2024. Based on the more modest CMAP projections, 80 percent capacity would be reached in the year 2034. The actual timing for design will be greatly dependent on development activity over the next 10 to 15 years.



2.4.3 Wastewater Flow and Loading Projections

The projected future wastewater flows and loading are calculated by adding the future theoretical flows and loading (as determined by IEPA design criteria) to the existing flow and loading data (as determined by NMWRD's monitoring data). For future PE, IEPA specifies a flow of 100 gcd, a BOD_5 loading of 0.17 lbs/PE/day, and a TSS loading of 0.20 lbs/PE/day. Table 2-9 summarizes the projected conditions at build-out of all areas within the current NMWRD corporate boundaries.

Table 2-9: Projected PE, Flows, and Loadings within the Current NMWRD Corporate Boundaries

Condition	PE	Flow (MGD)	BOD_5 (lbs/day)	TSS (lbs/day)	NH_3-N (lbs/day)
Current Loading	13,695	1.05	1,793	1,595	227
Additional Future Development	2,635	0.26	448	527	55
Projected Conditions	**16,330**	**1.31**	**2,241**	**2,122**	**282**
Design Loading	20,000	2.0	2,800	3,370	417
Future Percent of Design	**82%**	**66%**	**80%**	**63%**	**68%**

The projected average flow for build-out within the NMWRD corporate boundaries is 1.31 MGD, well below the existing facility's rated hydraulic capacity of 2.0 MGD. The capacity of the NMWRD treatment facility would not be exceeded, and expansion of the facility not needed, unless and until additional properties within the FPA are incorporated into the District.

Table 2-10 summarizes the projected 20-year planning period conditions for the population equivalent projections shown on Figure 2-15.

Table 2-10: Projected 20-year (year 2035) PE, Flows, and Loadings within the NMWRD FPA

Conditions	PE	Flow (MGD)	BOD_5 (lbs/day)	TSS (lbs/day)	NH_3-N (lbs/day)
CMAP (Low Rate Growth) Projections					
Current Loading	13,695	1.05	1,793	1,595	227
Additional Future Development	5,946	0.59	1,011	1,189	124
Projected 20-year Conditions	**19,641**	**1.64**	**2,804**	**2,784**	**351**
Design Loading	20,000	2.0	2,800	3,370	417
Future Percent of Design	**98%**	**82%**	**100%**	**83%**	**84%**
HISTORICAL (High Rate Growth) Projections					
Current Loading	13,695	1.05	1,793	1,595	227
Additional Future Development	14,226	1.42	2,418	2,845	297
Projected 20-year Conditions	**27,921**	**2.47**	**4,211**	**4,440**	**524**
Design Loading	20,000	2.0	2,800	3,370	417
Future Percent of Design	**140%**	**124%**	**150%**	**132%**	**126%**

The capacity of the existing NMWRD treatment facility would be exceeded within the 20-year facility planning period should development activity within the Northern Moraine FPA return to previous historical levels. However, current housing activity and leading indicators do not forecast that level of development, at least not in the near-term. Even the more modest CMAP lower rate growth projections exceed that experienced within the District over the past 5 years. As such, these projections are provided for planning purposes only and the District should monitor growth and wastewater flows throughout the FPA and plan for expansion accordingly.Table 2-11 summarizes the projected conditions at full build-out of the entire Northern Moraine FPA, including all development within both the corporate boundary and unincorporated areas.

Table 2-11: Projected PE, Flows, and Loadings within the NMWRD FPA

Condition	PE	Average Flow (MGD)	BOD_5 (lbs/day)	TSS (lbs/day)	NH_3-N (lbs/day)
Current Development	13,695	1.05	1,793	1,595	227
Future Development within NMWRD Incorporated Areas	2,635	0.26	448	527	55
Future Development within Unincorporated Areas	40,130	4.01	6,822	8,026	843
Projected Conditions	**56,460**	**5.32**	**9,063**	**10,148**	**1,125**
Design Loading	20,000	2.0	2,800	3,370	417
Ultimate Percent of Design	**282%**	**266%**	**324%**	**301%**	**270%**

The projected wastewater flow to the NMWRD treatment facility at build-out is approximately 5.33 MGD. Projected wastewater flows and pollutant loadings will exceed the treatment facility's current hydraulic and biological treatment capacities. However, instead of the previously planned expansion up to 10 MGD, the long range planning can be scaled back to provide for an ultimate average wastewater flow of approximately 6 MGD.

Expansion of the treatment facility would be implemented in phases, with the initial improvements most likely comprised of expansion from the current capacity of 2 MGD to 3 MGD by constructing a third ring to the existing oxidation ditch. However, even the most optimistic growth projections do not indicate that capacity will be exceeded for at least 15 years. CMAP projections indicate that the capacity of the existing treatment facility will not be exceeded for 35 years.

Provisions to extend and improve wastewater infrastructure must be made to address future build-out of the Facility Planning Area. The analysis that follows addresses future development within each wastewater drainage basin.

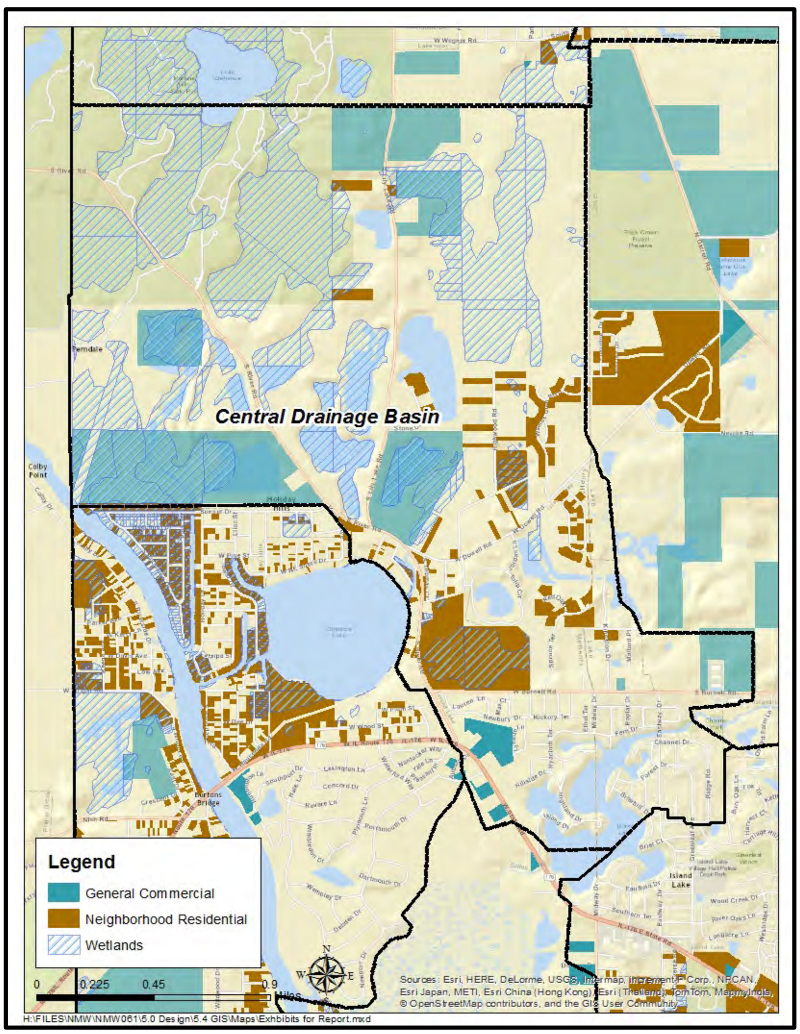
2.4.4 Central Drainage Basin

The Central Drainage Basin currently includes about 3,960 PE and is served by the Clearwater, Walnut Glen, Prairie Woods, Rolling Oaks, and Fern lift stations.

The expansion of service within the Central Basin includes the extension of service to the Village of Holiday Hills to the west, and to properties to the north and east along Dowell Road. These expansion areas would be served by sewer extensions that would convey wastewater to a regional lift station located on River Road directly north of Griswold Lake. Construction of the Darrell Road Collection System (see Section 3) will free capacity to allow this lift station to pump into the existing parallel 8-inch and 12-inch force mains on Lily Lake Road.

The projected residential population growth within the Central Basin is estimated to be 1,896 PE. The projected commercial PE growth was estimated to be 373 PE for a total projected additional PE of 2,270. This equates to a total PE at build-out of the Central Basin of 6,230 PE.

Exhibit 2-4: Central Drainage Basin



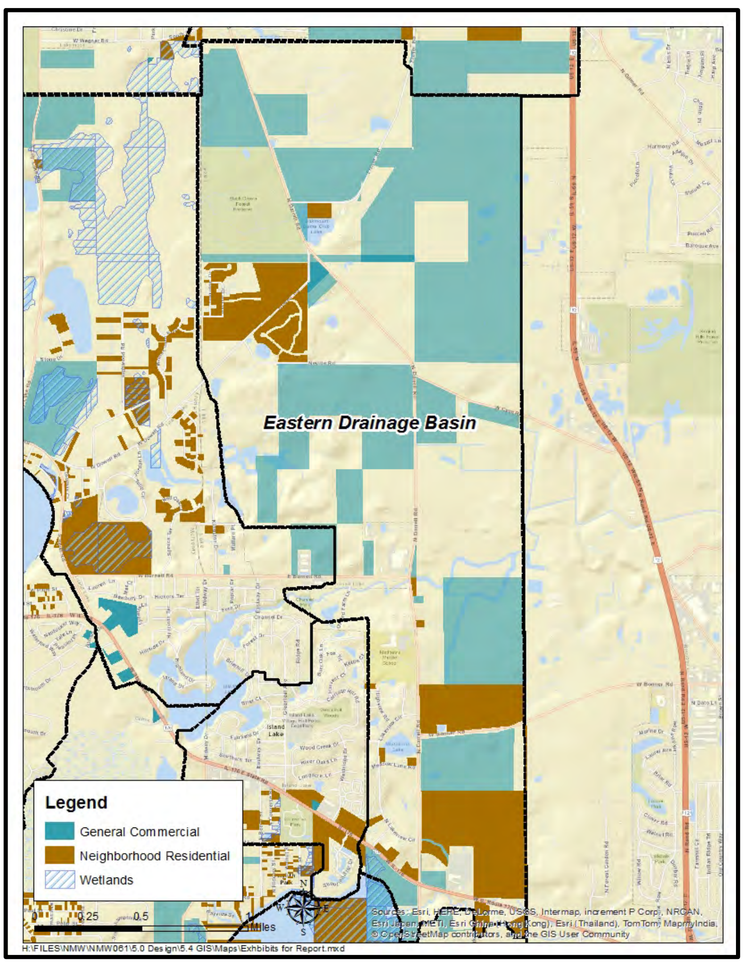
2.4.5 Eastern Drainage Basin

The Eastern Drainage Basin currently includes approximately 70 PE and is served by the Walnut Glen and Burr Oak lift stations.

Development of the Eastern Expansion Area will at some point require the installation of the lower reaches of the Darrell Road Collection System, including the Darrell Road Interceptor and the Mutton Creek regional lift station and force main. The Darrell Road Collection System will also convey flows from the Northeast Basin currently pumped to Island Lake through a 12-inch force main aligned along Lily Lake and River Roads. Completion of the Darrell Road Collection System would extend an additional service to Lakemoor, and would allow for the development of the eastern half of the District from Illinois Route 176 on the South to the Village of Volo to the North. Further discussion of the proposed interceptor sewers and the lift station are included in Sections 3 and 4.

Future development in the Eastern Basin is projected will increase the residential customer base by 2,869 PE and the commercial customer base by 2,385 PE for a total additional PE of 5,254. This equates to a total PE at build-out of the Eastern Basin of 5,324 PE.

Exhibit 2-5: Eastern Drainage Basin



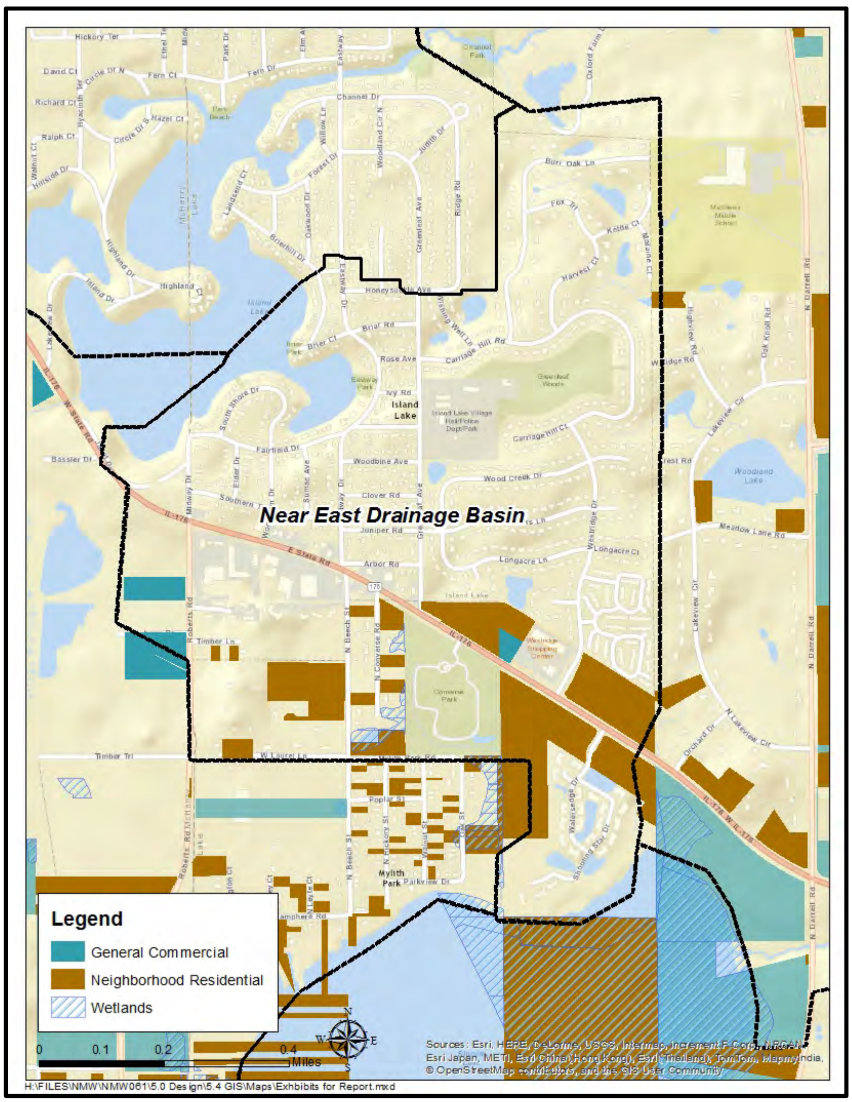
2.4.6 Near East Drainage Basin

The Near East Drainage Basin currently includes approximately 1,433 PE comprised of both residential and commercial development along the Route 176 corridor. This basin is served by the Burr Oak, South Shore, Water's Edge and Westridge Lift Stations.

Wastewater collected from the East and Northeast Drainage Basins are planned to be conveyed through the Near East Basin. As those flows increase the interceptor sewers and lift stations will reach capacity. These conditions are proposed to be relieved through construction of a new interceptor sewer from the treatment facility east to the site of the Water's Edge Lift Station.

It is projected that future development will contribute an additional residential PE of 1,151 and commercial PE of 100 for a total additional customer base of 1,252 PE. This equates to a total PE at build-out of the Near East Drainage Basin of 2,684 PE.

Exhibit 2-6: Near East Drainage Basin

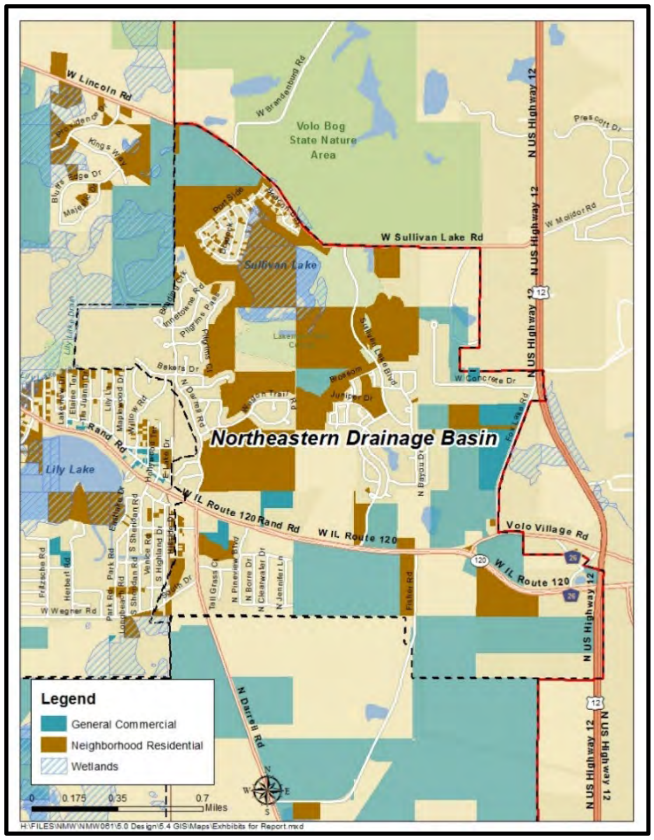


2.4.7 Northeastern Drainage Basin

The Northeastern Basin is located directly to the East of the Village of Lakemoor. Some of this property is located within the Village of Lakemoor, while some of the property is located in unincorporated Lake County. The basin currently includes approximately 1,912 PE and is served by Lakemoor Lift Stations 5, 6, and 7.

The Northeastern Drainage Basin is located 4-1/2 miles northeast of the existing NMWRD treatment facility. Currently, wastewater is pumped through a four-mile long 12-inch force main along Lily Lake Road, to the District's 24-inch interceptor sewer installed along Illinois 176. As the Northeastern Drainage Basin develops, the capacity of Lakemoor Lift Station 7 and its force main will be exceeded. It is proposed that once the current surplus capacity is exhausted, this condition be relieved by diverting Lift Station 7 to the upstream reach of the Darrell Road Interceptor Sewer. The entire Darrell Road Collection System is discussed in detail in Section 3.

Exhibit 2-7: Northeastern Drainage Basin



Future development in the Northeastern Basin is projected to increase the residential PE by 9,679 PE and the commercial PE by 2,997 PE for a total additional PE of 12,676 PE. This equates to a total PE at build-out of the Northeastern Drainage Basin of 14,589 PE.

The Rockwell Place Development is located in the Northeastern Basin, and is currently served by a private collection and treatment system designed to serve 3,210 PE. Since this system is located within the Northern Moraine FPA, the District is the Designated Management Agency (DMA). The District may be called upon to provide wastewater service to this development should the existing private system fail.

The population projections for the Northeast Basin include the population equivalent associated with connection of the Rockwell Place Development to the NMWRD collection and treatment system. In Section 4 it will be shown that Lakemoor Lift Station 7, its force main, and the downstream 24-inch Route 176 West Interceptor each currently have sufficient capacity to convey the Rockwell flow. Should the District decide to serve Rockwell, most all of the available capacity in the Route 176 West Interceptor would be consumed, thereby prohibiting development throughout the remainder of the Northwestern and Northeastern Drainage Basins, and thus triggering the necessity for either the Darrell Road Collection System or the Interim Solution Collection System.

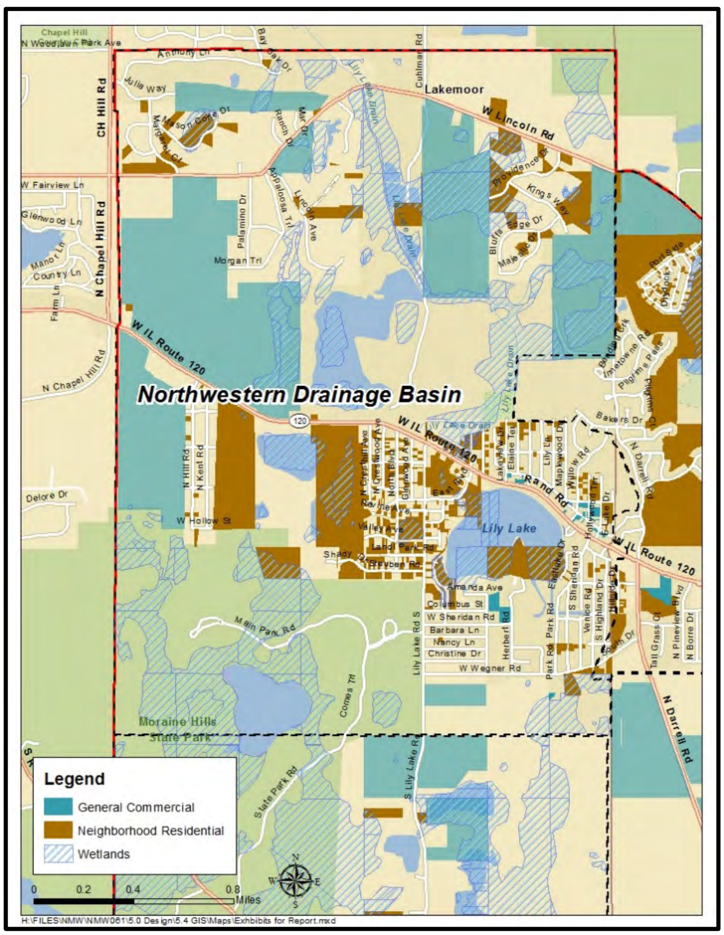
2.4.8 Northwestern Drainage Basin

The Northwestern Drainage Basin currently includes approximately 1,619 PE and is served by Lakemoor Lift Stations 1, 2, 3, 4, and 5. The Northwestern Expansion Area is located directly northwest of the Village of Lakemoor and encompasses property within the Village as well as land in unincorporated McHenry County.

Like the Northeastern Expansion Area, the Northwestern Expansion Area is located remotely from the NMWRD treatment facility. Currently, this area's wastewater is pumped through a four-mile long 8-inch diameter force main to the District's 24-inch interceptor sewer along Route 176. As the Northwestern Drainage Basin develops, the capacity of Lakemoor Lift Station 1 and its force main will be exceeded. It is proposed to address this situation by using the parallel 12-inch force main along Lily Lake Road that will be vacated if and when Lift Station 7 is diverted to the Darrell Road Collection System in the Northeast Basin as discussed in Section 3.

It is projected that the residential customer base will increase by 4,603 PE while the commercial base will increase by 1,022 PE for an additional contribution of 5,625 PE. This equates to a total PE at build-out of the Northwestern Drainage Basin of 7,244 PE.

Exhibit 2-8: Northwestern Drainage Basin



Northwestern Expansion Area – Service Areas

To further study the NW Expansion Area, it was divided into four wastewater service areas as shown on Exhibit 2-9. These service areas do not include low-density areas that can be served by on-site private septic systems. Population equivalent and wastewater flow projections within each service area are summarized in Table 2-12.

Exhibit 2-9: Northwestern Drainage Basin Service Areas

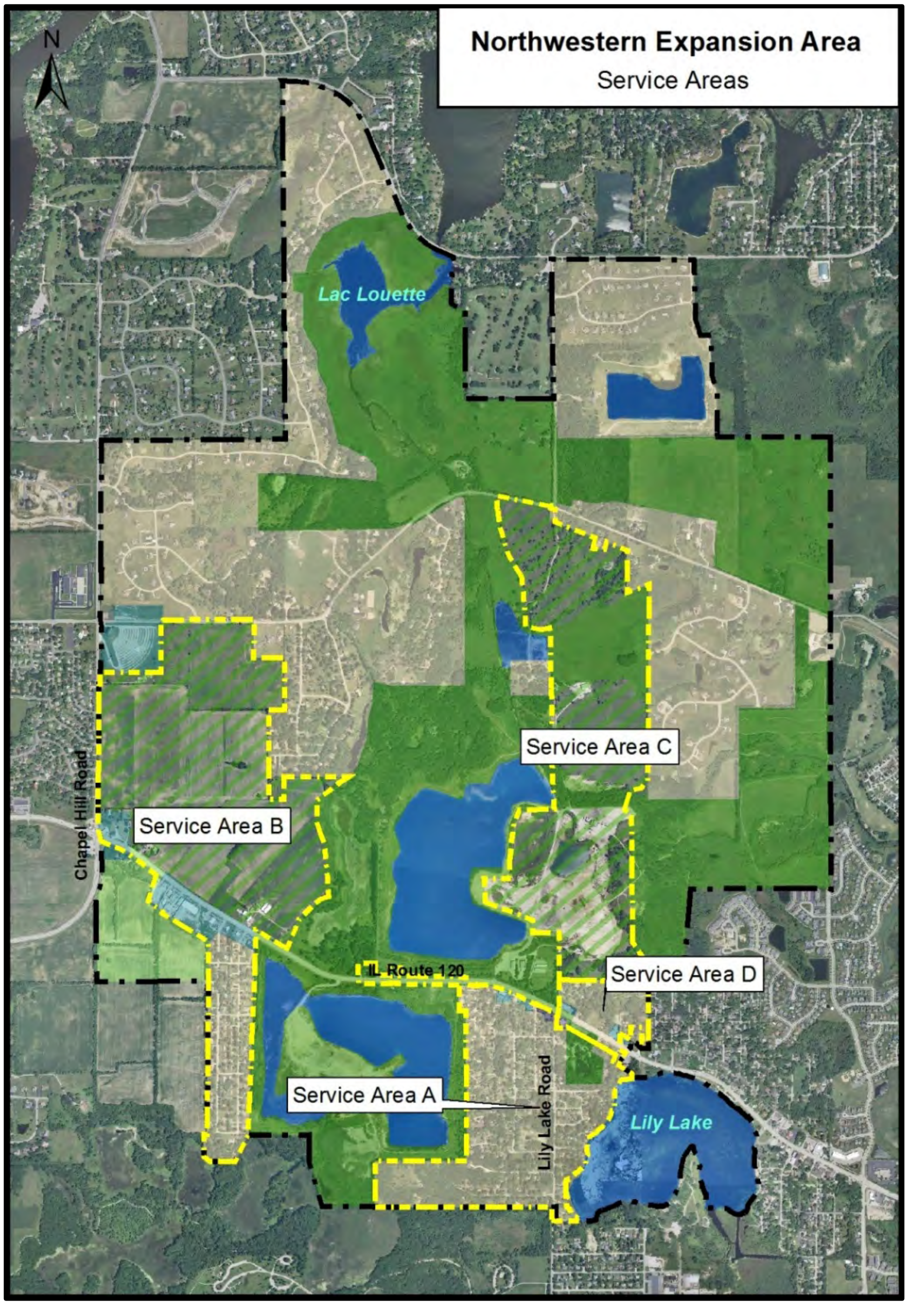


Table 2-12: Northwest Expansion Area PE and Wastewater Flows by Service Area

Service Area	Projected PE	Daily Average Flow (MGD)	Peak Hourly Flow (MGD)
Service Area A	746	0.07	0.27
Service Area B	2,820	0.28	0.96
Service Area C	1,826	0.18	0.65
Service Area D	233	0.02	0.08
Totals	**5,625**	**0.55**	**1.76**

Northwestern Expansion Area – Service Area A

Service Area A has an estimated PE of 783 PE from existing Neighborhood Residential, Commercial and Institutional properties directly west of Lily Lake.

Northwestern Expansion Area – Service Area B

Service Area B contains existing unsewered Commercial and Neighborhood Residential and future Conservation Neighborhood properties west on Illinois Route 120 to Chapel Hill Road. The PE within Service Area B is 2,960.

Exhibit 2-10: NW Expansion Area Service Area A

Exhibit 2-11: NW Expansion Area Service Area B

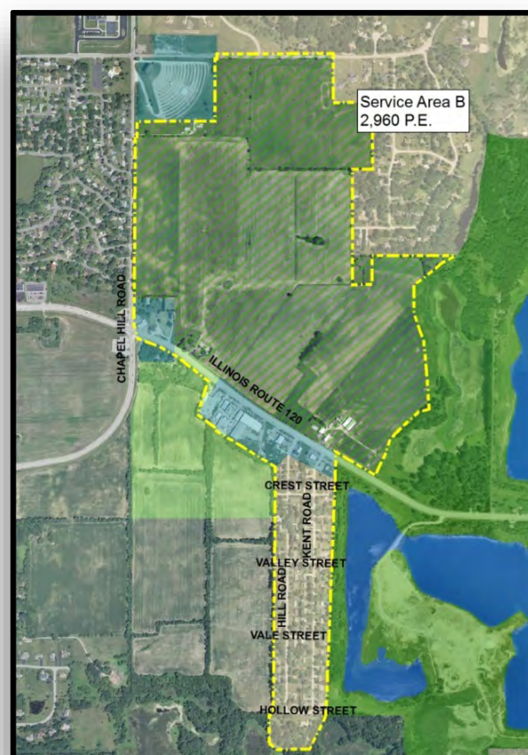


Exhibit 2-12: NW Expansion Area Service Area C

Northwestern Expansion Area – Service Area C

Service Area C contains future Conservation Neighborhoods and Open Space north of Illinois Route 120 and has an estimated PE of 1,910.

Exhibit 2-13: NW Expansion Area Service Area D

Northwestern Expansion Area – Service Area D

Service Area D contains a combination of existing Commercial and Neighborhood Residential and future Neighborhood Residential areas and has an estimated PE of 242.

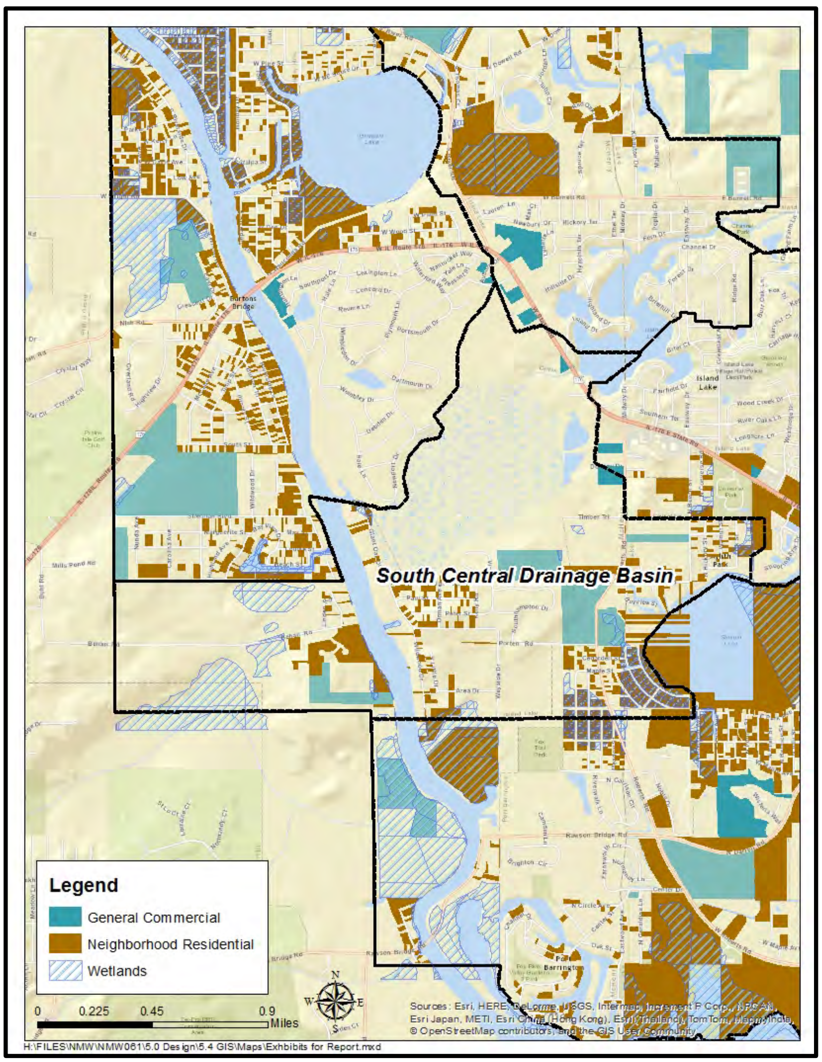
2.4.9 South Central Drainage Basin

The South Central Drainage Basin currently includes approximately 378 PE and is served by the Treatment Plant Lift Station. The NMWRD wastewater treatment facility is also located within the South Central Drainage Basin.

The South Central Expansion Area can be served through the extension of sewers. Similar to other areas in the District, the connection of existing residences will become more and more likely as existing septic systems continue to fail. It is important that the District be proactive with residents that are interested in annexing to the District. Once the East Side Interceptor is constructed, areas from this basin can be served.

The projected growth within the South Central Drainage Basin includes as additional residential PE of 1,555 PE and an additional commercial PE of 152 PE for a total addition of 1,707 PE. This equates to a total PE at build-out of the South Central Drainage Basin of 2,084 PE.

Exhibit 2-14: South Central Drainage Basin



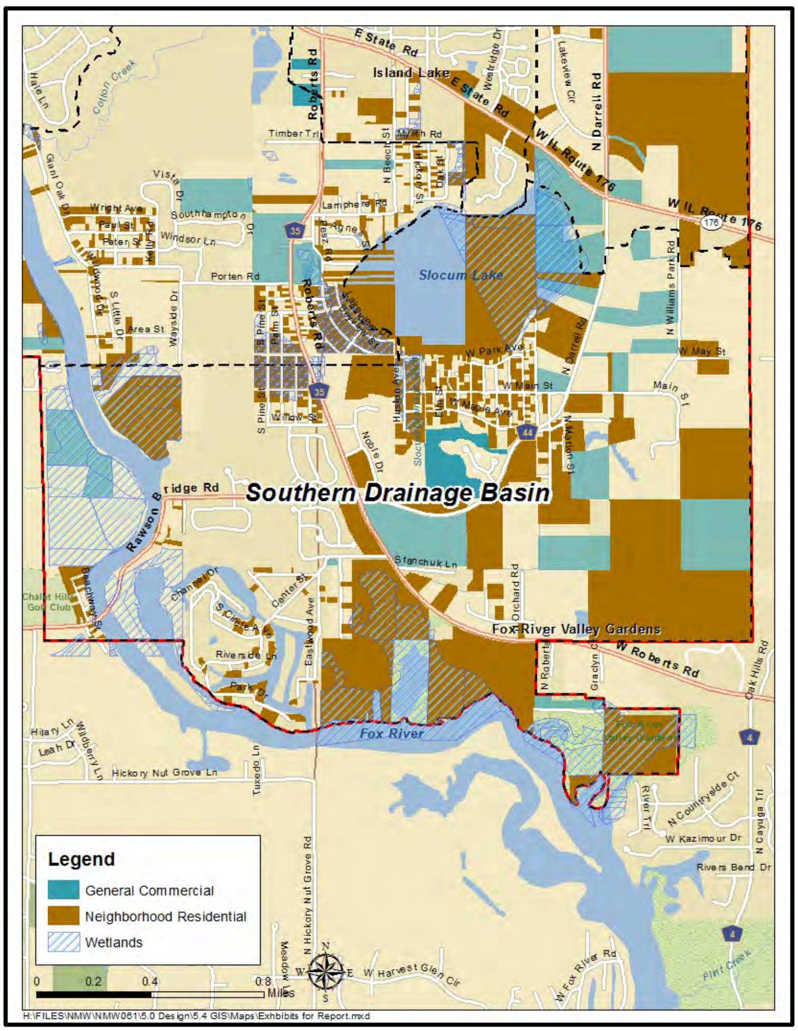
2.4.10 Southern Drainage Basin

The Southern Drainage Basin currently includes approximately 1,198 PE and is served by the Deer Grove North and Rawson Bridge lift stations.

The Southern Service Area can be served, in general, without the District constructing an extensive amount of infrastructure. The Deer Grove North Lift Station now serves the basin area to the west. In 2008, low-pressure sewers were installed in portions of Port Barrington to connect to the District's collection system.

It is projected that at build-out of the Southern Drainage Basin, the residential customer base will be increased by 9,839 PE and the commercial customer base by 453 PE, for a total additional contribution of 10,292 PE and a total PE at build-out of the Southern Basin of 11,490 PE.

Exhibit 2-15: Southern Drainage Basin



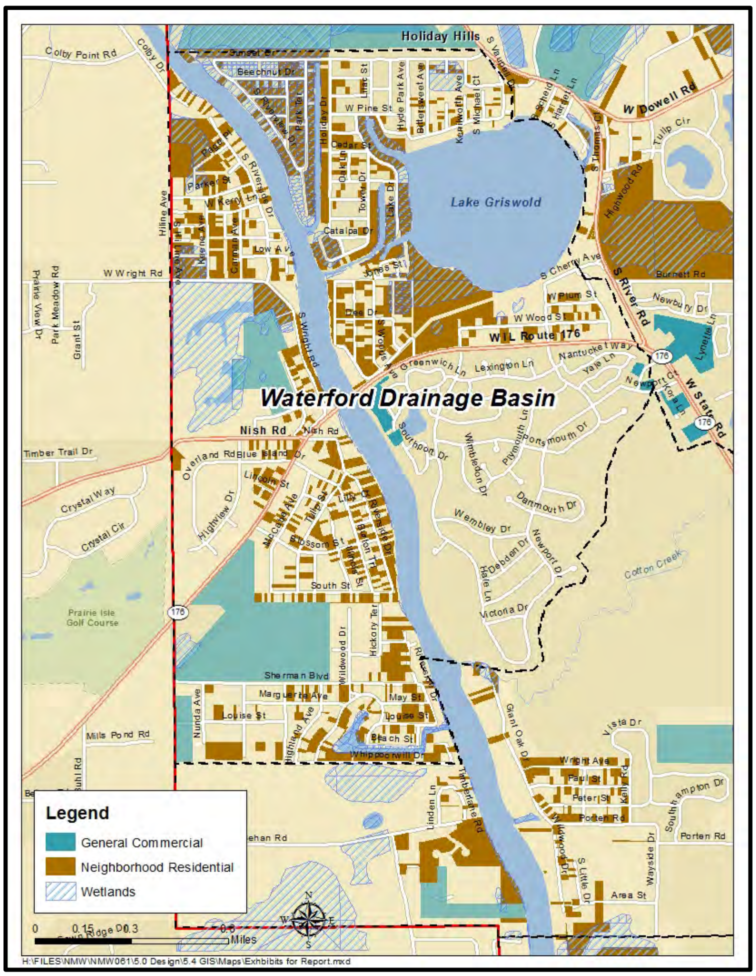
2.4.11 Waterford Drainage Basin

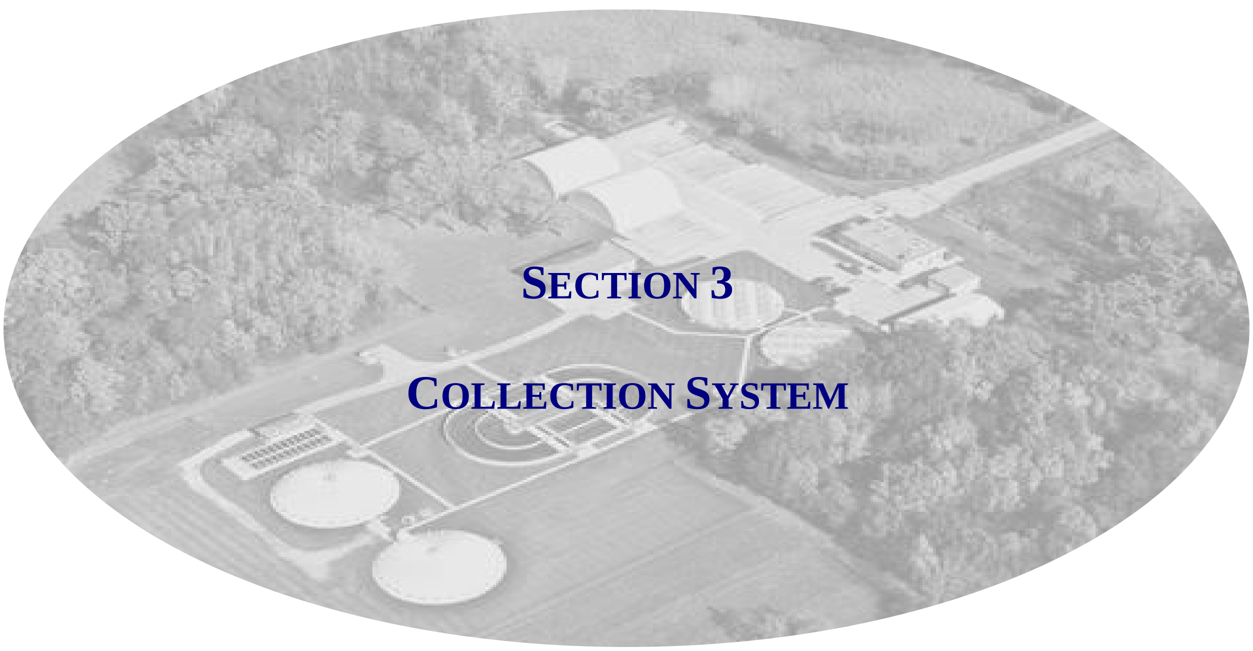
The Waterford Drainage Basin currently includes approximately 3,133 PE and is served by the Hale 1, Hale 2, and Waterford lift stations.

Much of the Waterford Expansion Area is comprised of completely developed subdivisions north of Route 176 or west of the Fox River. Extension of sewer service to these areas may become necessary as existing on-site septic systems age. The annexation of these subdivisions would allow the District to become contiguous to the Village of Holiday Hills. Service to the Waterford Expansion Area is viewed as a natural progression of the District.

It is projected that development of the remainder of the Waterford Basin would increase the residential customer base by 3,194 PE and the commercial base by 496 PE for a total additional contribution of 3,690 PE. This equates to a total PE at build-out of the basin of 6,823 PE.

Exhibit 2-16: Waterford Drainage Basin





SECTION 3

COLLECTION SYSTEM

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3. COLLECTION SYSTEM

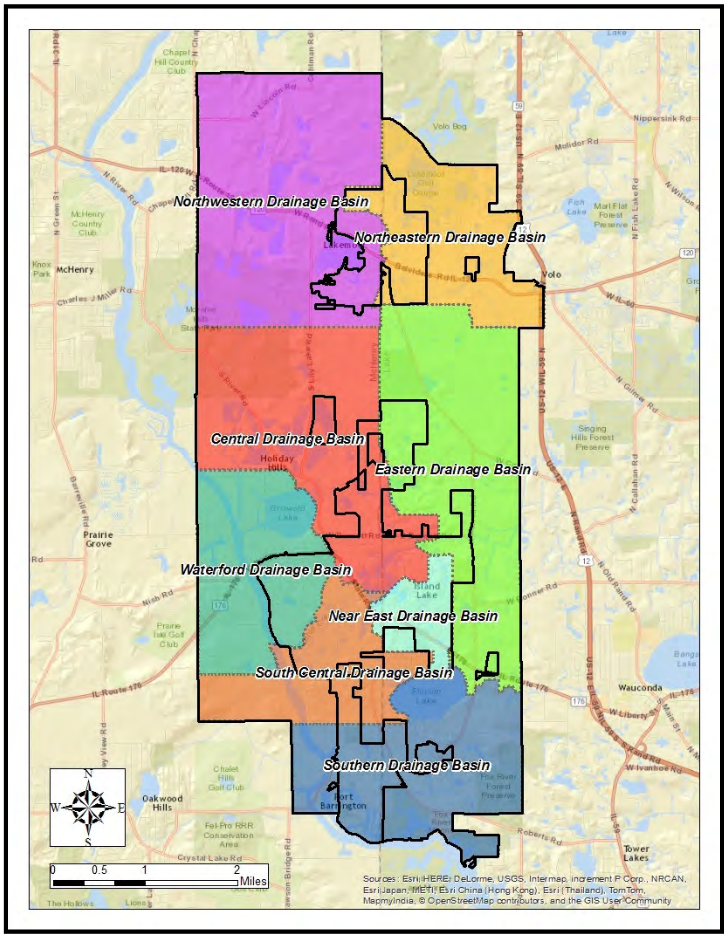
3.1 GENERAL BACKGROUND

The large amount of currently undeveloped land that could potentially be developed within the Northern Moraine FPA will require the extension of new interceptor sewers in the years to come. This section discusses current and projected flows within existing trunk interceptor sewers and identifies proposed interceptor sewers that would extend service to the outlying areas of the Northern Moraine FPA.

The sewers within the District's collection system are of varying age and condition. The older sections of the collection system are generally located near the treatment facility in the Village of Island Lake. These sewers are constructed of mostly ABS Truss pipe and PVC pipe.

Due to the relatively young age of the collection system, it does not include any vitrified clay sewer pipe. As a result, the District has not experienced the amount of root-intrusion or excessive levels of inflow/infiltration that are often associated with VCP pipe.

Exhibit 3-1: NMWRD Drainage Basins



NMWRD standard specifications require all new sanitary sewer construction to utilize long-lasting, corrosion resistant PVC pipe. Many of the newer subdivisions further from the treatment facility were constructed using PVC sewer pipe.

For facilities planning purposes, the District's collection system was divided into eight major wastewater drainage basins. These drainage basins are shown on Exhibit 3-1 and include the Northwestern, Northeastern, Eastern, Central, Near East, Waterford, South Central, and Southern Drainage Basins.

The District's 2013 Capital Improvements Plan estimates that the NMWRD collection system is comprised of approximately 57 miles of sanitary sewer, 15 miles of sanitary force main, 22 lift stations, and over 1500 manholes. The estimated value of the entire collection system in 2013 is summarized in Table 3-1.

Table 3-1: Collection System Asset Value

Asset	Quantity	Asset Value	Service Life	Depreciation
Gravity Sewer	57 miles	$ 30,600,000	100 years	$ 306,000
Force Main	15 miles	8,100,000	100 years	81,000
Manholes	1,500	7,600,000	100 years	76,000
Lift Stations	22	10,150,000		
Equipment & Controls	40%	4,060,000	20 years	203,000
Structures & Piping	60%	6,090,000	50 years	121,800
Total Value		**$ 56,450,000**		**$ 787,800**

Sufficient replacement funds should be established to support the rehabilitation and repair efforts necessary to ensure the continued future reliability of the aging infrastructure. Based on the depreciation rates listed in Table 3-1, the District should be reinvesting approximately $787,800 annually toward wastewater collection system rehabilitation and/or replacement.

3.2 POPULATION EQUIVALENT AND WASTEWATER FLOW PROJECTIONS

Table 3-2 summarizes the current population equivalent served, along with estimated (not measured) average and peak wastewater flow conditions, within each of the drainage basins.

Table 3-2: Existing Population Equivalents and Estimated Flows by Drainage Basin

Within NMWRD Corporate Boundary						
Drainage Basin	**Existing Residential PE**	**Existing Non-Residential PE**	**Total Existing PE**	**Design Average Flow (1) (MGD)**	**IEPA Peaking Factor**	**Peak Hourly Flow (MGD)**
Central	3,099	56	3,155	0.32	3.43	1.10
Eastern	38	31	69	0.01	4.29	0.04
Near East	2,011	224	2,235	0.22	3.55	0.78
Northeastern	1,765	147	1,912	0.19	3.60	0.68
Northwestern	1,480	138	1,618	0.16	3.66	0.59
South Central	311	67	378	0.04	4.04	0.16
Southern	1,188	9	1,197	0.12	3.75	0.45
Waterford	2,926	205	3,131	0.31	3.43	1.06
Total	**12,818**	**877**	**13,695**	**1.37 (2)**	**2.82**	**3.86**

(1) Design average flow for Basin at 100 gallons per capita per day (gcd).

(2) Measured average flow at treatment facility equaled 1.05 MGD in 2013 (approximately 77 gcd).

Table 3-3 summarizes projected future population equivalent, and wastewater flows, within each of the NMWRD drainage basins.

Table 3-3: Projected Population Equivalents and Flows by Drainage Basin

Within NMWRD Facility Planning Area						
Drainage Basin	Existing PE	Additional at Build-out of FPA PE	Total at Build-out of FPA PE	Future Design Average Flow [1] (MGD)	IEPA Peaking Factor	Future Peak Hourly Flow (MGD)
Central	3,155	2,270	5,425	0.54	3.21	1.73
Eastern	69	5,254	5,323	0.53	3.22	1.71
Near East	2,235	1,252	3,487	0.35	3.38	1.18
Northeastern	1,912	12,675	14,587	1.46	2.79	4.07
Northwestern	1,618	5,625	7,243	0.72	3.09	2.22
South Central	378	1,707	2,085	0.21	3.57	0.75
Southern	1,197	10,292	11,489	1.15	2.89	3.32
Waterford	3,131	3,690	6,821	0.68	3.12	2.12
Total	**13,695**	**42,765**	**56,460**	**5.64**	**2.22**	**12.52**

[1] Average design flow at 100 gcd.

3.3 TRUNK INTERCEPTOR SEWERS

The NMWRD wastewater treatment facility is located in the South Central Drainage Basin. All influent wastewater flow is conveyed to the treatment facility in either of five upstream interceptor sewers.

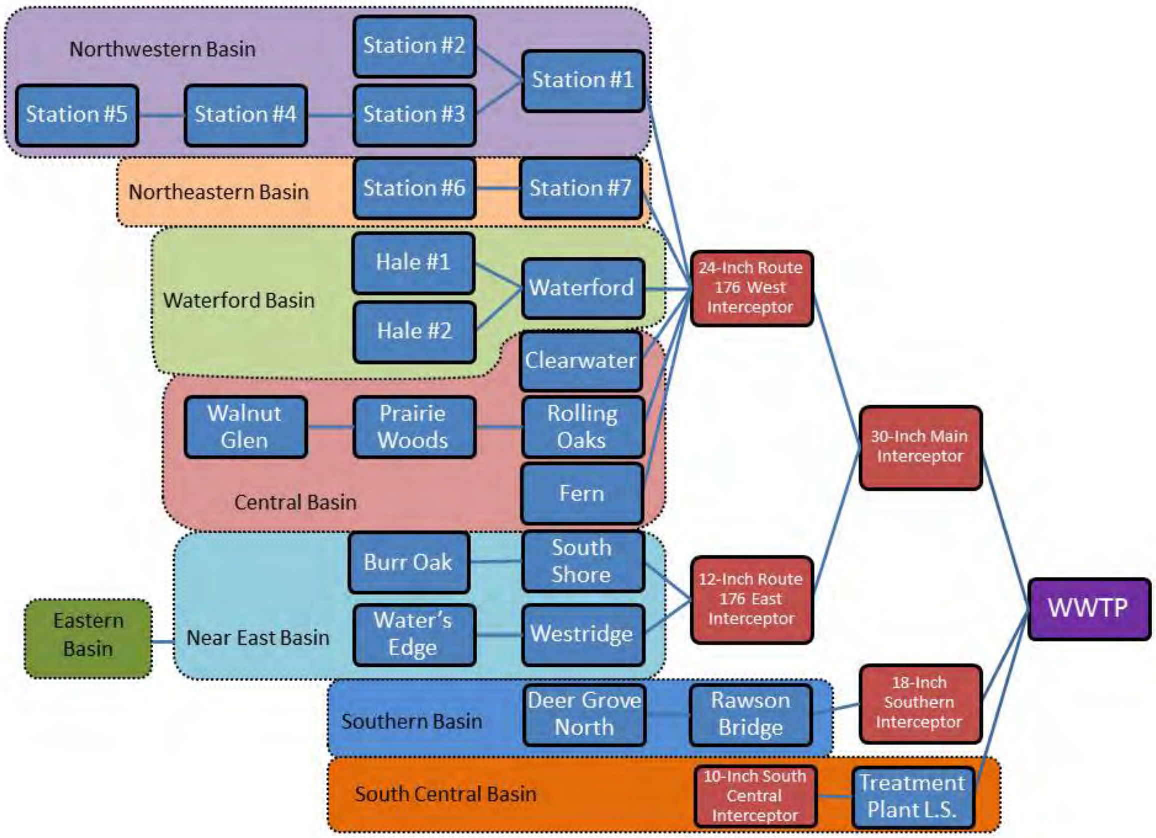
A 30-inch main interceptor sewer that extends north from the treatment facility conveys flow from six of the eight wastewater drainage basins. A 24-inch extension of the main interceptor extends northwest along Route 176 and conveys flows generated within the Central, Northeast, Northwest, and Waterford Drainage Basins. A 12-inch extension of the main interceptor extends southeasterly along Route 176 and conveys wastewater flows generated within the Eastern and Near East Drainage Basins.

Wastewater flow from the Southern Drainage Basin is pumped north and then conveyed by gravity to the treatment facility in the downstream 18-inch interceptor.

Wastewater generated in the South Central Drainage Basin is conveyed to the treatment facility site by gravity in a 10-inch interceptor sewer, and is then pumped into the treatment facility by the Treatment Plant Lift Station.

A flow diagram of the NMWRD collection system is shown on Exhibit 3-2.

Exhibit 3-2: Collection System Flow Diagram



3.3.1 Main Interceptor Sewer (30-inch)

The conveyance capacity of the 30-inch Main Interceptor sewer is measured against current and projected peak hourly flows in Table 3-4.

Table 3-4: Main Interceptor (30-inch) Capacities and Design Flows

Sewer Characteristics					Current Conditions		Projected Conditions @ FPA Full Build-out w/o Darrell Road Collection System		Projected Conditions @ FPA Full Build-out w/ Darrell Road Collection System	
Reach	Length (ft)	Slope (ft/lf)	Full Velocity (fps)	Full Capacity (MGD)	Current PHF (MGD)	Current Capacity Used	Projected PHF (MGD)	Projected Capacity Used	Projected PHF (MGD)	Projected Capacity Used
1	232	0.0022	3.89	12.34	3.48	28%	9.98	81%	6.38	52%
2	417	0.0034	4.86	15.40	3.48	23%	9.98	65%	6.38	41%
3	338	0.0030	4.56	14.46	3.48	24%	9.98	69%	6.38	44%
4	486	0.0012	2.94	9.34	3.48	37%	9.98	107%	6.38	68%
5	363	0.0011	2.78	8.82	3.48	39%	9.98	113%	6.38	72%
6	72	0.0014	3.12	9.91	3.48	35%	9.98	101%	6.38	64%

It can be seen that the Main Interceptor currently flows at approximately one-third capacity but will eventually become slightly overloaded in its upper reaches at full build-out. The Main Interceptor will be off-loaded in the future if the Darrell Road Collection System is constructed, because all of the flow from the Northeastern and Eastern Basins would be diverted into the proposed 42-inch interceptor sewer extending from the NMWRD treatment facility to the Water's Edge Lift Station, which would be abandoned at that time.

3.3.2 Route 176 West Interceptor (24-inch)

The conveyance capacity of the 24-inch Route 176 West Interceptor sewer is measured against current and projected peak hourly flows in Table 3-5.

Table 3-5: Route 176 West Interceptor (24-inch) Capacities and Design Flows

Sewer Characteristics					Current Conditions		Projected Conditions @ FPA Full Build-out w/o Darrell Road Collection System		Projected Conditions @ FPA Full Build-out w/ Darrell Road Collection System	
Reach	Length (ft)	Slope (ft/lf)	Full Velocity (fps)	Full Capacity (MGD)	Current PHF (MGD)	Current Capacity Used	Projected PHF (MGD)	Projected Capacity Used	Projected PHF (MGD)	Projected Capacity Used
1	171	0.0047	4.94	10.03	2.91	29%	8.26	82%	5.61	56%
2	151	0.0026	3.72	7.55	2.91	39%	8.26	109%	5.61	74%
3	669	0.0016	2.93	5.95	2.91	49%	8.26	139%	5.61	94%
4	298	0.0017	2.96	6.01	2.91	48%	8.26	138%	5.61	93%
5	282	0.0014	2.72	5.52	2.91	53%	8.26	150%	5.61	102%
6	68	0.0029	3.92	7.95	2.91	37%	8.26	104%	5.61	71%
7	105	0.0038	4.46	9.05	2.91	32%	8.26	91%	5.61	62%
8	110	0.0018	3.08	6.25	2.91	47%	8.26	132%	5.61	90%
9	285	0.0025	3.58	7.27	2.91	40%	8.26	114%	5.61	77%
10	77	0.0026	3.68	7.47	2.91	39%	8.26	111%	5.61	75%
11	438	0.0016	2.89	5.86	2.91	50%	8.26	141%	5.61	96%
12	456	0.0015	2.83	5.74	2.91	51%	8.26	144%	5.61	98%
13	501	0.0008	1.99	4.04	2.91	72%	8.26	205%	5.61	139%
14	220	0.0008	2.07	4.19	2.91	69%	8.26	197%	5.61	134%
15	160	0.0008	1.98	4.02	2.91	72%	8.26	206%	5.61	140%
16	394	0.0008	1.99	4.05	2.91	72%	8.26	204%	5.61	139%
17	218	0.0004	1.47	2.98	2.91	98%	8.26	277%	5.61	188%
18	57	0.0030	3.94	8.01	2.91	36%	8.26	103%	5.61	70%
19	102	0.0008	2.02	4.11	2.91	71%	8.26	201%	5.61	137%
20	231	0.0008	2.02	4.09	2.91	71%	8.26	202%	5.61	137%
21	230	0.0008	2.08	4.21	2.91	69%	8.26	196%	5.61	133%
22	207	0.0010	2.25	4.56	2.91	64%	8.26	181%	5.61	123%

The Route 176 West Interceptor is shown to have sufficient capacity for the current peak flows, with the exception of a 218-foot section of sewer installed at less than minimum slope (less than 2 fps). Under the projected peak flows at build-out the Route 176 West Interceptor will be overloaded throughout its length, especially its upper reaches.

Similar to the Main Interceptor, this sewer would benefit by construction of the Darrell Road Collection System. However, while implementation of the Darrell Road system will significantly reduce peak flows in the Route 176 West Interceptor, it will not completely alleviate the projected overloading of this sewer under the ultimate build-out conditions, especially in the uppermost 2,320 feet of pipe that is generally installed at minimum slope. This deficiency would be relieved in the future by either extending a parallel gravity sewer to the northwest, or by extending one or both of the Lakemoor force mains further to the southeast and connecting to the steeper sewer segments downstream.

3.3.3 Route 176 East Interceptor (12-inch)

The conveyance capacity of the 12-inch Route 176 East Interceptor sewer is measured against current and projected peak hourly flows in Table 3-6.

Table 3-6: Route 176 East Interceptor (12-inch) Capacities and Design Flows

Sewer Characteristics					Current Conditions		Projected Conditions @ FPA Full Build-out w/o Darrell Road Collection System		Projected Conditions @ FPA Full Build-out w/ Darrell Road Collection System	
Reach	Length (ft)	Slope (ft/lf)	Full Velocity (fps)	Full Capacity (MGD)	Current PHF (MGD)	Current Capacity Used	Projected PHF (MGD)	Projected Capacity Used	Projected PHF (MGD)	Projected Capacity Used
1	272	0.0165	5.86	2.97	0.82	28%	2.65	89%	1.20	40%
2	302	0.0079	4.06	2.06	0.82	40%	2.65	129%	1.20	58%
3	362	0.0094	4.41	2.24	0.82	37%	2.65	118%	1.20	54%
4	190	0.0084	4.18	2.12	0.82	39%	2.65	125%	1.20	57%
5	317	0.0148	5.54	2.81	0.82	29%	2.65	94%	1.20	43%
6	213	0.0042	2.96	1.50	0.82	55%	2.65	176%	1.20	80%
7	334	0.0024	2.23	1.13	0.82	73%	2.65	234%	1.20	106%
8	350	0.0020	2.04	1.03	0.82	79%	2.65	256%	1.20	116%
9	346	0.0026	2.32	1.18	0.82	70%	2.65	225%	1.20	102%

The Route 176 East Interceptor currently operates well below it full flow capacity, but is also projected to be overloaded under the projected peak hour flows as development occurs within the Eastern and Near Eastern Drainage Basins. It is proposed that this interceptor be off-loaded by construction of the lower reaches of the Darrell Road Collection System. A slight overload is still projected in the uppermost reaches of this sewer, although this may be offset by removal of the Water's Edge Lift Station from this drainage basin.

3.3.4 Southern Interceptor (18-inch)

The conveyance capacity of the 18-inch Southern Interceptor sewer is measured against current and projected peak hourly flows in Table 3-7.

Table 3-7: Southern Interceptor (18-inch) Capacities and Design Flows

Reach	Length (ft)	Slope (ft/lf)	Full Flow Velocity (fps)	Full Flow Capacity (MGD)	Current PHF (MGD)	Current Capacity Used	Projected PHF (MGD)	Projected Capacity Used
1	385	0.0038	3.66	4.18	0.45	11%	3.33	80%
2	394	0.0013	2.12	2.43	0.45	19%	3.33	137%
3	143	0.0204	8.52	9.73	0.45	5%	3.33	34%
4	150	0.0432	12.40	14.15	0.45	3%	3.33	24%
5	263	0.0589	14.48	16.53	0.45	3%	3.33	20%
6	19	0.1053	19.35	22.09	0.45	2%	3.33	15%
7	129	0.0581	14.38	16.42	0.45	3%	3.33	20%

The Southern Interceptor conveys flow pumped from the Southern Drainage Basin. This interceptor has more than sufficient capacity to convey both the current and future projected peak flows, with the exception of one reach of sewer near the NMWRD treatment facility. Wastewater flows in this sewer should be monitored over time but the District should not anticipate any capacity related issues with this sewer for the foreseeable future.

3.3.5 South Central Interceptor (10-inch)

The conveyance capacity of the 10-inch South Central Interceptor sewer is measured against current and projected peak hourly flows in Table 3-8.

Table 3-8: South Central Interceptor (10-inch) Capacities and Design Flows

Reach	Length (ft)	Slope (ft/lf)	Full Flow Velocity (fps)	Full Flow Capacity (MGD)	Current PHF (MGD)	Current Capacity Used	Projected PHF (MGD)	Projected Capacity Used
1	323	0.0037	2.46	0.87	0.15	17%	0.75	87%
2	300	0.0040	2.55	0.90	0.15	17%	0.75	83%
3	277	0.0043	2.65	0.94	0.15	16%	0.75	80%
4	419	0.0062	3.18	1.12	0.15	13%	0.75	67%
5	240	0.0040	2.55	0.90	0.15	17%	0.75	83%
6	246	0.0040	2.54	0.90	0.15	17%	0.75	84%

The 10-inch South Central Interceptor provides more than sufficient capacity to convey both the current and projected future peak hour flows. The District should not anticipate any capacity related issues with this sewer for the foreseeable future.

3.4 WASTEWATER DRAINAGE BASINS

3.4.1 Central Drainage Basin

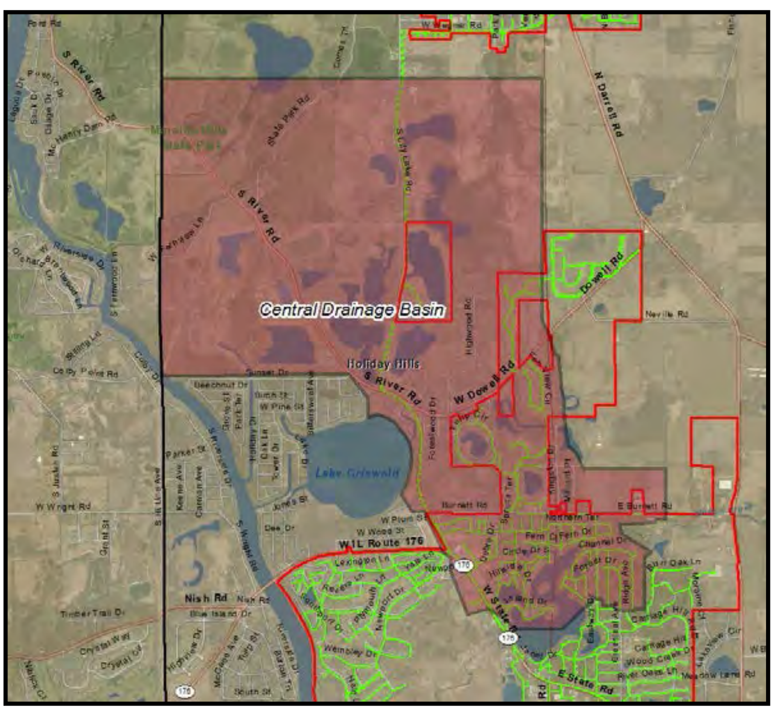
The extent of the Central Drainage Basin and the existing collection system are shown on Exhibit 3-3. The Central Basin encompasses a total of 2,806 acres, of which 660 acres are currently annexed to the Northern Moraine WRD. The Central Basin currently serves approximately 3,155 PE. Complete build-out of this basin could contribute an additional 2,270 PE for an ultimate total of 5,425 PE.

Wastewater generated within the Central Drainage Basin is conveyed to the treatment facility through a 1,930 lineal foot, 30-inch interceptor sewer that runs directly north from the treatment facility. A 24-inch interceptor extends 5,375 lineal feet northwest along Route 176 and terminates at River Road. The Clearwater, Walnut Glen, Prairie Woods, Rolling Oaks, and Fern lift stations are all located within the Central Drainage Basin.

The 24-inch Route 176 West Interceptor also receives pumped flow from the Northeastern and Northwestern Drainage Basins in Lakemoor. In addition it receives all the flow from the Waterford Drainage Basin via the Waterford Lift Station.

Future development in the Central Basin would occur in the northern extents of the basin. It was previously planned to serve these areas through an extension of the 24-inch interceptor to the north along River Road and a proposed regional lift station located directly north of Griswold Lake. The construction of the Darrell Road Collection System described later will free capacity in the existing parallel 8-inch and 12-inch force mains on Lily Lake Road. Small localized lift station could pump into these force mains in lieu of extending a gravity sewer.

Exhibit 3-3: Central Drainage Basin



3.4.2 Eastern Drainage Basin

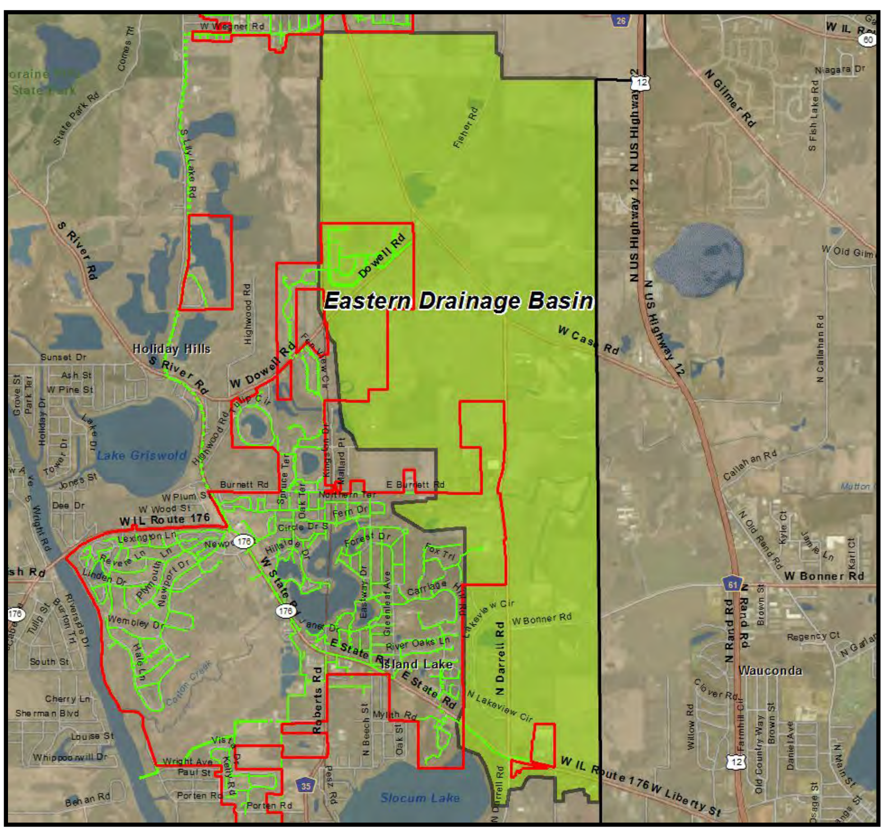
The Eastern Drainage Basin was delineated during construction of the first phase of the Darrell Road Interceptor Sewer from Water's Edge Drive to Route 176. This drainage basin includes the entire FPA east of Darrell Road, north of Route 176, and south of Lakemoor. The extents of the Eastern Drainage Basin and the existing collection system are shown on Exhibit 3-4. The Eastern Basin encompasses a total of 3,002 acres, of which 428 acres are currently annexed to the Northern Moraine WRD.

The District currently only serves 24 acres of development, contributing a total of 69 PE. Full build-out of this basin could contribute an additional 5,254 PE for an ultimate total of 5,323 PE.

Wastewater generated in this basin is currently conveyed to the Burr Oak and Water's Edge Lift Stations located in the Near East Drainage Basin (see Section 4).

Build-out of the Eastern Drainage Basin will be served primarily by the proposed Darrell Road Collection System. The conceptual design of the Darrell Road Collection System is geared towards minimizing or eliminating the need to construct several small lift stations throughout the system. A regional lift station is proposed to be located north of Mutton Creek at Darrell Road (see Section 4). The Mutton Creek force main would discharge to the Darrell Road Interceptor Sewer south of Bonner Road. The phased implementation of the Darrell Road Collection System is discussed in detail later in this section.

Exhibit 3-4: Eastern Drainage Basin



3.4.3 Near East Drainage Basin

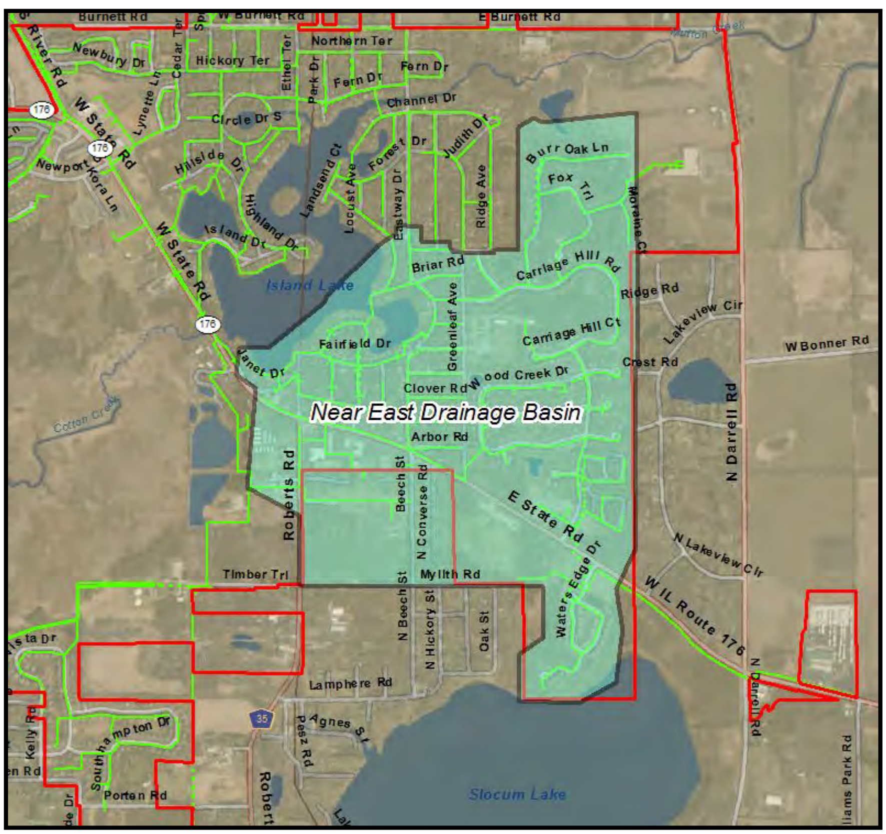
The Near East Drainage Basin is located directly to the northeast of the NMWRD treatment facility. The extents of the Near East Drainage Basin and the existing collection system are shown on Exhibit 3-5. The Near East Basin encompasses a total of 466 acres, of which 410 acres are currently annexed to the Northern Moraine WRD. The basin currently includes approximately 100 acres of development contributing about 2,235 PE. Complete build-out of the basin would result in the addition of about 1,252 PE for an ultimate total of 3,487 PE.

This basin is served by a 12-inch gravity interceptor sewer that extends from its connection to the Main Interceptor north to and then southeast along Route 176. It was shown in Table 3-6 that this interceptor has sufficient capacity for the current flows, but is running at 70 to 80 percent capacity in its upper reaches, and will become overloaded as growth occurs. This interceptor will reach full capacity with the connection of an additional 600 PE.

The projected overload conditions in the Route 176 East Interceptor are proposed to be relieved by construction of the lower reaches of the Darrell Road Collection System, specifically the Treatment Plant Interceptor as discussed later in this Section.

The Westridge, Water's Edge, Burr Oak and South Shore Lift Stations serve the Near East Drainage Basin (see Section 4).

Exhibit 3-5: Near East Drainage Basin



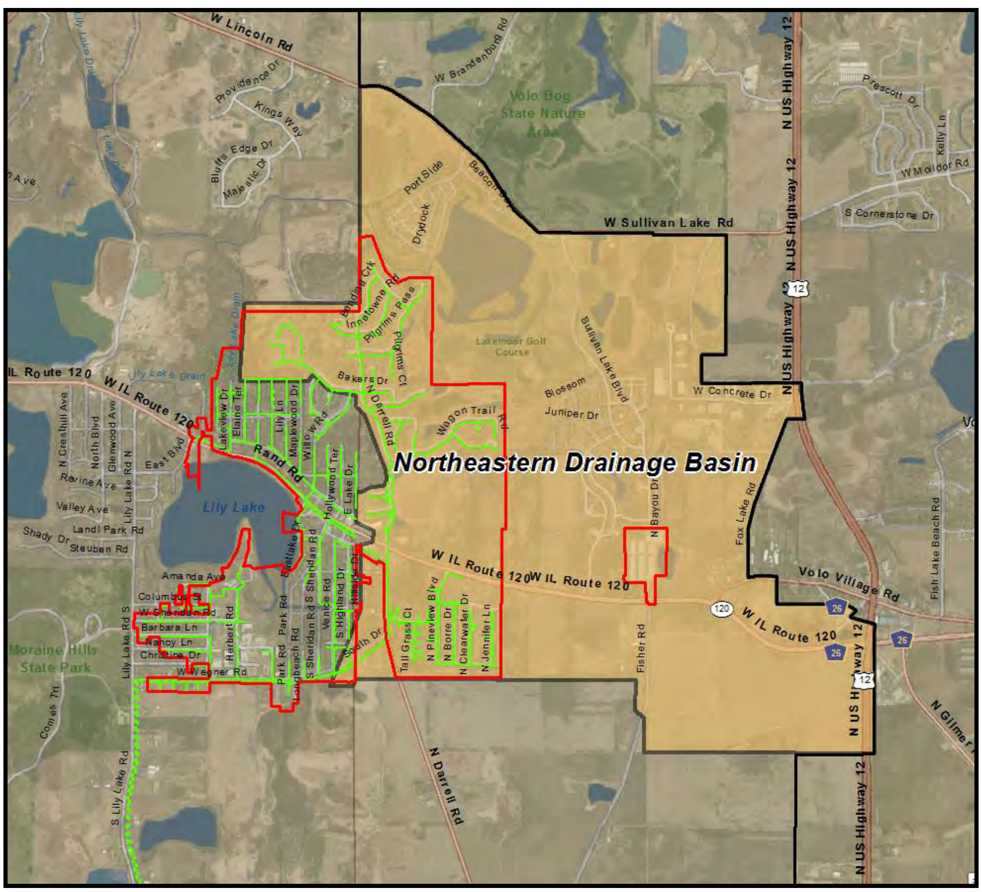
3.4.4 Northeastern Drainage Basin

The Northeastern Drainage Basin includes the eastern half of the Village of Lakemoor. The extents of the Northeastern Drainage Basin and the existing collection system are shown on Exhibit 3-6. The Northeastern Basin encompasses a total of 1,676 acres, of which 423 acres are currently annexed to the Northern Moraine WRD. It currently includes 1,912 PE with the potential to connect an additional 12,675 PE, for an ultimate total of 14,587 PE. The existing collection system includes nearly 7 miles of sanitary sewers and two lift stations, Lakemoor Lift Stations 6 and 7 (see Section 4).

The entire Northeastern Drainage Basin is currently tributary to Lakemoor Lift Station 7. Wastewater is pumped through a 12-inch diameter 19,000 lineal foot force main to the 24-inch Route 176 West Interceptor. Currently, the amount of development that can occur in this drainage basin is governed by the capacity of Lakemoor Lift Station 7. Based on future PE projections, neither this lift station, nor the downstream Route 176 West Interceptor Sewer will be able to serve complete build-out of this drainage basin.

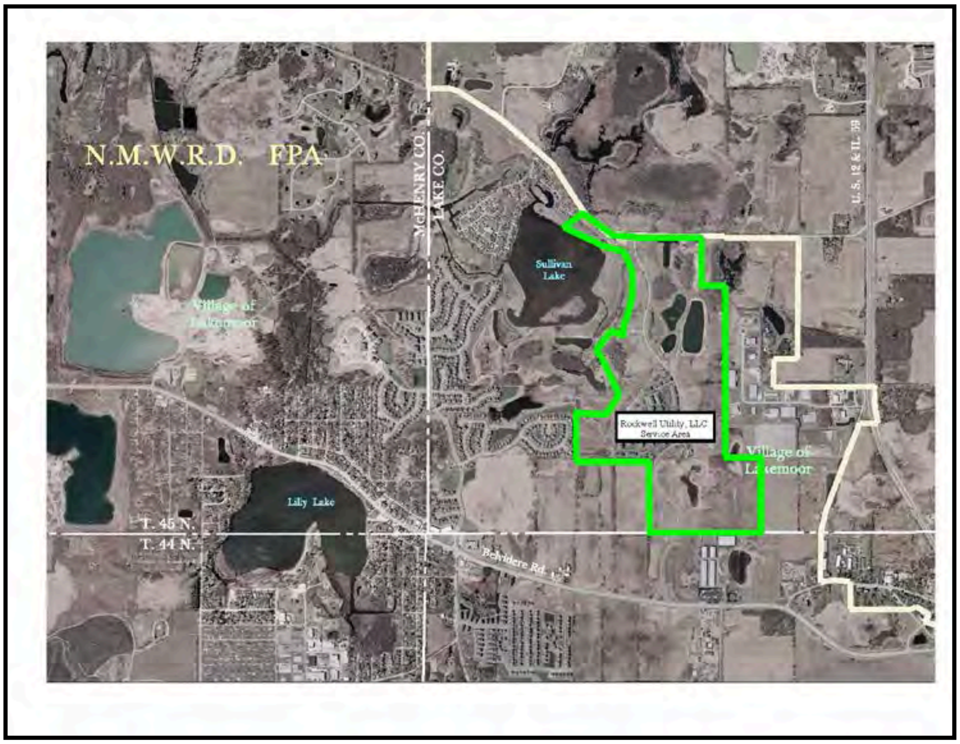
The extension of increased conveyance capacity to the Northeast Basin is proposed to include adding a third pump to Lakemoor Lift Station 7 and rerouting the force main from Lift Station 7 eastward to the uppermost reach of the proposed Darrell Road Interceptor.

Exhibit 3-6: Northeastern Drainage Basin



A key factor in determining when the current capacity of Lakemoor Lift Station 7 will be exceeded is whether or not the District chooses to connect the Rockwell Place Development to the NMWRD collection system. The Rockwell Place Development is shown on Exhibit 3-7.

Exhibit 3-7: Rockwell Place Development Boundary



Rockwell Place consists of four neighborhoods in the Northeastern Basin located north of Route 120 and south of Sullivan Lake Road. Wastewater is currently treated at Rockwell Utilities private treatment system consisting of a two cell aerated lagoon, a winter storage lagoon, disinfection by chlorination and disposal by irrigation. The irrigation system is used for land application of wastewater effluent on approximately 87 acres of land. The system is designed to serve 3,210 PE contributing an average flow of 321,000 gpd. Based on IEPA peaking factors, the peak hourly flow is estimated at approximately 1.44 MGD.

The Rockwell Utilities wastewater system is privately owned. However, it is located within the Northern Moraine FPA and the District is the Designated Management Agency (DMA) for this previously developed area. Should the private treatment system fail, the District may be called upon to provide wastewater service to these developments. In consideration of this, the connection of Rockwell Place was included in the population and flow projections for the Northeastern Basin so that the District is aware of the impacts should it decide to connect this development in the future.

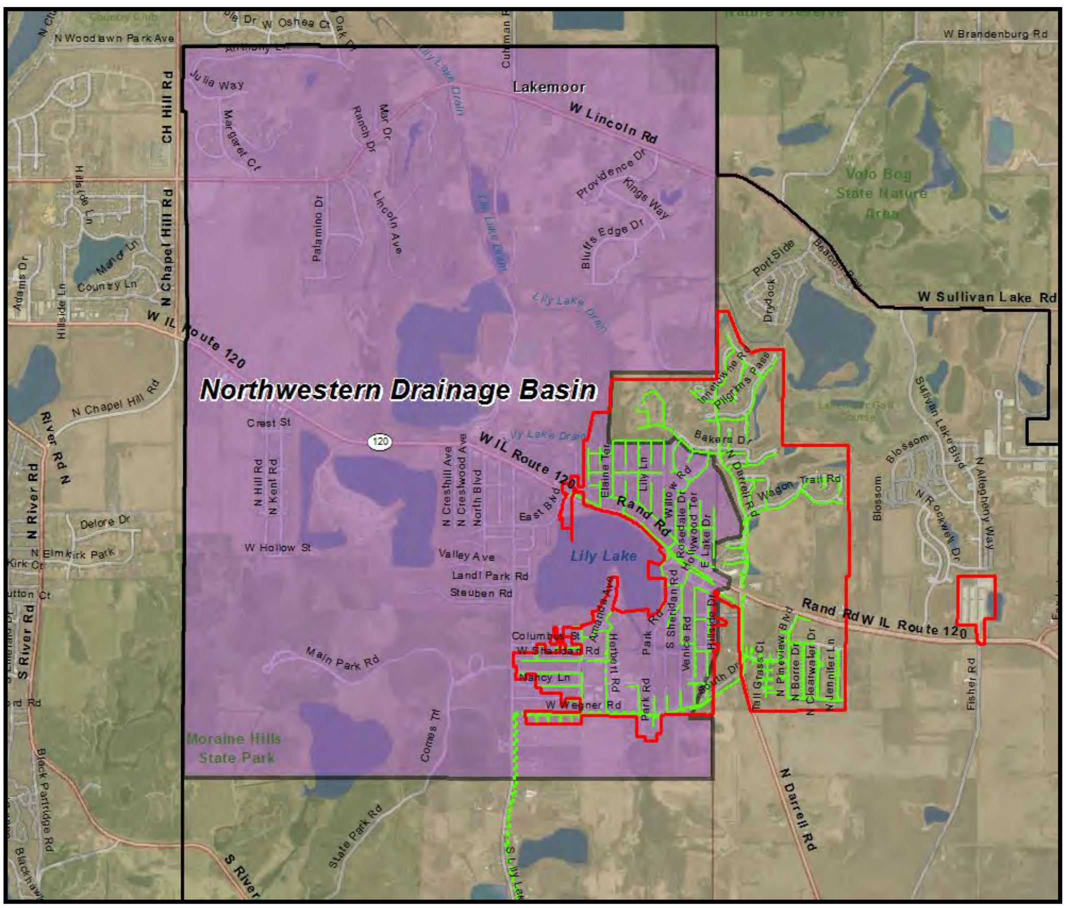
The construction of the Darrell Road Collection System would allow service to be provided to this development through either the proposed Darrell Road and/or Fisher Road Interceptor sewers that are discussed later in this Section.

3.4.5 Northwestern Drainage Basin

The Northwestern Drainage Basin drains the entire west side of the Village of Lakemoor. The extents of the Northwestern Drainage Basin and the existing collection system are shown on Exhibit 3-8. The Northwestern Basin encompasses a total of 3,452 acres, of which 323 acres are currently annexed to the Northern Moraine WRD. The Northwestern Basin currently serves roughly 1,618 PE, all within the Village of Lakemoor. Complete build-out of this basin will contribute an additional 5,625 PE for an ultimate total of 7,243 PE.

The Northwestern Basin collection includes nearly 4 miles of sanitary sewers, four small lift stations (Lakemoor Lift Stations 2, 3, 4, and 5), and one major lift station (Lakemoor Lift Station 1). The entire drainage basin is tributary to Lakemoor Lift Station 1 (located near the intersection of Wegner Road and Fritzsche Road) which pumps wastewater through an 8-inch diameter, 16,500 lineal foot long force main to the 24-inch Route 176 West Interceptor. A more in depth study of necessary extensions of the collection system to serve the Northwestern Expansion Area was conducted in 2012. The recommendations of that study are summarized in the following pages.

Exhibit 3- 8: Northwestern Drainage Basin



To further study the Northwestern expansion areas, the drainage basin was divided into four Service Areas (A through D) as shown on Exhibit 3-9.

The current and proposed land use within the Northwestern Expansion Area is shown on Exhibit 3-10 and includes a mixture of low density residential development and open space. Current and proposed low-density neighborhood residential areas were assumed will be serviced by private on-site disposal systems. The higher-density land uses were assumed to be tributary to the wastewater collection system.

Population projections and wastewater flows in each service area are listed in Table 3-9.

Exhibit 3-9: NW Expansion Area Service Areas

Exhibit 3-10: NW Expansion Area Land Use

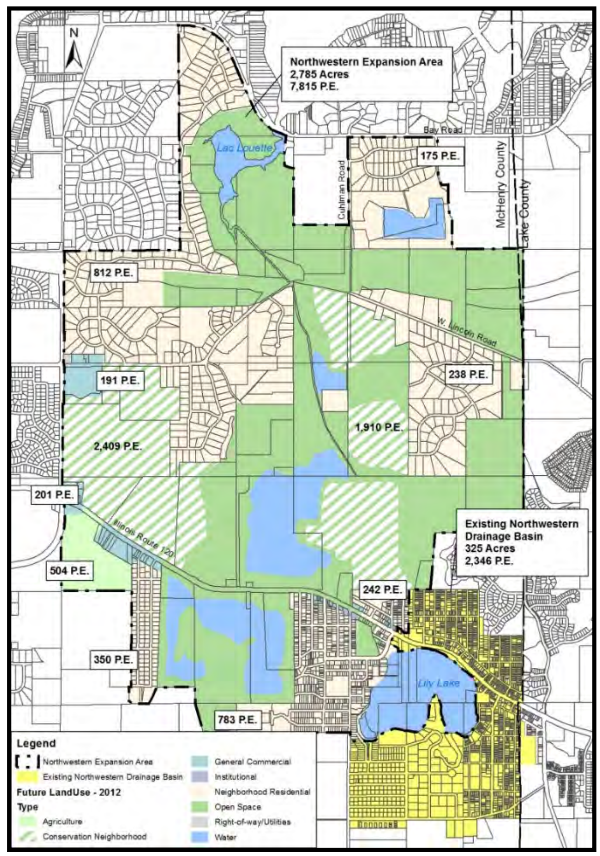


Table 3-9: Northwestern Expansion Area - Projected PE and Wastewater Flows

Service Area	Projected In-Basin			Projected Upstream Basins			Projected Totals		
	Population Equivalent	Daily Average Flow (MGD)	Peak Hourly Flow (MGD)	Population Equivalent	Daily Average Flow (MGD)	Peak Hourly Flow (MGD)	Population Equivalent	Daily Average Flow (MGD	Peak Hourly Flow (MGD)
A	746	0.07	0.27	4,938	0.49	1.59	5,634	0.56	1.79
B	2,820	0.28	0.96	n/a	n/a	n/a	2,820	0.28	0.96
C-1	485	0.05	0.20	n/a	n/a	n/a	485	0.05	0.20
C-2	443	0.04	0.16	485	0.05	0.20	928	0.09	0.34
C-3	898	0.09	0.34	928	0.09	0.34	1,826	0.18	0.65
D	242	0.02	0.08	n/a	n/a	n/a	242	0.02	0.08

During previous facility planning efforts it was anticipated that the Central and Northwestern Drainage Basins would be served by a new interceptor on Lily Lake Road that would flow to a new lift station on River Road. It was expected that these facilities would be funded by proposed development. However, at this time there are no proposed developments in the Central Drainage Basin to facilitate construction of the proposed Lily Lake Interceptor.

In lieu of constructing the Lily Lake Interceptor and River Road Lift Station, existing capacity will be made available by off-loading the existing parallel 8-inch and 12-inch force mains along Lily Lake and River Roads through construction of the Darrell Road Collection System.

Wastewater flows from throughout the Northwestern Expansion Area would be collected to proposed Lift Station A. The lift station would pump through a 12-inch force main south along Lily Lake Road to connect to one or both of the existing parallel 8-inch and 12-inch force mains that extend south along Lily Lake Road and River Road to the 24-inch Route 176 West Interceptor as shown on Exhibit 3-11. A 21-inch Lily Lake Road Interceptor would extend north from Lift Station A to Route 120 (see Exhibit 3-12).

The existing Northwestern Basin collection system, including Lift Station 1, has more than sufficient capacity to convey the current flows from the connected areas. However, growth of the Northwestern Basin will not result in significantly increased flows within the existing sewers or at Lift

Exhibit 3-11: Lift Station A Force Main

Station 1, only that resulting from fill-in of the already near fully developed connected area. Growth will occur in areas that will not flow to the existing collection system. Wastewater from all the newly developed areas will flow to Lift Station A, from which it can be pumped directly into the existing force main(s) and not to Lift Station 1.

The collection system improvements proposed to extend the collection system through the Northwestern Expansion Area are shown on Exhibit 3-12.

Exhibit 3-12: NW Expansion Area Collection System

A proposed 15-inch Route 120 West Interceptor would extend northwesterly along Route 120 and accept flow pumped through an 8-inch force main originating at the proposed Service Area B Lift Station. A separate 15-inch and 8-inch Route 120 West Interceptor would extend further northwest along Route 120 to convey flow collected from the north.

A proposed 8-inch Route 120 East Interceptor would extend southeasterly from Lily Lake Road along Route 120 and convey flow from Service Area D.

Wastewater generated in Service Area C-1 would be pumped at the proposed Service Area C-1 Lift Station to the Service Area C-2 Lift Station, which would also pump flows collected from within Service Areas C-2 and C-3 to the 21-inch Lily Lake Road Interceptor.

Projected population equivalents, wastewater flows, and interceptor sewer sizing within the Northwestern Expansion Area are summarized in Table 3-10.

Table 3-10: NW Expansion Area - Interceptor Sewer Sizing and Flows

Proposed Interceptor Sewer	Population Equivalent PE	Daily Average Flow (MGD)	Peak Hourly Flow (MGD)	Interceptor Diameter (inch)	Interceptor Capacity (MGD)	Capacity Utilized
Lily Lake Road Interceptor	5,634	0.56	1.79	21	3.11	58 %
Route 120 West Interceptor	2,820	0.28	0.96	15	1.58	61 %
Route 120 East Interceptor	242	0.02	0.08	8	0.45	18 %

Projected population equivalents, wastewater flows, and lift station and force main sizing within the Northwestern Expansion Area are summarized in Table 3-11.

Table 3-11: NW Expansion Area - Lift Station and Force Main Sizing and Flows

Proposed Force Main	Population Equivalent PE	Daily Average Flow (MGD)	Peak Hourly Flow (MGD)	Lift Station Firm Capacity (gpm)	Force Main Diameter (inch)	Force Main Velocity (fps)
Lift Station A Force Main	5,634	0.56	1.79	1,245	12	3.53
Lift Station B Force Main	2,820	0.28	0.96	670	8	4.40
Lift Station C-1 Force Main	485	0.05	0.20	275 (2)	6 (1)	2.00
Lift Station C-2 Force Main	1,826	0.18	0.65	450	6	5.11
(1) Minimum 6-inch diameter force main recommended to minimize plugging.						
(2) Firm capacity set to maintain minimum velocity in 6-inch force main.						

As discussed earlier, the projected flows from additional connections to the Northwestern collection system will be conveyed through the existing parallel 8-inch and 12-inch force mains along Lily Lake and River Roads to the 24-inch Route 176 West Interceptor. Resultant velocities in the existing force mains by any combination of use are summarized in Table 3-12.

Table 3-12: Lakemoor Lift Stations 1 and 7 – Parallel Force Main Flows

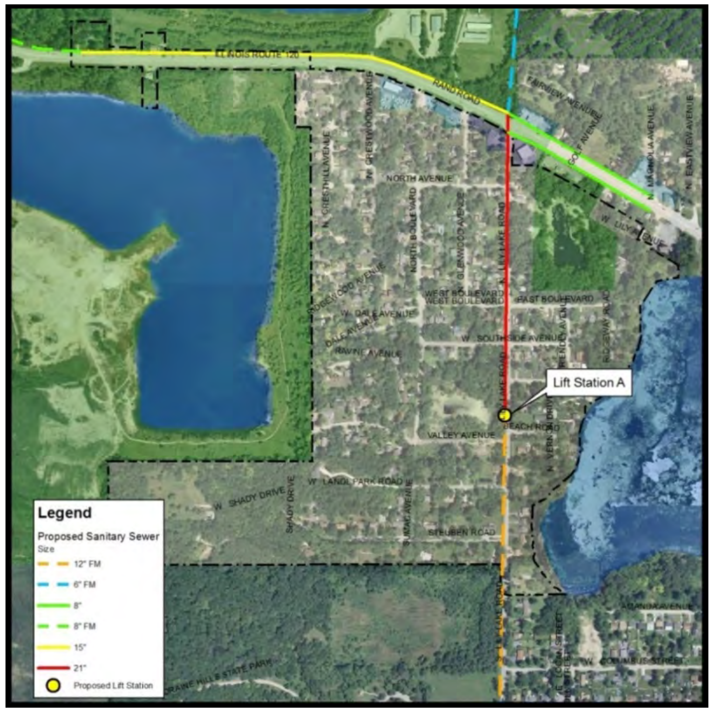
Proposed Force Main	Population Equivalent PE	Daily Average Flow (MGD)	Peak Hourly Flow (MGD)	Lift Station Firm Capacity (gpm)	Force Main Diameter (inch)	Force Main Velocity (fps)
Lakemoor Lift Station 1 Force Main	7,243	0.72	2.22	1,540	8	9.85
Lakemoor Lift Station 7 Force Main	7,243	0.72	2.22	1,540	12	4.38
Combined Parallel Force Mains	7,243	0.72	2.22	1,540	8 & 12	3.03

It is proposed that Phase 1 of the sewer expansion within the NW Expansion Area be limited to 3,550 PE, which corresponds with the remaining capacity available within the downstream Route 176 West Interceptor, or until the Darrell Road Collection System is constructed in its entirety.

Service Area A

The 21-inch Lily Lake Interceptor would convey flow from Service Area A, as well as flows from all other Service Areas, to Lift Station A on Lily Lake Road and Beach Avenue. This lift station would be located at the low point of this Service Area (ground elevation of 750 feet) and would pump all flows through a 3,000 foot long, 12-inch force main to the existing 8-inch Lift Station 1 force main at Lily Lake Road and Wegner Road. The northern portion of Service Area A would be served by the 2,900 foot long 15-inch Route 120 West Interceptor, that would also convey wastewater from Service Area B.

Exhibit 3-13: NW Expansion Area - Service Area A



Service Area B

The extension of the Route 120 West Interceptor through Service Area B (approximately 1,300 feet of 8-inch sewer and 1,400 feet of 15-inch sewer) will serve Service Area B. All wastewater from Service Area B will be conveyed to Lift Station B, similarly located at the low elevation in the Service Area (ground elevation of 762 feet), on Illinois Route 120 near Kent Road. Flow would be pumped through a 1,600 foot long 8-inch force main to the downstream 15-inch reach of the Route 120 West Interceptor.

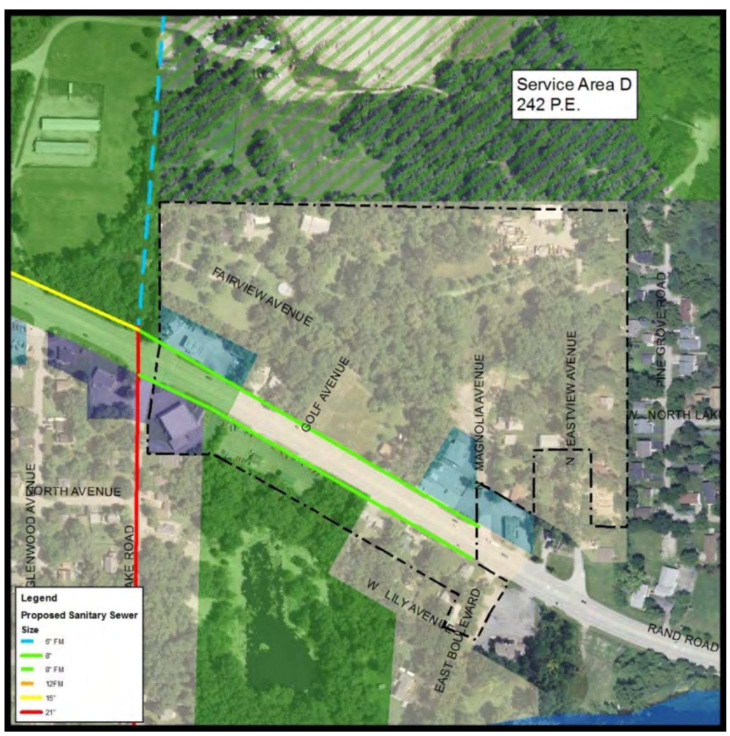
Exhibit 3-14: NW Expansion Area Service Area B

Exhibit 3-15: NW Expansion Area - Service Area C

Service Area C

The northern portion of Service Area C will be served by a small lift station (Lift Station C-1) and a 6-inch diameter, 2,600 foot long force main to Lift Station C-2. Wastewater generated in Service Areas C-2 and C-3 would be collected to Lift Station C-2 and pumped along with the Area C-1 flow through a 3,400 foot long 6-inch force main the 21-inch Lily Lake Road Interceptor at Route 120.

Exhibit 3-16: NW Expansion Area - Service Area D



Service Area D

Collection system improvements in Service Area D would be limited to installing parallel 8-inch sewers on either side of Route 120, each 1,100 feet in length to convey flows from Neighborhood Residential and Commercial properties to the 21-inch Lily Lake Interceptor at Illinois Route 120 and Lily Lake Road.

Probable capital costs for the construction of the Northwestern Expansion Area improvements to the collection systems described above are summarized in Table 3-13.

Table 3-13: NW Expansion Area - Probable Capital Costs

NW Expansion Area Collection System				
SUMMARY				
CONSTRUCTION SUBTOTAL				10,008,200
GENERAL CONDITIONS				1,251,025
CONTINGENCY @ 15%				1,501,230
CONSTRUCTION TOTAL				12,760,455
DESIGN ENGINEERING @ 7.5%				957,034
CONSTRUCTION ENGINEERING @ 7.5%				957,034
TOTAL CAPITAL COSTS - PHASE 1				**14,674,523**
GENERAL CONDITIONS				
Bond & Insurance @ 2.5%				250,205
Overhead and Profit @ 10%				1,000,820
TOTAL GENERAL CONDITIONS				**1,251,025**
LIFT STATIONS				
Lift Station A				1,000,000
Lift Station B				850,000
Lift Station C				350,000
Lift Station D				750,000
TOTAL LIFT STATIONS				**2,950,000**
INTERCEPTOR SEWER AND FORCE MAIN				
Description	**Quantity**	**Unit**	**Unit Price**	**Total**
8-inch Sanitary Sewer	3,500	LF	$120	420,000
15-inch Sanitary Sewer	4,300	LF	$180	774,000
21-inch Sanitary Sewer	2,000	LF	$210	420,000
Sanitary Manholes	34	EA	$6,000	204,000
6-inch Force Main	6,000	LF	$75	450,000
8-inch Force Main	1,600	LF	$100	160,000
12-inch Force Main	3,000	LF	$120	360,000
Trench Backfill	22,215	CY	$40	888,600
Pavement Removal and Replacement	30,400	SY	$75	2,280,000
Connection to Existing Force Main	1	EA	$5,000	5,000
Bore and Jack 42-inch Casing for 24-inch Sewer	100	LF	$750	75,000
Sewer Undercrossing of Route 120	100	LF	$750	75,000
Erosion Control Fence	20,400	LF	$4	81,600
Seeding, Class II w/ Excelsior Blanket	10	Acres	$3,500	35,000
Stream Crossing	1	EA	$30,000	30,000
Dewatering	4	EA	$150,000	600,000
Mobilization	4	EA	$25,000	100,000
Traffic Control and Protection	4	EA	$25,000	100,000
TOTAL INTERCEPTOR & FORCE MAIN				**7,058,200**

3.4.6 South Central Drainage Basin

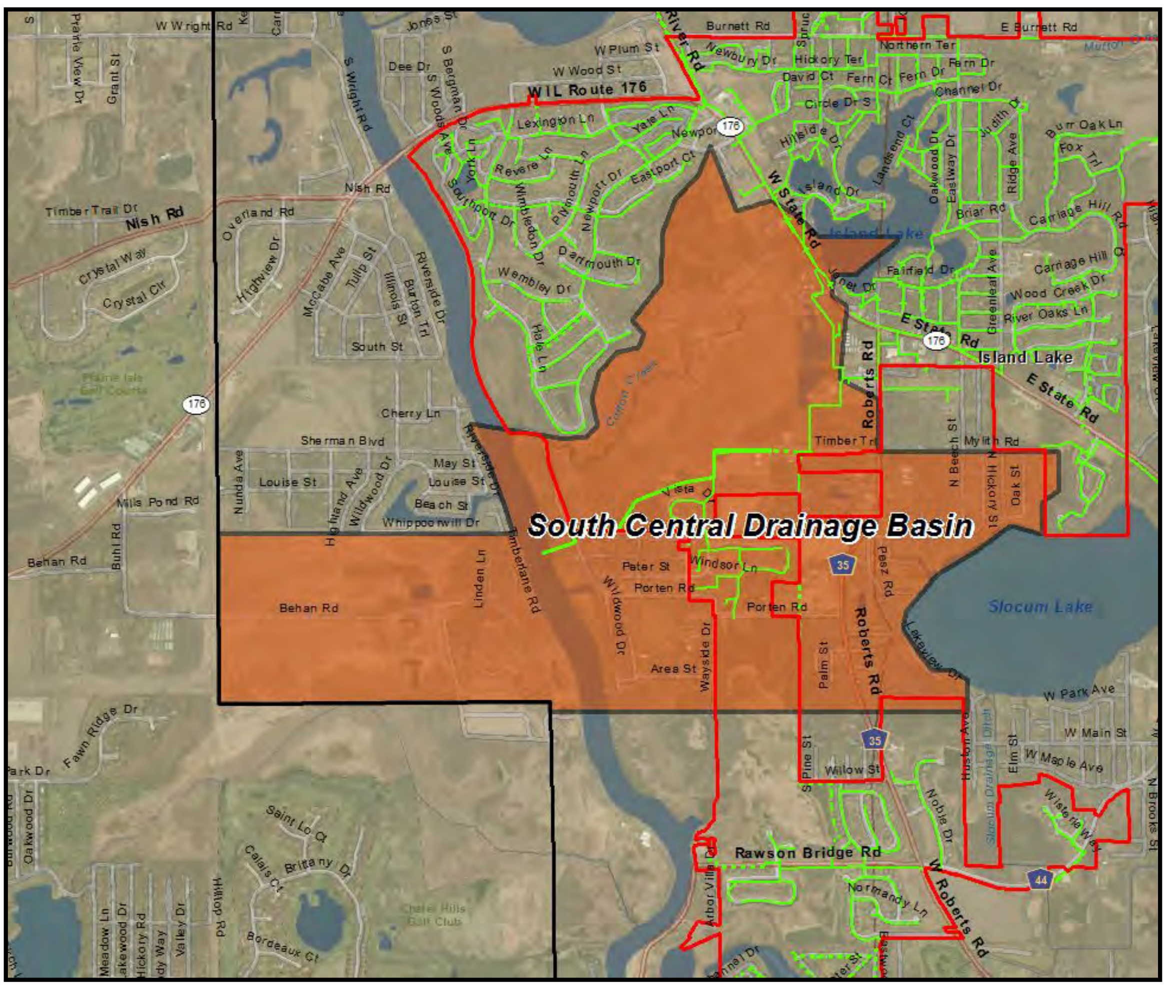
The extents of the South Central Drainage Basin and the existing collection system are shown on Exhibit 3-17. The South Central Basin encompasses a total of 1,335 acres, of which 547 acres are currently annexed to the Northern Moraine WRD.

The South Central Drainage Basin currently includes approximately 378 PE. Current development within this basin is located to the south of the NMWRD treatment facility. Development of this basin could contribute an additional 1,707 PE at build-out for an ultimate total of 2,085 PE.

This basin is tributary to a 10-inch gravity interceptor sewer that flows north 4,300 lineal feet through the center of the basin and directly to the treatment facility. This basin serves a portion of the Village of Island Lake and is bounded to the west by the Fox River and to the East by Slocum Lake.

The Treatment Plant Lift Station serves all development within the South Central Basin. This lift station is discussed further in Section 4.

Exhibit 3-17: South Central Drainage Basin



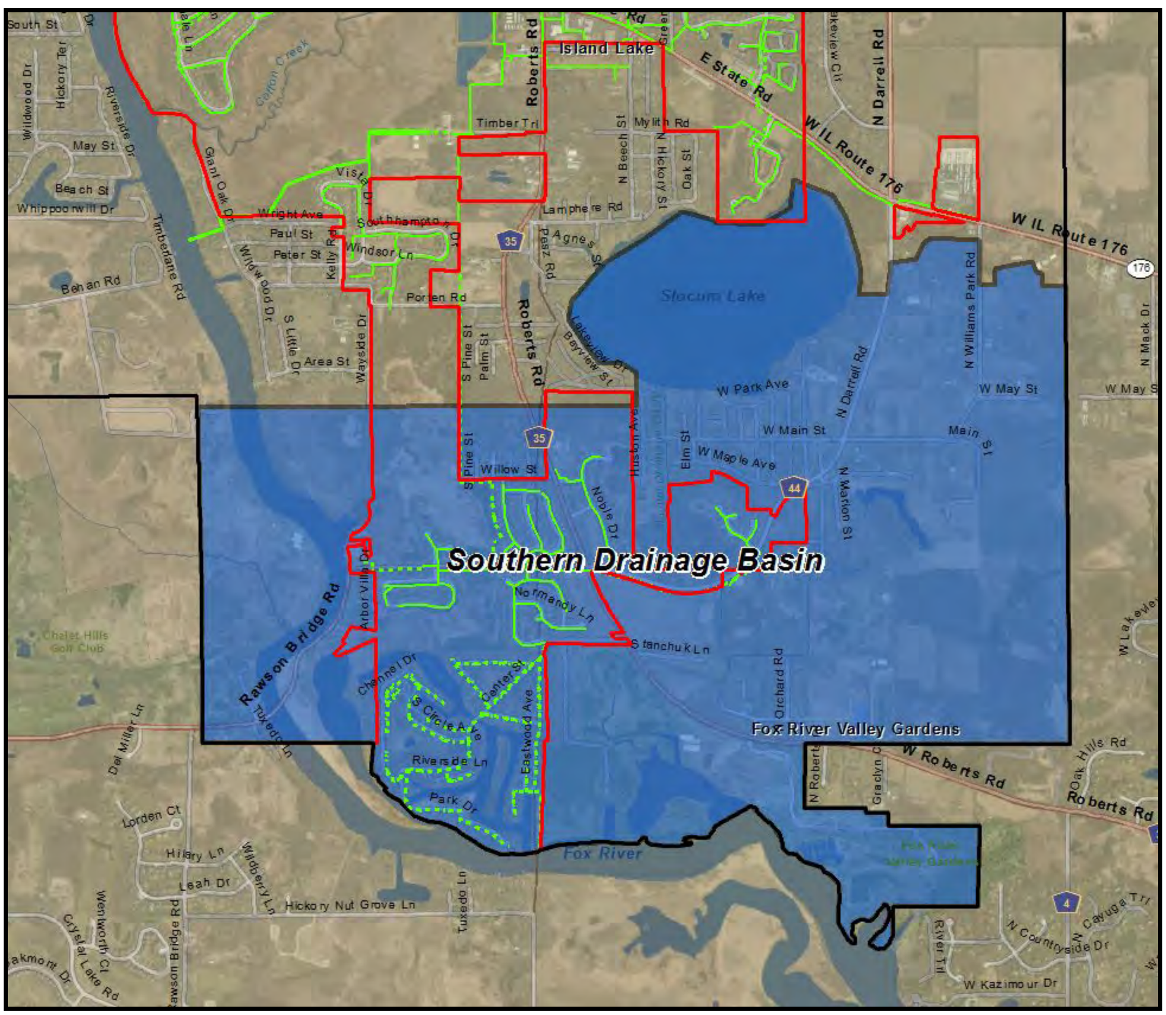
3.4.7 Southern Drainage Basin

The Southern Drainage Basin currently encompasses approximately 139 acres of development in the southernmost portion of the District, including areas north of the Fox River and south of Slocum Lake. The extents of the Southern Drainage Basin and the existing collection system are shown on Exhibit 3-18. The Southern Basin encompasses a total of 2,239 acres, of which 519 acres are currently annexed to the Northern Moraine WRD.

This basin has grown to serve the Deer Grove North Subdivision in The Village of Port Barrington, which connects to District sewers via a low-pressure sewer system. The basin currently includes approximately 1,197 PE, with development having the potential to contribute an additional 10,292 PE at build-out for an ultimate total of 11,489 PE.

The Rawson Bridge Road Lift Station pumps wastewater flow from this drainage basin north through a 10-inch 5,600 lineal foot force main and includes flow pumped from the Deer Grove North Lift Station. The force main from Rawson Bridge Road discharges to a 15-inch gravity sewer directly south of the treatment facility. These lift stations are discussed in Section 4.

Exhibit 3-18: Southern Drainage Basin



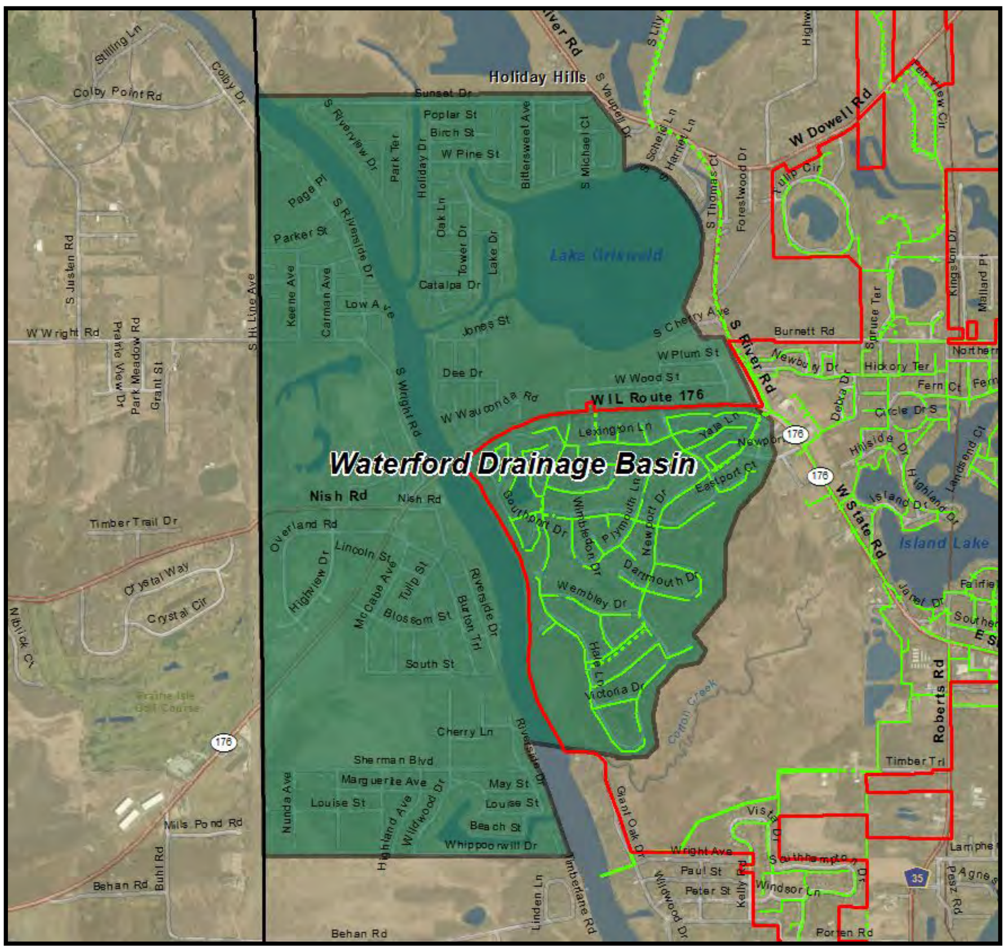
3.4.8 Waterford Drainage Basin

The Waterford Drainage Basin is located northwest of the treatment facility and is bounded by the Fox River to the west and Route 176 to the north. The extents of the Waterford Drainage Basin and the existing collection system are shown on Exhibit 3-19. The basin encompasses a total of 1,743 acres, of which 372 acres are currently annexed to the Northern Moraine WRD.

The Waterford Drainage Basin consists of 214 acres of development that contribute 3,131 PE. At build-out, this basin could contribute an additional 3,690 PE for an ultimate total of 6,821 PE. This would include existing development in the Village of Holiday Hills as well as residential areas located west of the Fox River in unincorporated McHenry County.

This basin currently serves a portion of the Village of Island Lake and includes the Hale 1, Hale 2, and Waterford Lift Stations. The lift stations are discussed in Section 4. The Waterford Lift Station pumps wastewater from the entire basin through an 8-inch 1,200 lineal foot long force main to the 24-inch Route 176 Interceptor sewer.

Exhibit 3-19: Waterford Drainage Basin

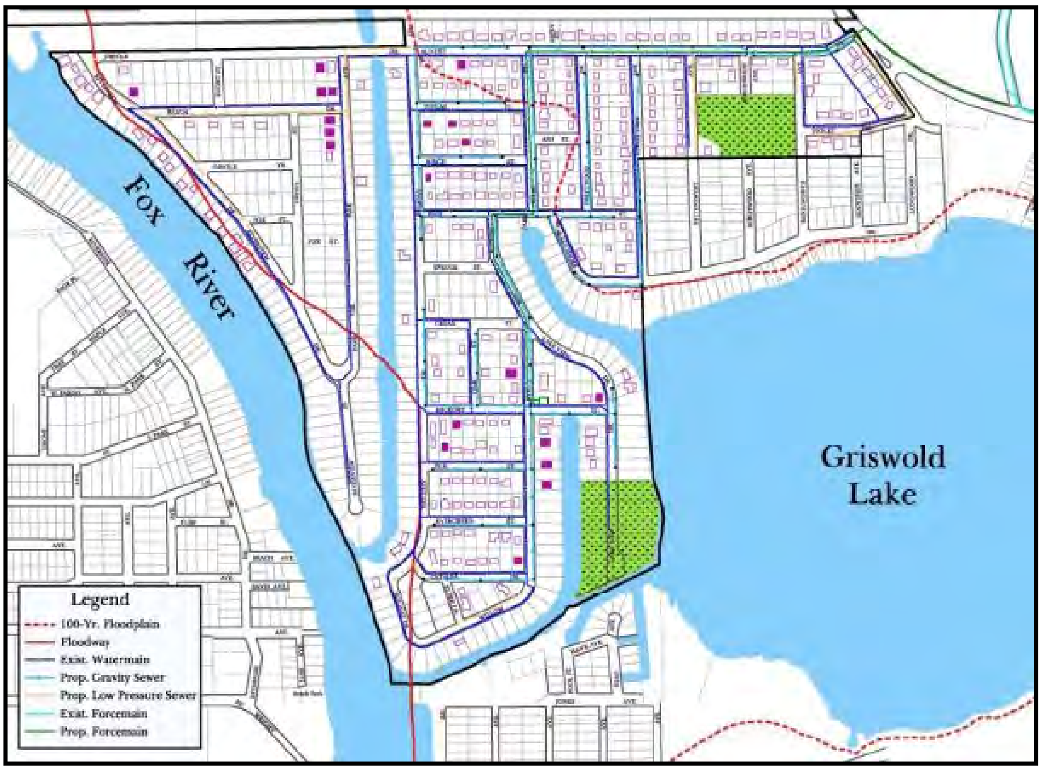


Holiday Hills Collection System

The Village of Holiday Hills completed a feasibility study in 1998 that addressed alternatives to extending NMWRD wastewater service to those areas shown on Exhibit 3-20. That study identified the use of a gravity sewers in conjunction with low-pressure sewers to connect 276 single-family homes to NMWRD 24-inch Route 176 West Interceptor. The feasibility study estimated the capital cost of the project in 1998 dollars at about $2.8 million.

An update to that feasibility study was prepared in 2009. It was concluded that if enough households in Holiday Hills support the extension of sanitary service, an intergovernmental agreement should be entered into for NMWRD to provide sewer service with subsequent construction of the project and disconnection of all residences from their private septic systems. The opinion of the probable cost of this project was updated and stated at $4.9 million. It is currently estimated that connection of Holiday Hills would add 264 residences (924 PE) to the NMWRD service area.

Exhibit 3-20: Holiday Hills Study Area



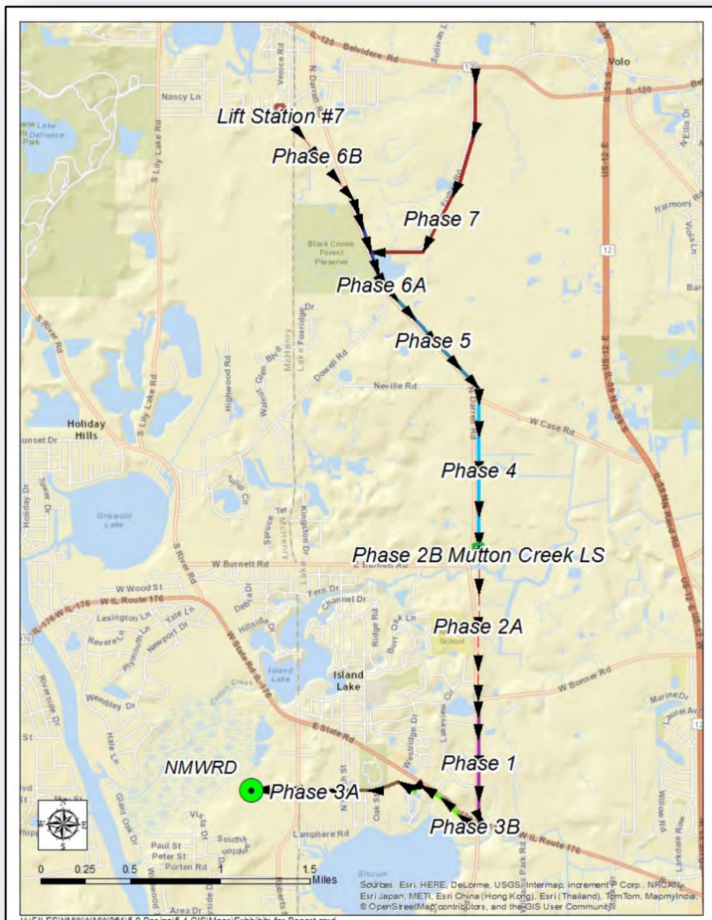
3.5 DARRELL ROAD COLLECTION SYSTEM

The proposed Darrell Road Collection System was recommended in the 2004 Facility Plan Update as a solution to provide service to future development, particularly within the Northeastern and Eastern Drainage Basins. It would also provide the flexibility to reroute flow from built-out areas in the Northeastern Basin thereby off-loading the existing downstream sewers and lift stations. The District has already constructed a section of this system from the Water's Edge Drive Lift Station to the intersection of Darrell Road and Route 176. A phased approach to constructing the remainder of the Darrell Road Collection System is shown on Exhibit 3-21 and is described below.

Phase 1 - Darrell Road Interceptor (South)

The first phase of the Darrell Road Collection System would extend a 36-inch sanitary sewer 3,700 feet north along Darrell Road from Route 176 northeast to Bonner Road. This extension would serve Crown Properties as their land is developed. The upstream terminus manhole in this extension would be constructed to serve as the discharge manhole for the Phase 2 Mutton Creek Lift Station and force.

Exhibit 3-21: Darrell Road Collection System Phased Implementation Plan



Phase 2A - Mutton Creek Force Main

Phase 2A of the Darrell Road Collection System would include the installation of 4,500 lineal feet of 16-inch force main from the proposed Mutton Creek Lift Station to the proposed Phase 1 interceptor at the intersection of Darrell and Bonner Roads.

Phase 2B - Mutton Creek Lift Station

Phase 2B of the Darrell Road Collection System would include the construction of the Mutton Creek Lift Station along Darrell Road near Burnett Road.

Phase 3A - Treatment Plant Interceptor

The Eastern Drainage Basin will include nearly 5,500 PE at build-out and would be served by a proposed 24-inch to 42-inch interceptor sewer that would be routed directly to the treatment facility. The District has already constructed a 2,800 lineal foot portion of this interceptor that runs along the south side of Illinois Route 176 from Water's Edge Drive to the Northeast corner of Illinois Route 176 and Darrell Road, as well as from Water's Edge Drive west to Oak Street.

Phase 3A will include roughly 4,800 lineal feet of 42-inch interceptor sewer to connect the existing 24-inch interceptor to the NMWRD treatment facility and allow for the removal of the Water's Edge Lift Station. This installation would be the catalyst for development north of Illinois Route 176 along Darrell Road.

Phase 3B - Water's Edge Interceptor Replacement

At some point in the future the District will need to increase capacity along Route 176 from Darrell Road to the Water's Edge Lift Station, either paralleling the existing 24-inch interceptor sewer or replacing it with 2,800 lineal feet of 36-inch sewer. The parallel sewer would provide additional capacity in this reach to serve all of the upstream connected areas.

Phase 4 - Darrell Road Interceptor (Central)

Phase 4 of the Darrell Road Collection System would extend a 36-inch sanitary sewer 4,700 lineal feet north along Darrell Road from the proposed Mutton Creek Lift Station north to Case Road. This extension would serve the surrounding properties along Darrell Road from Mutton Creek to Case Road, including Crown Property.

Phase 5 - Darrell Road Interceptor (North)

Phase 5 would extend a 24-inch sanitary sewer 4,000 lineal feet northwest along Darrell Road from Case Road to Dowell Road. This extension would serve the surrounding properties along Darrell Road from Case Road to Dowell Road, including the Crown Property.

Phase 6A - Darrell Road Interceptor (Far North)

Phase 6A would extend a 24-inch sanitary sewer northwest roughly 3,000 lineal feet along Darrell Road from Dowell Road to roughly 3,500 lineal feet south of Wegner Road. This sewer extension will receive the re-directed pumped flow from Lakemoor Lift Station 7.

Phase 6B - Lakemoor Lift Station 7 Force Main

Topographic constraints prevent the Phase 6A interceptor from being extended by gravity all the way into the Village of Lakemoor. Therefore, Phase 6B will include re-routing the force main from Lakemoor Lift Station 7, located at the intersection of Darrell Road and Wegner Road, to pump through a 16-inch 3,500 lineal foot long force main to discharge at a manhole located at the upstream terminus of the Phase 6A interceptor sewer. The Village's existing 24-inch interceptor sewer would then become tributary to this installation.

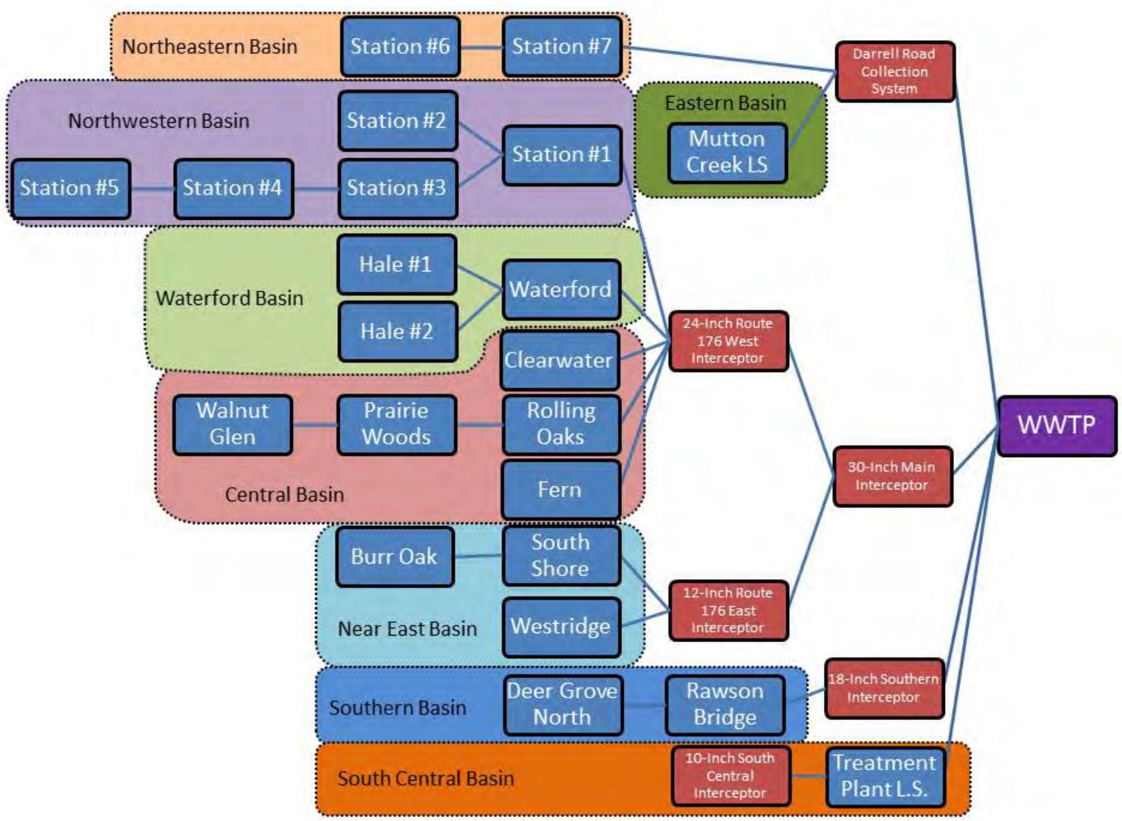
Phase 7 - Fisher Road Interceptor

Phase 7 of the Darrell Road Collection System would extend an 18-inch sanitary sewer approximately 7,500 lineal feet east and north along Fisher Road from Darrell Road to Route 120 within the Village of Lakemoor. This interceptor would serve the eastern portion of the Northeastern Drainage Basin.

Summary – Darrell Road Collection System

The construction of the Darrell Road Collection System would allow the Northeastern Basin to be removed from the 24-inch Route 176 West Interceptor, thereby freeing up capacity to support continued growth in the Waterford, Central and Northwestern Basins. It would also free capacity in the existing 12-inch Lakemoor Lift Station 7 force main on Lily Lake Road which would be sufficient to convey all projected flows resulting from development of the Northwestern Basin. This is depicted in schematic form on Exhibit 3-22.

Exhibit 3-22: Darrelll Road Collection System Schematic



The Darrell Road Collection System would allow for the future development of the Eastern and Northeastern Drainage Basins. Probable capital costs to implement each phase of the Darrell Road Collection System are summarized in Table 3-14.

Table 3-14: Probable Capital Costs - Darrell Road Collection System

Phase	Description	Probable Cost
1	Darrell Road Interceptor (South)	$ 2,255,000
2A	Mutton Creek Force Main	$ 2,149,000
2B	Mutton Creek Lift Station	$ 3,112,000
3A	Treatment Plant Interceptor	$ 5,056,000
3B	Water's Edge Interceptor Replacement	$ 1,902,000
4	Darrell Road Interceptor (Central)	$ 3,279,000
5	Darrell Road Interceptor (North)	$ 2,065,000
6A	Darrell Road Interceptor (Far North)	$ 1,687,000
6B	Lakemoor Lift Station 7 Force Main	$ 1,615,000
7	Fisher Road Interceptor	$ 3,321,000
TOTAL PROBABLE CAPITAL COSTS		**$ 26,441,000**

Because development of the Eastern Basin has been postponed, and is not anticipated to commence in the foreseeable, the Interim Solution Collection System was developed and is recommended as a lesser cost alternative method of directly accommodating long-term growth in the Northeastern and Northwestern Basins.

3.6 INTERIM SOLUTION COLLECTION SYSTEM

Funding of the Darrell Road Collection System has always been contingent upon development within the Eastern Basin. Previously anticipated development within the Eastern Basin has been postponed indefinitely. Because of that, an Interim Solution Collection System was conceived that would allow for growth to continue in the Northwestern and Northeastern Basins by constructing a completely pumped system to serve the northernmost reaches of the FPA.

The interim solution includes upgrades to Lakemoor Lift Station 7 and construction of a new force main from the Northeastern Basin to the intersection of Darrell Road and Route 176, as well as completion of the previously discussed Phase 3A project (42-inch Treatment Plant Interceptor from the NMWRD

Exhibit 3-23: Interim Solution Collection System

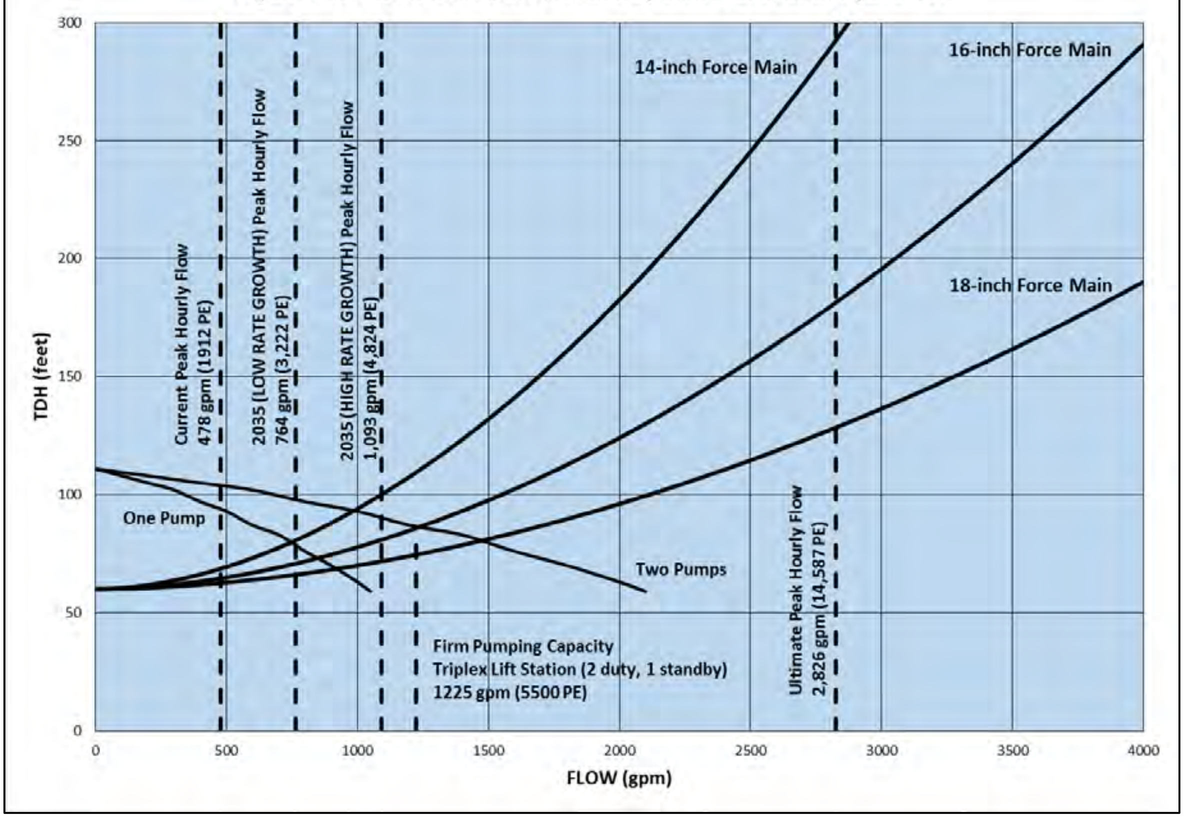
treatment facility to the Water's Edge Lift Station). Therefore, the District would be able to construct all other phases of the Darrell Road Collection System if and when the Eastern Basin is developed.

The sizing of the proposed force main along Darrell Road is dictated by pumping head requirements. Non-clog sewage pumps are only available in pumping heads of up to approximately 200 feet total dynamic head (TDH). A few larger selections exist up to 230 feet but they are not well suited to this application.

Figure 3-1 illustrates the impact of force main sizing to total pumping heads. A 16-inch force main would be adequately sized to convey the ultimate pumped flow of 2,826 gpm without exceeding the maximum pumping head criteria (200 feet).

Also shown on Figure 3-1 are pump curves for the existing pumps installed at Lift Station 7. If a third pump were installed in the existing wet well, the curves indicate that the triplex pumping arrangement would be sufficient to meet even the most optimistic high rate 20-year growth projections. For the lower CMAP projections, the existing dual pump (1 duty, 1 standby) installation would be sufficient through the year 2035. Major modifications to the lift station including much larger pumping units, additional wet well volume, replacement of power centers and the standby power generator, to meet the ultimate conditions could all be deferred to a later date when the total population equivalent in the Northeastern Basin reaches 5,500 PE.

Figure 3 -1: Interim Solution Collection System - Force Main Hydraulics



Probable capital costs for the Interim Solution Collection System are summarized in Table 3-15.

Table 3-15: Probable Capital Costs – Interim Solution Collection System

Phase	Description	Probable Cost
1	**Darrell Road Force Main** (16-inch Force Main on Darrell Road from NE Basin to Route 176)	$8,464,000
2	**Lift Station 7 Upgrades - Future** (Located at existing Lift Station 7 site)	$ 3,112,000
3A	**Treatment Plant Interceptor** (42-inch Sewer from WWTF to Water's Edge Lift Station)	$ 5,056,000
TOTAL PROBABLE CAPITAL COSTS		**$ 16,632,000**

The interim solution provides capacity to serve only the Northeastern Basin, and can be constructed at less cost than implementation of the full Darrell Road Collection System. The interim solution includes major improvements to Lakemoor Lift Station 7 in the future to increase its capacity to serve future growth in the Northeast Basin and to pump that flow all the way to the intersection of Route 176 and Darrell Road.

Under the lower CMAP growth projections, the existing dual pump installation at Lift Station 7 would be sufficient, as is, to meet the year 2035 peak flow projections. Even with the most optimistic high rate growth projections, the installation of a third similarly sized pump is all that would be needed to meet the 20-year peak flow projections.

It is recommended that the District implement the Interim Solution to allow for unimpeded development of the Northeastern Basin. This project would need to be completed prior to exhaustion of the available capacity in the Route 176 West Interceptor (an additional 4,425 PE).

3.7 CONCLUSIONS AND RECOMMENDATIONS

The District's existing collection system is generally in very good condition. Infiltration and inflow during periods of high groundwater and/or wet weather events is currently not excessive. As the existing infrastructure ages the District should be funding replacement accounts at an annual level of approximately $942,100 to provide support for the long-term rehabilitation and replacement of the collection system.

All of the existing interceptor sewers have sufficient capacity to convey current peak hour wastewater flows. However, projected growth within the Waterford, Central, Eastern, Northwestern and Northeastern Drainage Basins will eventually overload the existing 24-inch and 12-inch Route 176 Interceptors. The existing parallel 8-inch and 12-inch force mains from Lakemoor along Lily Lake and River Roads will also one day become overloaded.

The construction of the Darrell Road Collection System would allow the Northeastern Basin to be removed from the 24-inch Route 176 West Interceptor, thereby freely capacity to support continued growth in the Waterford, Central and Northwestern Basins. It would also free capacity in the existing 12-inch Lakemoor Lift Station 7 force main on Lily Lake Road which would be sufficient to convey all projected flows resulting from development of the Northwestern Basin.

However, construction of this system may not be feasible or cost-effective without the participation of development in the Eastern Basin.

Because development of the Eastern Basin has been postponed, and is not anticipated to commence in the foreseeable future, the Interim Solution Collection System was developed and is recommended as a lesser cost alternative method of directly accommodating long-term growth in the Northeastern and Northwestern Basins.

It is recommended that the District implement the Interim Solution to allow for unimpeded development of the Northeastern Basin. This project would need to be completed prior to exhaustion of the available capacity in the Route 176 West Interceptor (an additional 4,425 PE), which represents a 48 percent increase in the combined tributary population from the Northeast, Northwest, Central, and Waterford Drainage Basins.

The remainder of the Darrell Road Collection System could be constructed if and when development of the Eastern Basin occurs.

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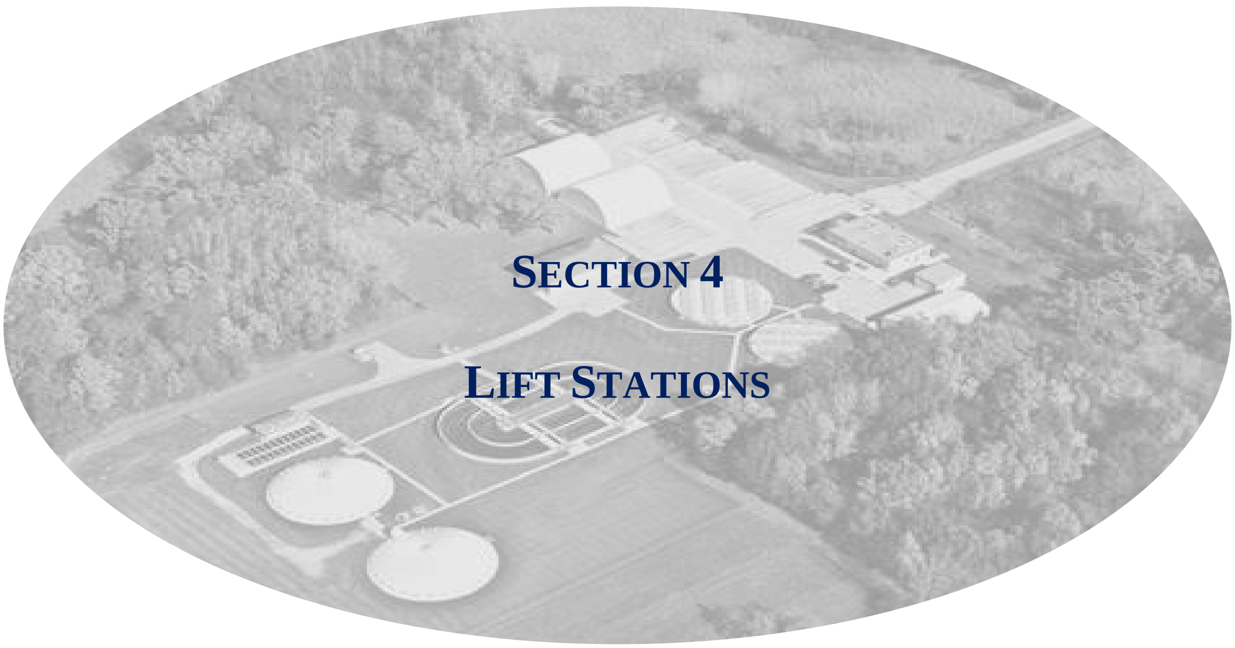

SECTION 4

LIFT STATIONS

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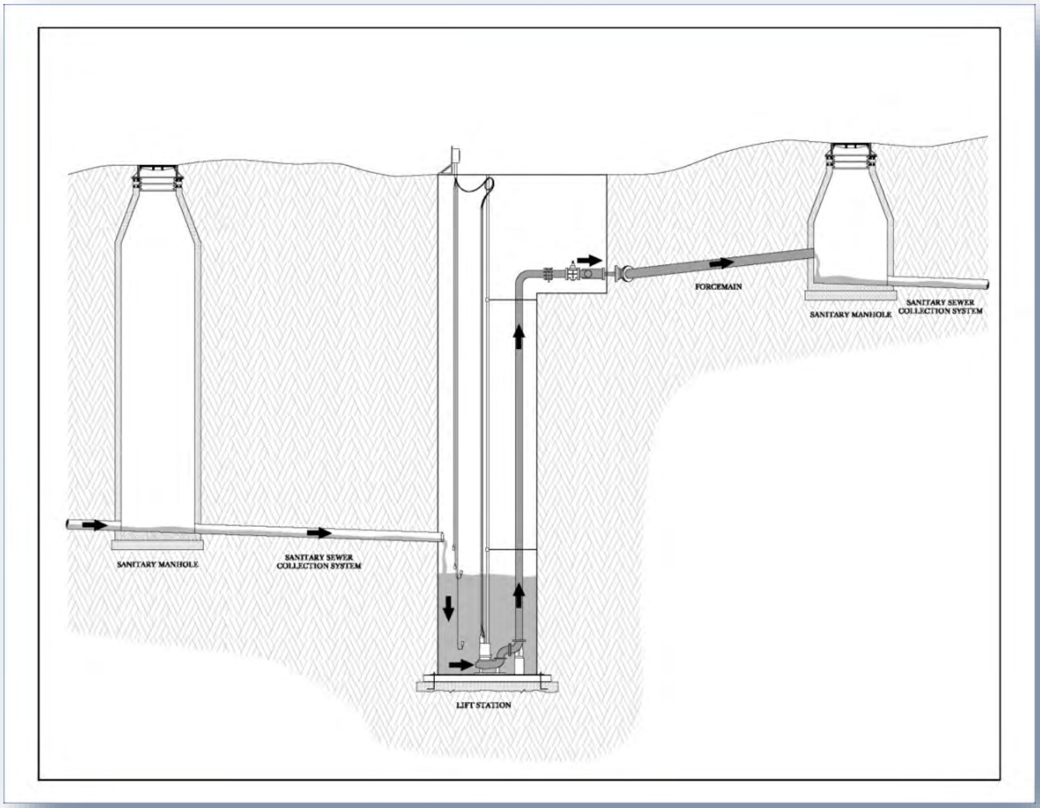
4. LIFT STATIONS

This section describes each of the existing NMWRD lift stations, including their tributary service areas and force mains. Each lift station is assessed based on flows and the results of pump drawdown tests are presented.

4.1 GENERAL INFORMATION

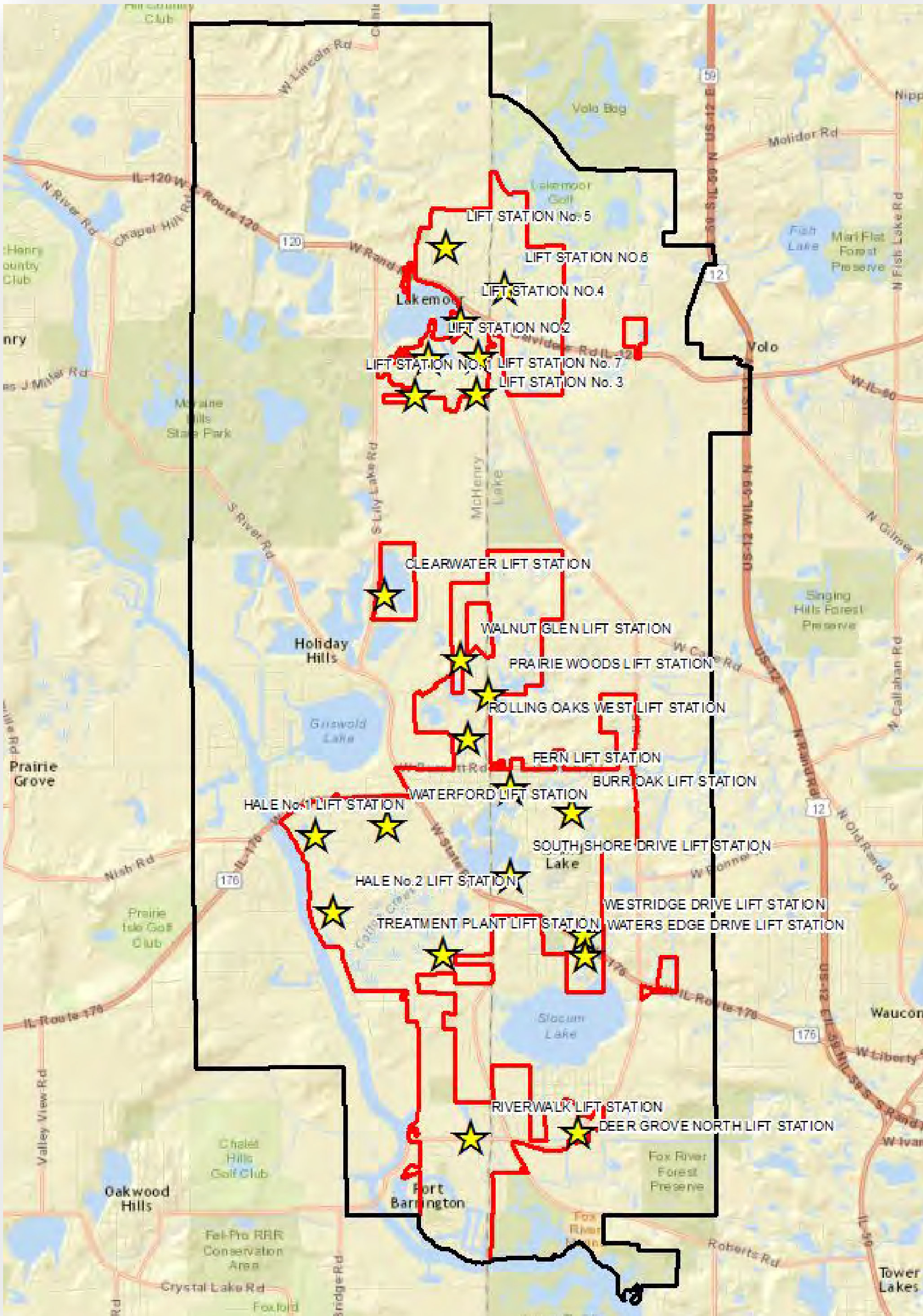
The NMWRD collection system that includes twenty-two (22) wastewater lift stations. Sanitary sewers utilize gravity to convey wastewater. Depending on soil conditions and other factors, such as ground water and ground surface topography, gravity sewer are not always feasible or cost-effective. Lift stations are constructed to pump the wastewater through force mains to higher elevations. The wastewater is then discharged to the downstream gravity sewer. The arrangement of a typical lift station is shown on Exhibit 4-1.

Exhibit 4-1: Typical Lift Station



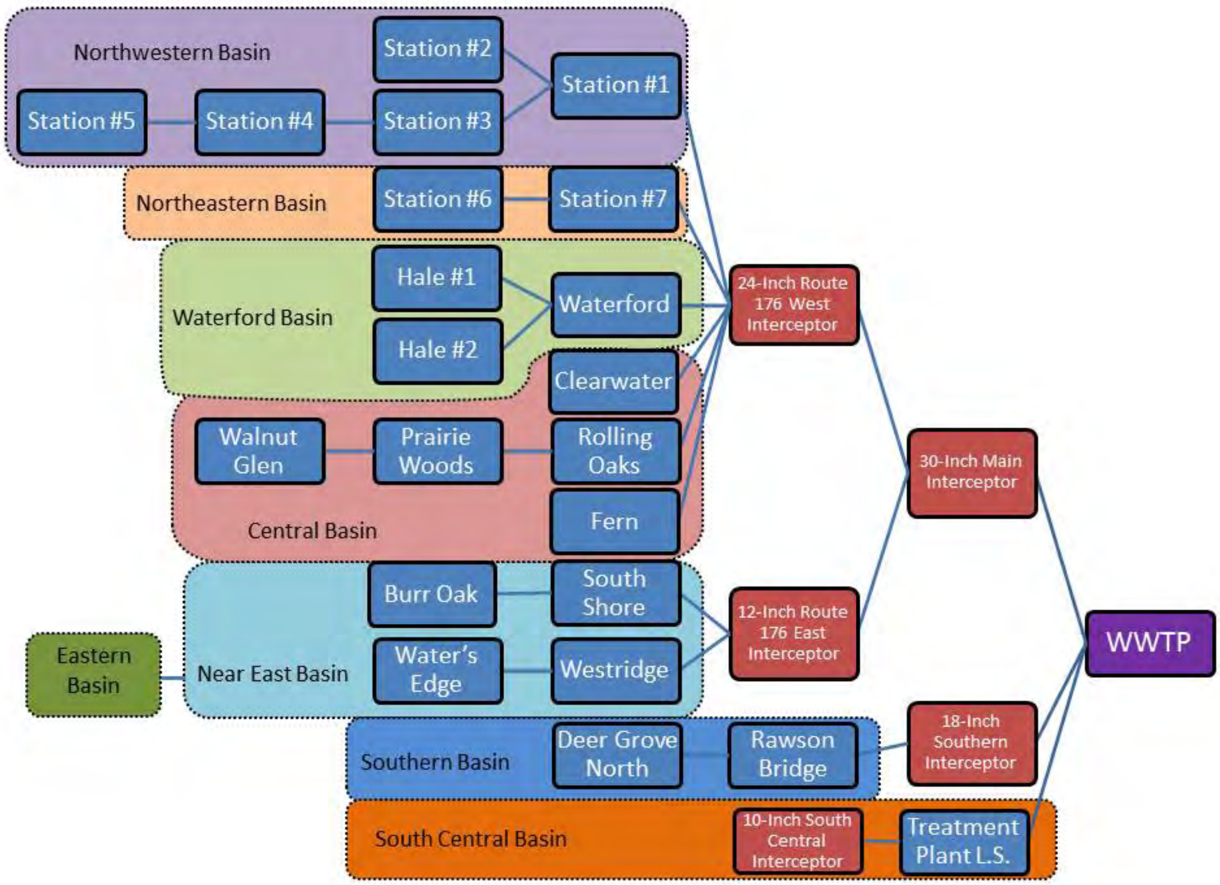
The locations of each NMWRD lift station is shown on Exhibit 4-2.

Exhibit 4-2: NMWRD Lift Station Locations



A schematic of the interconnection of and flow from each lift station through the NMWRD collection system is shown on Exhibit 4-3.

Exhibit 4-3: Lift Station System Flow Schematic



The estimated asset value of each existing NMWRD lift station, as presented in the District's Capital Improvements Plan, is listed in Table 4-1.

Table 4-1: Asset Value of NMWRD Lift Stations

Lift Station	Recorded Value	Equipment/Pumps & Controls		Structures, Piping & Valves	
		Estimated Value (1)	Annual Depreciation (2)	Estimated Value (1)	Annual Depreciation (2)
Hale 1	$ 350,000	$ 140,000	7,000	$ 210,000	4,200
Hale 2	350,000	140,000	7,000	210,000	4,200
Waterford	500,000	200,000	10,000	300,000	6,000
Clearwater	250,000	100,000	5,000	150,000	3,000
Walnut Glen	500,000	200,000	10,000	300,000	6,000
Prairie Woods	500,000	200,000	10,000	300,000	6,000
Rolling Oaks	350,000	140,000	7,000	210,000	4,200
Deer Grove North	750,000	300,000	15,000	450,000	9,000
Rawson Bridge	750,000	300,000	15,000	450,000	9,000
Water's Edge	350,000	140,000	7,000	210,000	4,200
Westridge	350,000	140,000	7,000	210,000	4,200
Burr Oak	350,000	140,000	7,000	210,000	4,200
South Shore	350,000	140,000	7,000	210,000	4,200
Fern	350,000	140,000	7,000	210,000	4,200
Treatment Plant	350,000	140,000	7,000	210,000	4,200
Lakemoor LS 1	750,000	300,000	15,000	450,000	9,000
Lakemoor LS 2	350,000	140,000	7,000	210,000	4,200
Lakemoor LS 3	350,000	140,000	7,000	210,000	4,200
Lakemoor LS 4	350,000	140,000	7,000	210,000	4,200
Lakemoor LS 5	350,000	140,000	7,000	210,000	4,200
Lakemoor LS 6	750,000	300,000	15,000	450,000	9,000
Lakemoor LS 7	850,000	340,000	17,000	510,000	10,200
Total	**$ 10,150,000**	**$ 4,060,000**	**$ 203,000**	**$ 6,090,000**	**$ 121,800**

(1) Equipment estimated at 40% total value. Structures and piping estimated at 60% total value.

(2) Equipment assumed to have 20-year life. Structures and piping assumed have 50-year life.

Sufficient replacement funds should be established to support the rehabilitation and repair efforts necessary to ensure the continued future reliability of the aging lift stations. Based on the depreciation rates listed in Table 4-1, the District should be reinvesting approximately $324,800 annually toward lift station rehabilitation and/or replacement.

4.2 PUMP DRAWDOWN TESTS

Pump drawdown tests consist of timing the wet well fill and draw cycles, and for the known wet well volume, computing the average flow into the wet well and the average pumping rate. Drawdown tests were performed by District staff in September and October of 2014 at each NMWRD lift station to estimate each pump's current output in relation to their rated capacities. Two trials were conducted at each lift station running each pump individually. Due to the piping configurations, pressure gages could not be attached and thus system pressures during the tests were not recorded. The results of the pump drawdown tests are summarized in Table 4-2. This table provides a "snap shot" of pump conditions on the day tested. Many of the pumps have been replaced since the drawdown testing was performed.

Significant deviations from the rated capacity could be a result of a worn pump impeller, varying motor speed due to utility power supply, partially obstructed pump discharge or force main piping, or an improperly designed installation. Hydraulic Institute Standards allow for a deviation from rated pump capacity of plus or minus 8 percent for municipal water and wastewater service (Grade 2B).

Table 4-2: Lift Station Drawdown Test Results

Lift Station	Pump Rated Capacity (gpm)	Pump 1 Drawdown Test (gpm)	Tolerance from Rated Capacity	Pump 2 Drawdown Test (gpm)	Tolerance from Rated Capacity
Hale 1	363	384	6%	231	-36%
Hale 2	238	244	3%	362	52%
Waterford	850	834	-2%	762	-10%
Clearwater	60	43	-28%	63	5%
Walnut Glen	284	285	0%	230	-19%
Prairie Woods	265	319	20%	366	38%
Rolling Oaks	200	193	-4%	212	6%
Deer Grove North	532	499	-6%	433	-19%
Rawson Bridge	740	584	-21%	572	-23%
Water's Edge	100	95	-5%	92	-8%
Westridge	236	247	5%	268	14%
Burr Oak	90	155	72%	103	14%
South Shore	200	208	4%	187	-6%
Fern	576	558	-3%	601	4%
Treatment Plant	235	222	-6%	238	1%
Lakemoor Station 1	450	497	10%	461	2%
Lakemoor Station 2	270	212	-21%	212	-21%
Lakemoor Station 3	270	294	9%	283	5%
Lakemoor Station 4	270	278	3%	214	-21%
Lakemoor Station 5	200	167	-17%	163	-19%
Lakemoor Station 6	502	583	16%	305	-39%
Lakemoor Station 7	800	869	9%	704	-12%

4.3 LIFT STATION ASSESSMENTS

4.3.1 Hale Lift Station 1

General Description

The Hale Lift Station 1 is located at 3440 Hale Lane in Island Lake, which is on the west side of Hale Lane at Southport Village Lake. This lift station serves 70 acres of development in the Waterford Drainage Basin and pumps through about 470 feet of 6-inch force main to manhole B8SE13.

The lift station was constructed in 1986, utilizes a duplex submersible pumping system, and is equipped with a Sensaphone alarm system.

Table 4-3: Hale Lift Station 1 - Pump and Force Main Data

Lift Station	Install Date	Pump Manufacturer	Pump Motors	Rated Capacity (gpm)	Force Main Diameter
Hale 1	1986	Hydromatic (model undocumented)	5 HP	363	6-inch

Exhibit 4-4: Hale Lift Station 1 Service Area

Pump Run Times and Drawdown Tests

Pump runtimes at Hale Lift Station 1 indicate the pumps are sufficiently sized for the current flows. While Pump 1 tested within tolerances, Pump 2 was found to pump only two-thirds its rated capacity, which is confirmed by the fact that it runs on average 30 percent longer than Pump 1. Operators discovered that faulty check valve operation at this station likely was causing recirculation, thereby explaining the decreased pump output. The check valve was replaced in October of 2014.

Table 4-4: Hale Lift Station 1 - Pump Drawdown Test Results

Pump	Avg Operation (hours/day)	Rated Capacity (gpm)	Test Pumping Rate (gpm)	Tolerance from Rated Capacity
1	1.88	363	384	6%
2	2.43	363	231	-36%

Impact of Future Development

Hale Lift Station 1 was designed to serve a built-out neighborhood that encompasses its service area. At this time there are no plans for additional areas to be served by this station.

4.3.2 Hale Lift Station 2

General Description

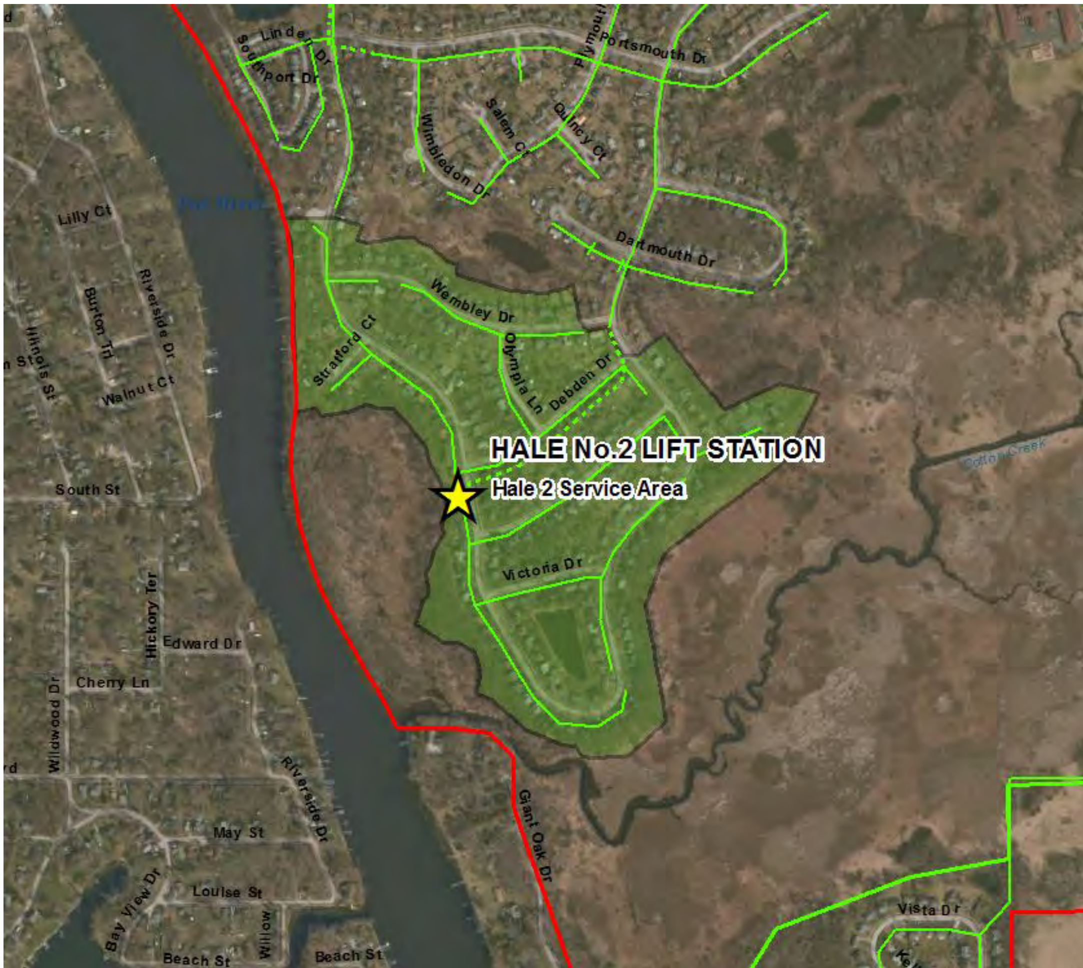
Hale Lift Station 2 is located at 3923 Hale Lane in Island Lake. It was constructed in 1992, utilizes a duplex submersible pumping system, and is equipped with a Sensaphone alarm system.

The lift station serves 90 acres of development in the Waterford Drainage Basin and pumps wastewater through about 1,300 feet of 6-inch force main to manhole C8SW24, located at the intersection of Newport Drive and Wembley Drive.

Table 4-5: Hale Lift Station 2 - Pump and Force Main Data

Lift Station	Install Date	Pump Manufacturer	Pump Motors	Rated Capacity (gpm)	Force Main Diameter
Hale 2	1992	Hydromatic (model undocumented)	7.5 HP	238	6-inch

Exhibit 4-5: Hale Lift Station 2 Service Area



Pump Run Times and Drawdown Tests

Pump runtimes at Hale Lift Station 2 indicate the pumps are sufficiently sized for the current flows. Pump 2 was observed to discharge more than 150 percent more than Pump 1, which pumped very close to rated capacity. Pump 2 was replaced subsequent to the drawdown testing.

Table 4-6: Hale Lift Station 2 - Pump Drawdown Test Results

Pump	Avg Operation (hours/day)	Rated Capacity (gpm)	Test Pumping Rate (gpm)	Tolerance from Rated Capacity
1	1.57	238	244	3%
2	1.64	238	362	52%

Impact of Future Development

Hale Lift Station 2 was designed to serve a built-out neighborhood that encompasses its service area. At this time there are no plans for additional areas to be served by this station.

4.3.3 Waterford Lift Station

General Description

The Waterford Lift Station is located across from 3391 Waterford Way in Island Lake. It was constructed in 1984 and utilizes a duplex submersible pumping system. This lift station serves the entire Waterford Drainage Basin which amounts to roughly 155 acres plus all flow pumped from both Hale Lift Stations 1 and 2, and discharges through about 1200 feet of 8-inch force main to manhole C8NW56.

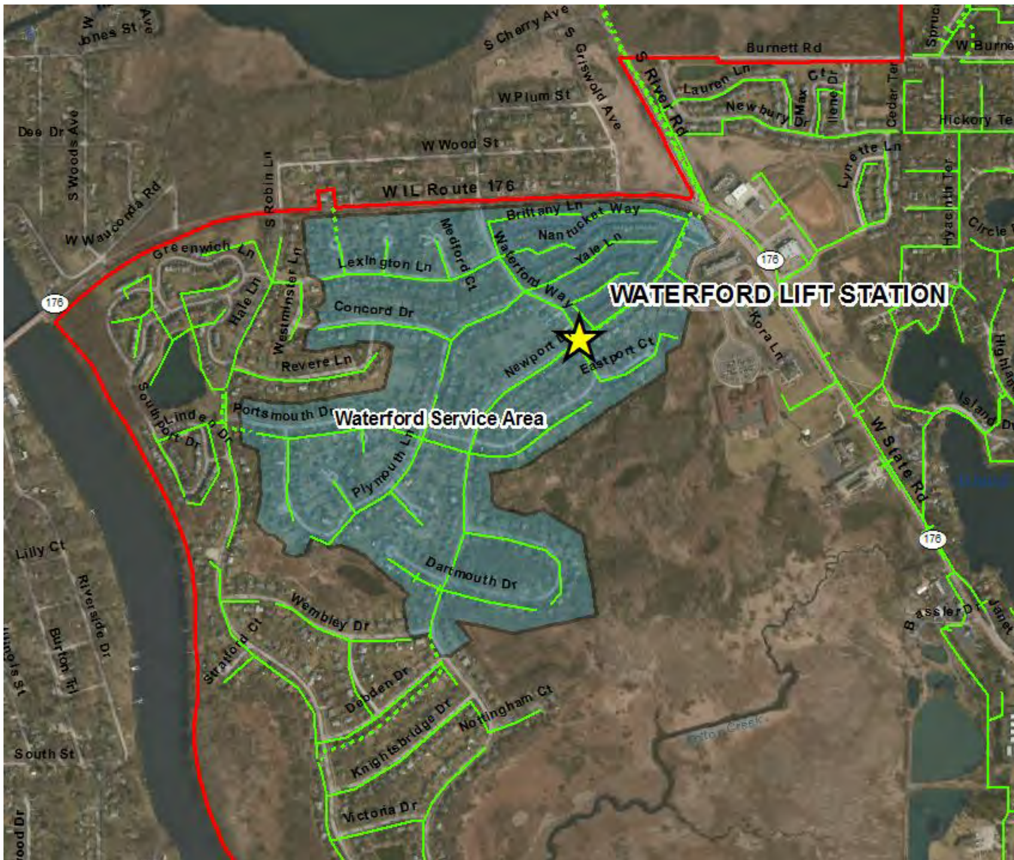
The lift station was upgraded in 2005 with new hatches, rails, and supports.

In 2014 an on-site permanent standby power generator was installed. The lift station is also equipped with a Sensaphone alarm system.

Table 4-7: Waterford Lift Station - Pump and Force Main Data

Lift Station	Install Date	Pump Manufacturer	Pump Motors	Rated Flow (gpm)	Force Main Diameter
Waterford	1985	Hydromatic Model S-4N	20 HP	850	8-inch

Exhibit 4-6: Waterford Lift Station Service Area



Pump Run Times and Drawdown Tests

Pump run times at the Waterford Lift Station indicate the pumps are sufficiently sized for the current flows. Pump run times for Pump 2 were significantly higher than for Pump 1 because Pump 2 was in the LEAD position for an extended period of time. Both pumps tested within or very close to acceptable tolerances.

Table 4-8: Waterford Lift Station - Pump Drawdown Test Results

Pump	Avg Operation (hours/day)	Rated Capacity (gpm)	Test Pumping Rate (gpm)	Tolerance from Rated Capacity
1	1.18	850	834	-2%
2	2.40	850	762	-10%

Impact of Future Development

The Waterford Lift Station was designed to serve a built-out neighborhood that encompasses its service area and also pumped flow from Hale Lift Stations 1 and 2. At this time there are no plans for additional areas to be served by this station.

4.3.4 Clearwater Lift Station

General Description

The Clearwater Lift Station is located at Clearwater Subdivision Outlot #21 at the north end of Stone Drive in McHenry. It was installed in 1990 and most recently rebuilt in 2012.

The lift station utilizes a duplex submersible pumping system, has manual transfer switching to accept a portable generator, and is equipped with a Sensaphone alarm system.

The lift station serves about 20 acres of development in the Central Drainage Basin and discharges south through a 2-inch force main to manhole C8NW56 at the 24-inch Route 176 West Interceptor.

Table 4-9: Clearwater Lift Station - Pump and Force Main Data

Lift Station	Install Date	Rehab Date	Pump Manufacturer	Pump Motors	Rated Capacity (gpm)	Force Main Diameter
Clearwater	1990	2012	Hydromatic Submersible Grinder, Model G2FX500	3 HP	60	2-inch

Exhibit 4-7: Clearwater Lift Station Service Area

Pump Run Times and Drawdown Tests

Pump run times at the Clearwater Lift Station indicate the pumps are sufficiently sized for the current flows. Pump 1 was observed to discharge well below its rated capacity and outside acceptable tolerances. Since this is not indicated by the pump run times, which are nearly identical, it is recommended that the pump drawdown test on Pump 1 be verified.

Table 4-10: Clearwater Lift Station - Pump Drawdown Test Results

Pump	Avg Operation (hours/day)	Rated Capacity (gpm)	Test Pumping Rate (gpm)	Tolerance from Rated Capacity
1	0.26	60	43	-28%
2	0.27	60	63	5%

Impact of Future Development

The Clearwater Lift Station was designed to serve the neighborhood that encompasses its service area. At this time there are no plans for additional areas to be served by this station.

4.3.5 Walnut Glen Lift Station

General Description

The Walnut Glen Lift Station is located at 2285 Walnut Glen Boulevard in Island Lake. It utilizes a duplex submersible pumping system, and serves around 100 acres of development in the Central and Eastern Drainage Basins.

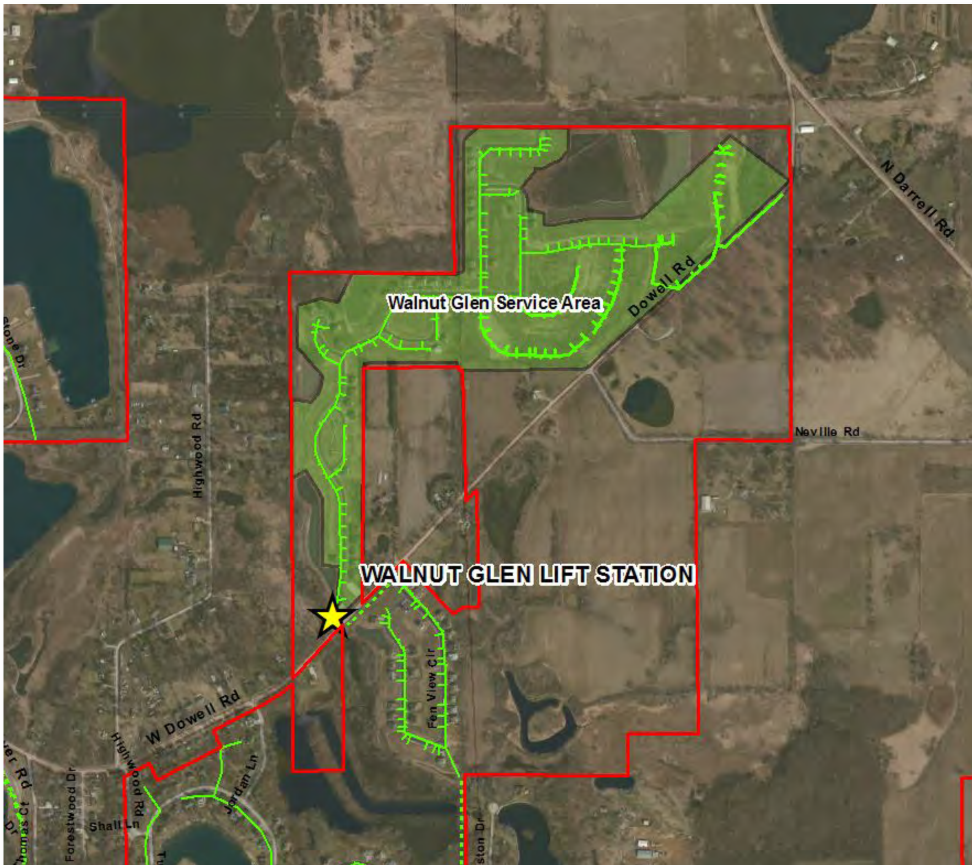
Wastewater is pumped from the lift station through about 630 feet of 6-inch force main.

This lift station is equipped with an on-site standby power generator, a quick connect for emergency bypass pumping, and a Sensaphone alarm system.

Table 4-11: Walnut Glen Lift Station - Pump and Force Main Data

Lift Station	Install Date	Pump Manufacturer	Pump Motors	Rated Capacity (gpm)	TDH (feet)	Force Main Diameter
Walnut Glen	2006	4” KJI Hydro Submersible	10 HP	284	28	6-inch

Exhibit 4-8: Walnut Glen Service Area



Pump Run Times and Drawdown Tests

Pump run times at the Walnut Glen Lift Station indicate the pumps are sufficiently sized for the current flows. Pump 1 was observed to discharge exactly at its rated capacity, while Pump 2 tested well below its rated capacity and outside acceptable tolerances. Since this is not indicated by the pump run times, reduced output from Pump 2 would result in increased run times which is opposite to that recorded. It is recommended that the Pump 2 drawdown test be verified.

Table 4-12: Walnut Glen Lift Station - Pump Drawdown Test Results

Pump	Avg Operation (hours/day)	Rated Capacity (gpm)	Test Pumping Rate (gpm)	Tolerance from Rated Capacity
1	0.30	284	285	0%
2	0.27	284	230	-19%

Impact of Future Development

The Walnut Glen Lift Station was designed to serve the neighborhood that encompasses its service area. At this time there are no plans for additional areas to be served by this station.

4.3.6 Prairie Woods Lift Station

General Description

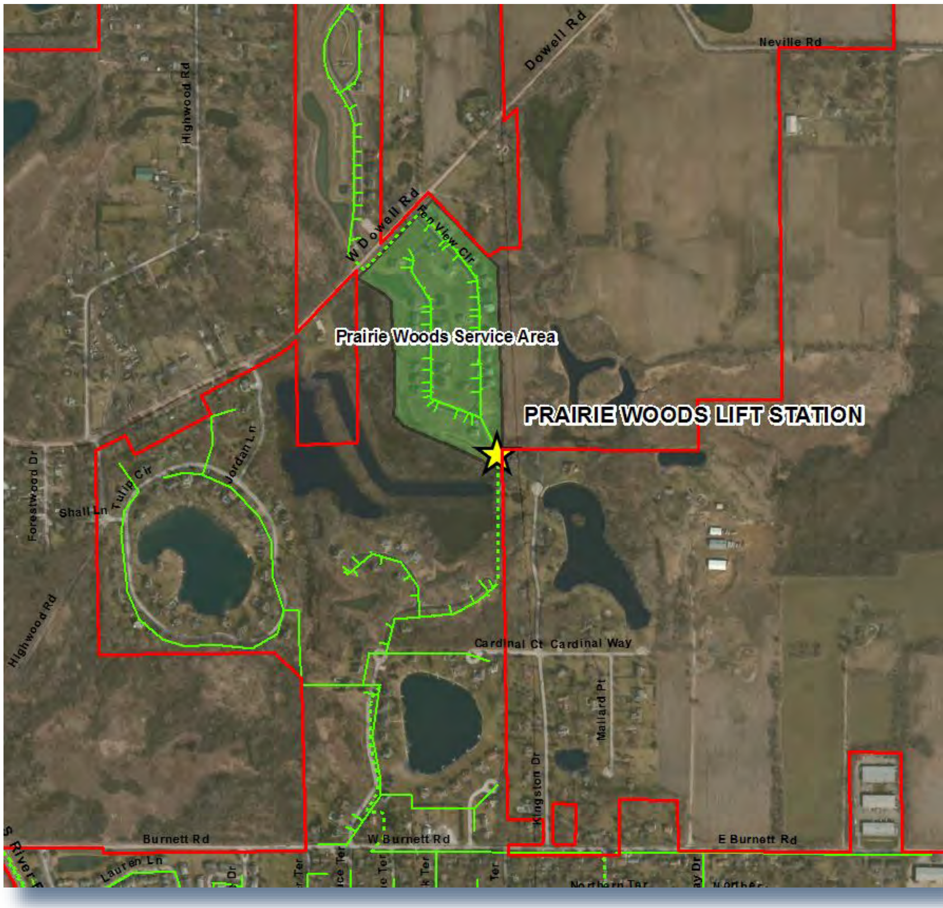
The Prairie Wood Lift Station is located in the Prairie Woods Subdivision at the south end of Fen View Circle in Island Lake. It utilizes a duplex submersible pumping system, and discharges through 815 feet of 6-inch force main to manhole C7SE47. The lift station serves roughly 25 acres of development in the Central Drainage Basin as well as flow pumped from the Walnut Glen Lift Station.

This lift station is equipped with an on-site standby power generator and a Sensaphone alarm system.

Table 4-13: Prairie Woods Lift Station - Pump and Force Main Data

Lift Station	Install Date	Pump Manufacturer	Pump Motors	Rated Capacity (gpm)	TDH (feet)	Force Main Diameter
Prairie Woods	2006	(undocumented)	7.5 HP	265	36	6-inch

Exhibit 4-9: Prairie Woods Lift Station Service Area



Pump Run Times and Drawdown Tests

Pump run times at the Prairie Woods Lift Station indicate the pumps are sufficiently sized for the current flows. Both pumps tests significantly above their rated capacities. The hydraulic characteristics of these pumps should be checked to ensure that the motors are not being overloaded and/or that cavitation is not occurring.

Table 4-14: Prairie Woods Lift Station - Pump Drawdown Test Results

Pump	Avg Operation (hours/day)	Rated Flow (gpm)	Test Flow Rate (gpm)	Tolerance from Rated Capacity
1	0.35	265	319	20%
2	0.32	265	366	38%

Impact of Future Development

The Prairie Woods Lift Station was designed to serve the neighborhood that encompasses its service area and tributary flow from Walnut Glen. At this time there are no plans for additional areas to be served by this station.

4.4.7 Rolling Oaks Lift Station

General Description

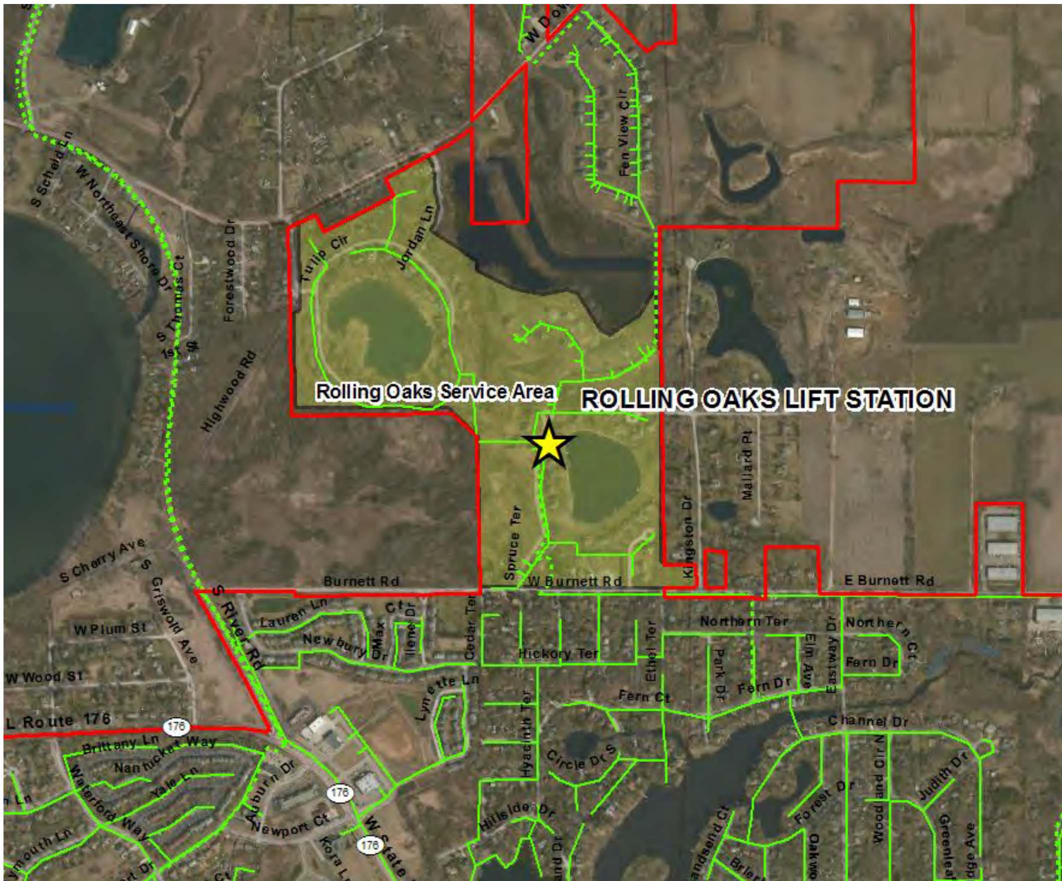
The Rolling Oaks Lift Station is located at 2900 Spruce Terrace in Island Lake. Formerly known as the Spruce Lift Station, it was built in 1992 as part of the Rolling Oaks West Subdivision. The lift station utilizes a duplex submersible pumping system, and is equipped with a Sensaphone alarm system.

The Rolling Oaks Lift Station serves around 110 acres of development in the Central Drainage Basin as well as flow pumped from the Prairie Woods and Walnut Glen Lift Stations. The lift station discharges through about 1275 feet of 6-inch force main to manhole C8NE60.

Table 4-15: Rolling Oaks Lift Station - Pump and Force Main Data

Lift Station	Install Date	Pump Manufacturer	Pump Motors	Rated Capacity (gpm)	Force Main Diameter
Rolling Oaks	1992	Hydromatic S4750 N	7.5	200	6-inch

Exhibit 4-10: Rolling Oaks Lift Station Service Area



Pump Run Times and Drawdown Tests

Pump run times at the Rolling Oaks Lift Station indicate the pumps are sufficiently sized for the current flows. Pump run times for Pump 1 were significantly higher than for Pump 2 because Pump 1 was in the LEAD position for an extended period of time. Both pumps tested within or very close to acceptable tolerances.

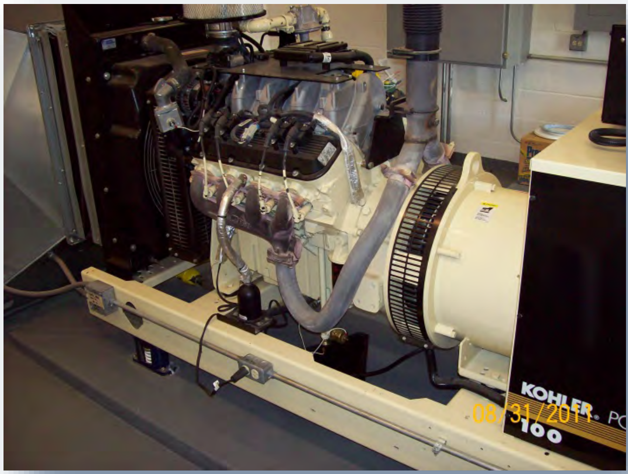
Table 4-16: Rolling Oaks Lift Station - Pump Drawdown Test Results

Pump	Avg Operation (hours/day)	Rated Capacity (gpm)	Test Pumping Rate (gpm)	Tolerance from Rated Capacity
1	1.64	200	193	-4%
2	1.06	200	212	6%

Impact of Future Development

The Rolling Oaks Lift Station was designed to serve the neighborhood that encompasses its service area and tributary flow from Walnut Glen and Prairie Woods. At this time there are no plans for additional areas to be served by this station.

4.4.8 Deer Grove Lift Station



General Description

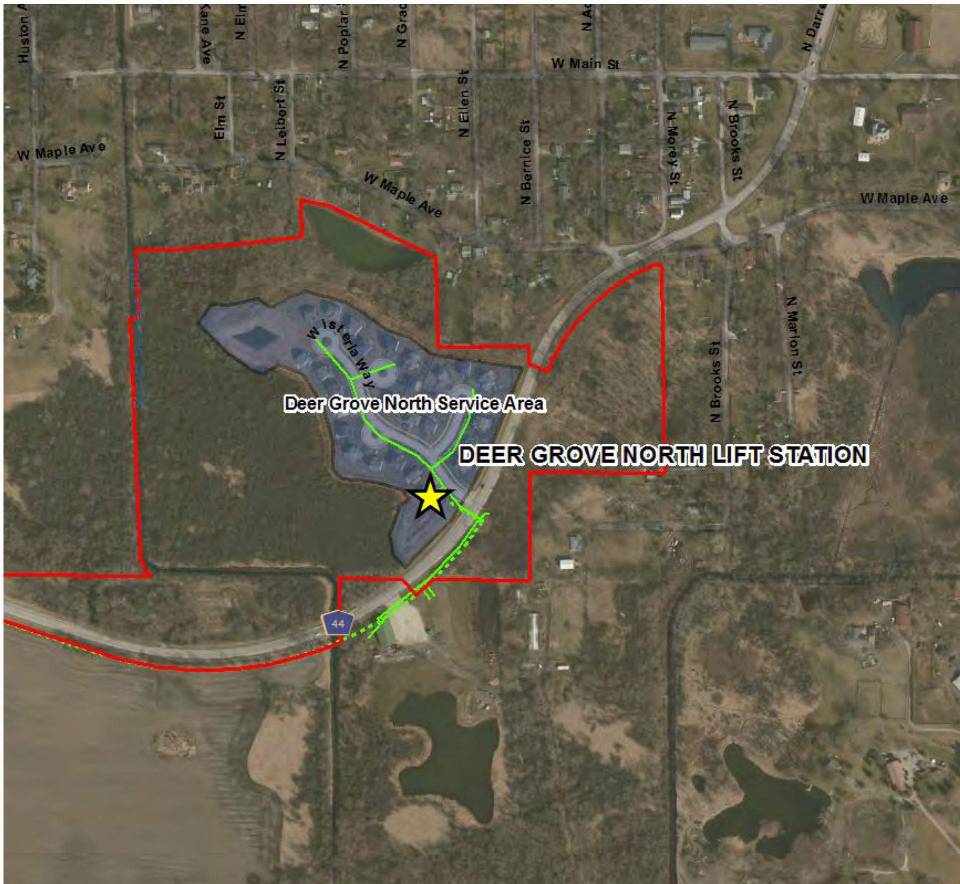
The Deer Grove Lift Station is located at 2629 Wisteria Way in Port Barrington. The station was constructed in 2005 as part of the Deer Grove North Subdivision. It uses a duplex submersible pumping system, serves roughly 15 acres of development in the Southern Drainage Basin, and discharges through approximately 2,720 feet of 8-inch force main to manhole D10SW05.

This lift station is equipped with an on-site standby power generator and a Sensaphone alarm system.

Table 4-17: Deer Grove Lift Station - Pump and Force Main Data

Lift Station	Install Date	Pump Manufacturer	Pump Motors	Rated Flow (gpm)	TDH (feet)	Force Main Diameter
Deer Grove	2005	KJI Hydro Model 4” KSE-10-4T	10 HP	532	43	8-inch

Exhibit 4-11: Deer Grove North Lift Station Service Area



Pump Run Times and Drawdown Tests

Pump run times at the Deer Grove Lift Station indicate the pumps are sufficiently sized for the current flows. Pump 1 was observed to discharge within acceptable tolerances, while Pump 2 tested well below its rated capacity and outside acceptable tolerances. It is recommended that the Pump 2 drawdown test be verified, and that Pump 2 be checked for clogging.

Table 4-18: Deer Grove Lift Station - Pump Drawdown Test Results

Pump	Avg Operation (hours/day)	Rated Capacity (gpm)	Test Pumping Rate (gpm)	Tolerance from Rated Capacity
1	0.15	532	499	-6%
2	0.17	532	433	-19%

Impact of Future Development

The Deer Grove North Lift Station was designed to serve the neighborhood that encompasses its service area. At this time there are no plans for additional areas to be served by this station.

4.4.9 Rawson Bridge (Riverwalk) Lift Station

General Description

The Rawson Bridge Road Lift Station is located at 100 Rawson Bridge Road in Port Barrington. It is also known as the Riverwalk Lift Station. It was constructed in 1998 and consists of a duplex submersible pumping system which discharges through about 5,000 feet of 10-inch force main to manhole C9SE23.

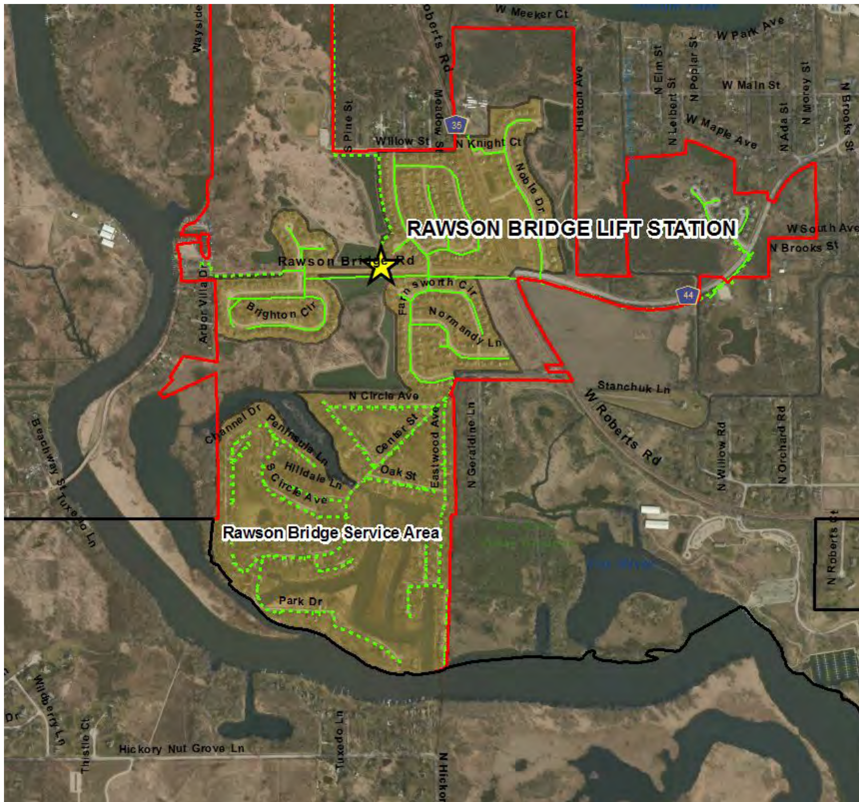
This lift station serves the entire 275 acres of development in the Southern Drainage Basin, as well as all flow pumped from the Deer Grove North Lift Station.

The station includes a natural gas engine standby power generator, located inside the adjacent control building, and is equipped with a Sensaphone alarm system.

Table 4-19: Rawson Bridge Lift Station - Pump and Force Main Data

Lift Station	Install Date	Pump Manufacturer	Pump Motors	Rated Flow (gpm)	Force Main Diameter
Rawson	1998	Hydromatic (model undocumented)	30 HP	740	10-inch

Exhibit 4-12: Rawson Bridge Lift Station Service Area



Pump Run Times and Drawdown Tests

Pump run times at the Rawson Bridge Lift Station indicate the pumps are sufficiently sized for the current flows. However, both pumps were tested to output (almost identically) significantly less than their rated capacity and outside acceptable tolerances. This is indicative that any clogging is affecting both pumps and would thus be located in the force main. On the other hand, these pumps may have simply been under-designed. It is recommended that the drawdown tests be verified, and that hydraulic computations be performed for this system.

Table 4-20: Rawson Bridge Lift Station – Pump Drawdown Test Results

Pump	Avg Operation (hours/day)	Rated Capacity (gpm)	Test Pumping Rate (gpm)	Tolerance from Rated Capacity
1	1.08	740	584	-21%
2	1.19	740	572	-23%

Impact of Future Development

The Rawson Bridge Lift Station was designed to serve the neighborhood that encompasses its service area and tributary flow from Deer Grove North. At this time there are no plans for additional areas to be served by this station.

4.4.10 Water's Edge Lift Station

General Description

The Water's Edge Lift Station is located at 4320 Water's Edge Drive in Island Lake, south of Illinois Route 176. The lift station consists of a small duplex submersible pumping system, and is equipped with a Sensaphone alarm system.

This lift station serves about 35 acres of development in the Eastern and Near East Drainage Basins. Wastewater is pumped through around 720 feet of 4-inch force main directly to the Westridge Lift Station, located on the east side of Westridge Drive directly north of Illinois Route 176.

Table 4-21: Water's Edge Lift Station - Pump and Force Main Data

Lift Station	Install Date	Pump Manufacturer	Pumps	Rated Flow (GPM)	Force Main Diameter
Water's Edge	2000	Hydromatic Model S4NX300	5 HP	100	4-inch

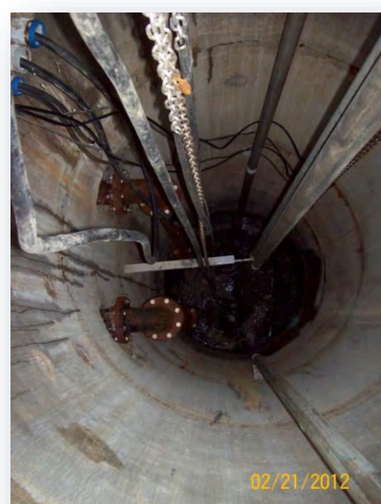


Exhibit 4-13: Water's Edge Lift Station Service Area

Pump Run Times and Drawdown Tests

Pump run times at the Water's Edge Lift Station indicate the pumps are sufficiently sized for the current flows. In addition, both pumps tested within acceptable tolerances.

Table 4-22: Water's Edge Lift Station - Pump Drawdown Test Results

Pump	Avg Hours/Day of Operation	Rated Flow (gpm)	Test Flow Rate (gpm)	Tolerance from Rated Capacity
1	0.88	100	95	-5%
2	0.80	100	92	-8%

Impact of Future Development

The Water's Edge Lift Station was designed to serve the neighborhood and commercial development that encompasses its service area. At this time there are no plans for additional areas to be served by this station.

4.4.11 Westridge Lift Station

General Description

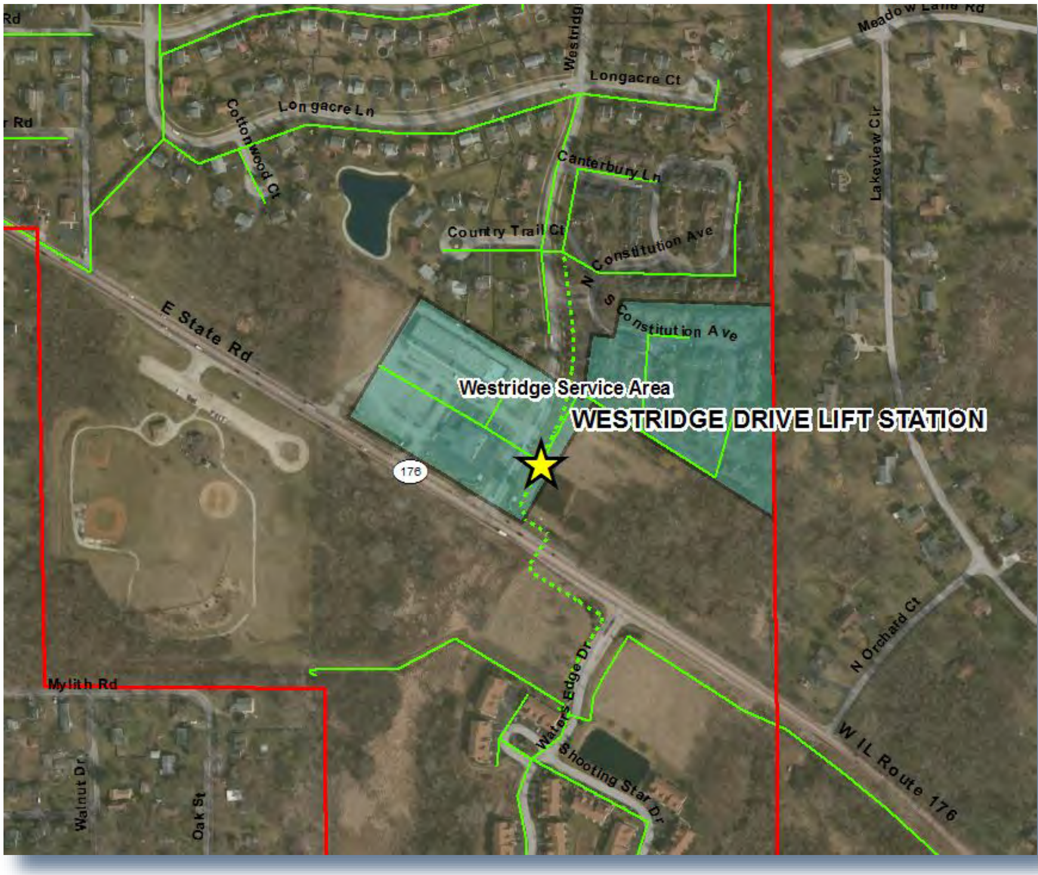
Westridge Lift Station is located directly north of Illinois Route 176 on the east side of Westridge Drive in Island Lake. This lift station was constructed in 1994, is comprised of a duplex submersible pumping system, and is equipped with a Sensaphone alarm system. Wastewater is pumped through 575 feet of 4-inch force main that discharges to manhole D9NE03.

The lift station serves around 12 acres of development in the Near East Drainage Basin, as well as all flow pumped from the Water's Edge Lift Station.

Table 4-23: Westridge Lift Station - Pump and Force Main Data

Lift Station	Install Date	Pump Manufacturer	Pump Motors	Rated Capacity (gpm)	Force Main Diameter
Westridge	1994	Hydromatic (model undocumented)	15 HP	236	4-inch

Exhibit 4-14: Westridge Drive Lift Service Area



Pump Run Times and Drawdown Tests

Pump run times at the Westridge Lift Station indicate the pumps are sufficiently sized for the current flows. Pump 1 tested within tolerances, while Pump 2 tested significantly above its rated capacity. It is recommended that the drawdown test be verified, and if repeated that the hydraulic characteristics of Pump 2 be checked to ensure that the motors are not being overloaded and/or that cavitation is not occurring.

Table 4-24: Westridge Lift Station - Pump Drawdown Test Results

Pump	Avg Operation (hours/day)	Rated Capacity (gpm)	Test Pumping Rate (gpm)	Tolerance from Rated Capacity
1	0.87	236	247	5%
2	0.70	236	268	14%

Impact of Future Development

The Westridge Lift Station was designed to serve the neighborhood and commercial development that encompasses its service area and tributary flow from Water's Edge. At this time there are no plans for additional areas to be served by this station.

4.4.12 Burr Oak Lift Station

General Description

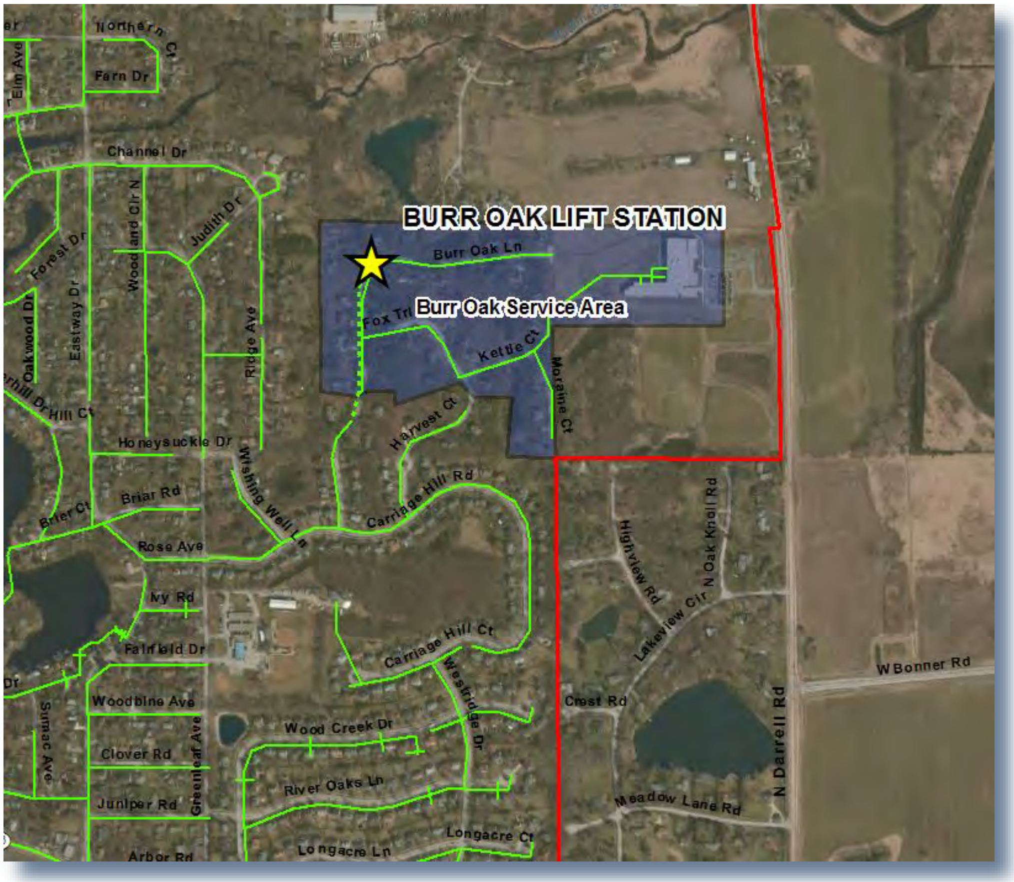
The Burr Oak Lift Station is located at 3314 Burr Oak Lane directly north of Fox Trail in Island Lake. This lift station was installed in 1997, uses a duplex submersible pumping system, and is equipped with a Sensaphone alarm system.

Wastewater is pumped through about 950 feet of 4-inch force main to manhole D8NE21 on Burr Oak Lane. The lift station serves about 45 acres of development in both the Near East and Eastern Drainage Basins.

Table 4-25: Burr Oak Lift Station - Pump and Force Main Data

Lift Station	Install Date	Rehab Date	Pump Manufacturer	Pump Motors	Rated Capacity (gpm)	Force Main Diameter
Burr Oak	1997	2001	Hydromatic (model undocumented)	5 HP	90	4-inch

Exhibit 4-15: Burr Oak Lift Station Service Area



Pump Run Times and Drawdown Tests

Pump run times at the Burr Oak Lift Station indicate the pumps are sufficiently sized for the current flows. However, both pumps tested significantly above their rated capacities, and alarmingly so in the case of Pump 1. It is recommended that the drawdown test be verified, and if repeated that the hydraulic characteristics of both pump, especially Pump 1, be checked to ensure that the motors are not being overloaded and/or that cavitation is not occurring.

Table 4-26: Burr Oak Lift Station - Pump Drawdown Test Results

Pump	Avg Operation (hours/day)	Rated Capacity (gpm)	Test Pumping Rate (gpm)	Tolerance from Rated Capacity
1	0.52	90	155	72%
2	0.55	90	103	14%

Impact of Future Development

The Burr Oak Lift Station was designed to serve the neighborhood and commercial development that encompasses its service area. At this time there are no plans for additional areas to be served by this station.

4.3.13 South Shore Lift Station

General Description

The South Shore Lift Station is located on 230 South Shore Drive in Island Lake, directly south of the lake. This lift station was constructed in 1978, had new pumps installed in 2003, and was rebuilt in 2011. The lift station is equipped with an on-site standby power generator and a Sensaphone alarm system.

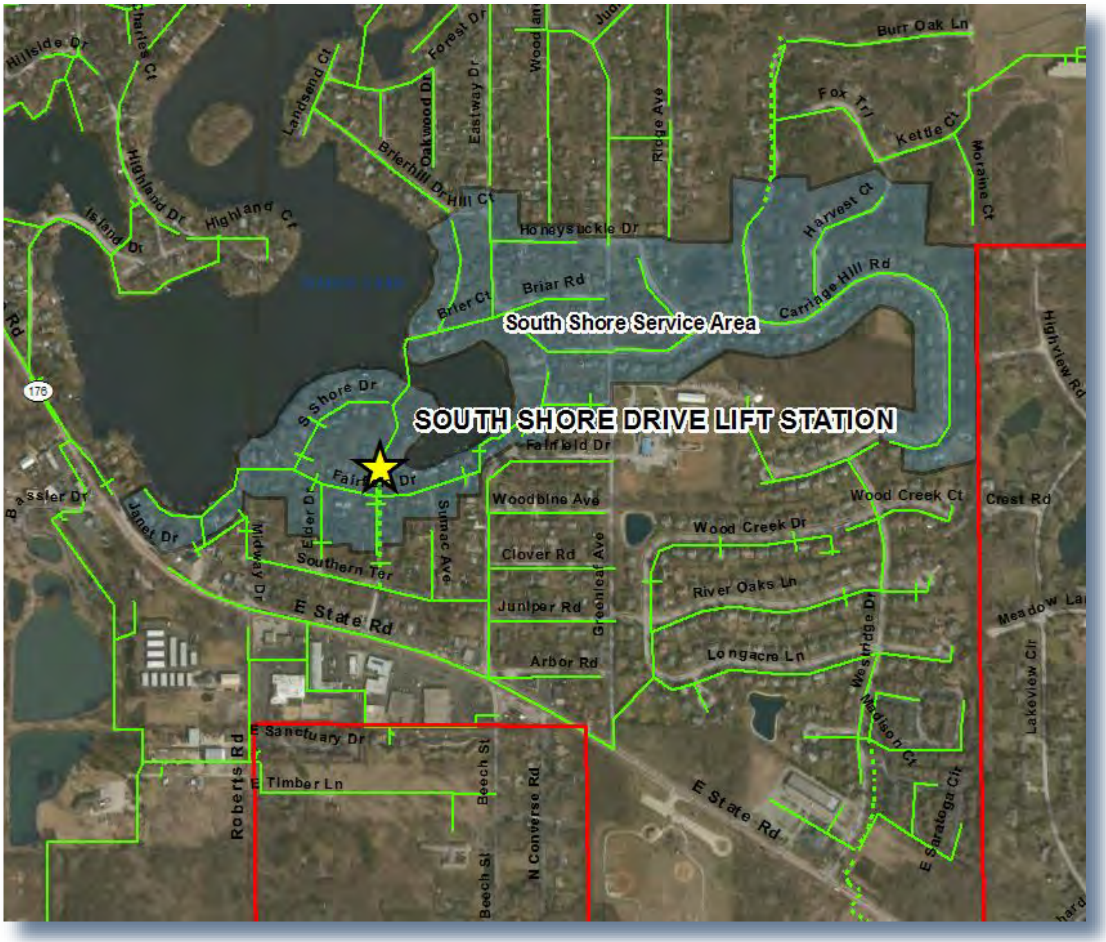
The South Shore Lift Station uses a duplex submersible pumping system, and discharges through 650 feet of 6-inch force main to manhole D8SW16 in the intersection of Southern Terrace and Woodlawn Drive.

This lift station serves around 90 acres of development in the Near East Drainage Basin, as well as flow pumped from the Burr Oak Lift Station.

Table 4-27: South Shore Lift Station - Pump and Force Main Data

Lift Station	Install Date	Rehab Date	Pump Manufacturer	Pump Motors	Rated Capacity (gpm)	TDH (feet)	Force Main Diameter
South Shore	1978	2011	Flygt Model NP3102.090	5 HP	200	27	6-inch

Exhibit 4-16: South Shore Lift Station Service Area



Pump Performance Evaluation

Pump run times at the South Shore Lift Station indicate the pumps are sufficiently sized for the current flows. In addition, both pumps tested within acceptable tolerances.

Table 4-28: South Shore Drawdown Results

Pump	Avg Hours/Day of Operation	Rated Flow (gpm)	Test Flow Rate (gpm)	Tolerance from Rated Capacity
1	1.69	200	208	4%
2	1.88	200	187	-6%

Impact of Future Development on the Lift Station Infrastructure

The South Shore Lift Station was designed to serve the neighborhoods that encompass its service area and tributary flow from Burr Oak. At this time there are no plans for additional areas to be served by this station.

4.3.14 Fern Lift Station

General Description

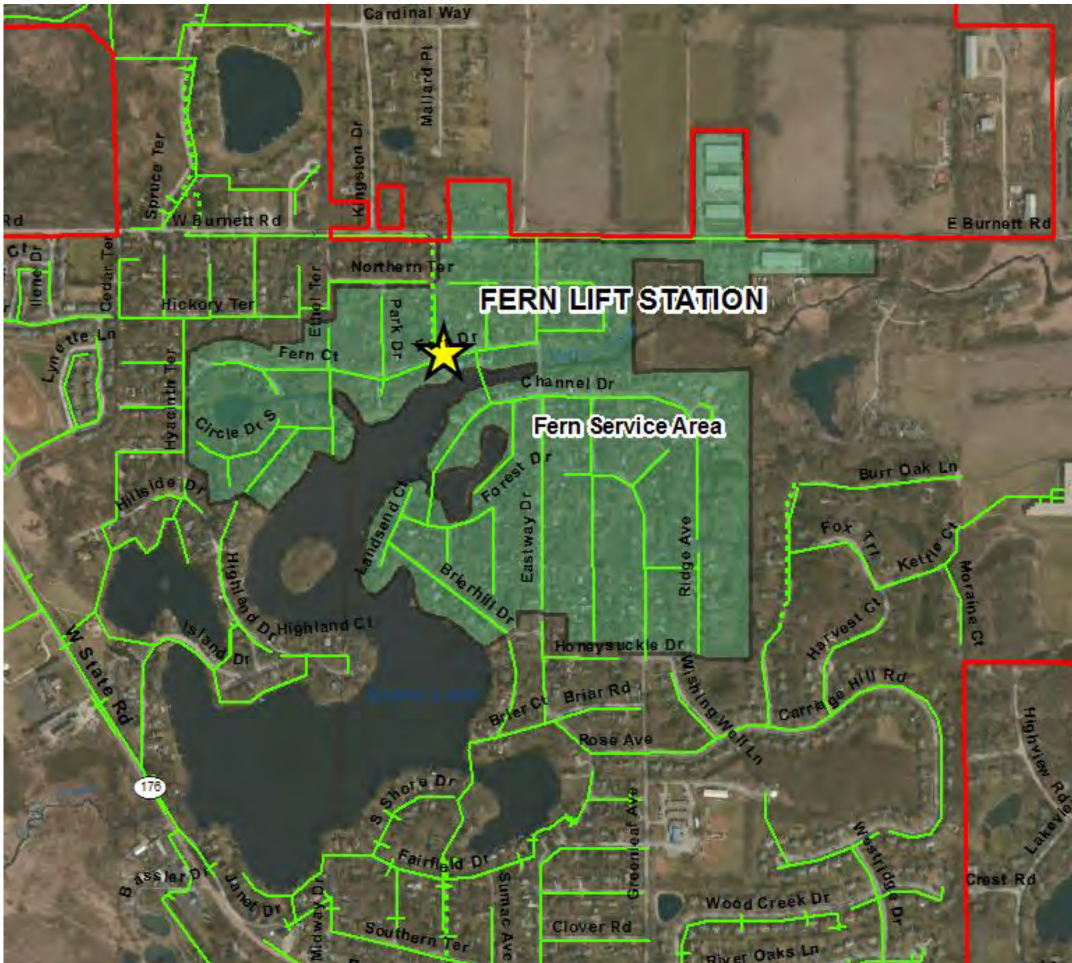
The Fern Lift Station is located on the northeast corner of Fern Drive and Poplar Drive in Island Lake. It uses a duplex submersible pumping system which discharges through 675 feet of 8-inch force main to manhole D8NW02 on Burnett Drive. This lift station serves 150 acres of development in the Central Drainage Basin.

The Fern Lift Station was installed in 1978 and most recently rebuilt in 2011. This lift station is equipped with an on-site standby power generator and has a Sensaphone alarm system.

Table 4-29: Fern Lift Station - Pump and Force Main Data

Lift Station	Install Date	Rehab Date	Pump Manufacturer	Pump Motors	Rated Capacity (gpm)	TDH (feet)	Force Main Diameter
Fern	1978	2011	Flygt Model NP3127.090	10 HP	576	43	8-inch

Exhibit 4-17: Fern Lift Station Service Area



Pump Performance Evaluation

Pump run times at the Fern Lift Station indicate the pumps are sufficiently sized for the current flows. In addition, both pumps tested within acceptable tolerances.

Table 4-30: Fern Lift Station - Pump Drawdown Test Results

Pump	Avg Operation (hours/day)	Rated Capacity (gpm)	Test Pumping Rate (gpm)	Tolerance from Rated Capacity
1	2.85	576	558	-3%
2	1.74	576	601	4%

Impact of Future Development on the Lift Station Infrastructure

The Fern Lift Station was designed to serve the neighborhood and commercial development that encompasses its service area. At this time there are no plans for additional areas to be served by this station.

4.3.15 Treatment Plant Lift Station

General Description

The Treatment Plant Lift Station is located on-site at the NMWRD wastewater treatment facility. The station was installed in 1993 and most recently rebuilt in 2009.

This lift station's duplex submersible pumping system discharges wastewater through about 350 feet of 4-inch force main directly to manhole C9NE12 at the treatment facility Headworks.

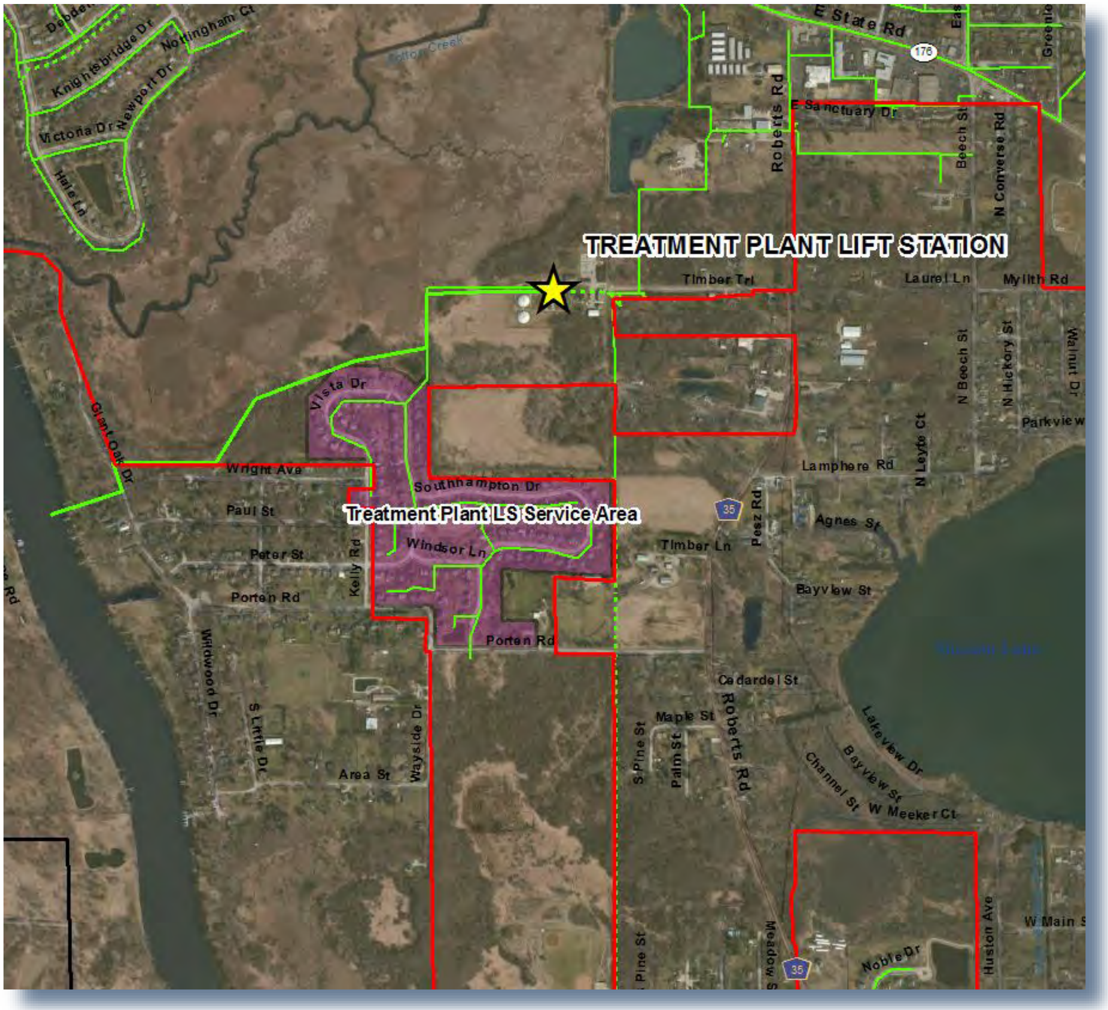
This lift station also receives the skimmings that come off the top of the chlorine contact tank. The lift station is connected to the treatment facility's standby power generator and alarm system.

The station serves about 60 acres of development in the South Central Drainage Basin.

Table 4-31: Treatment Plant Lift Station - Pump and Force Main Data

Lift Station	Install Date	Rehab Date	Pump Manufacturer	Pump Motors	Rated Capacity (gpm)	Force Main Diameter
Treatment Plant	1993	2009	Hydromatic (model undocumented)	3 HP	235	4-inch

Exhibit 4-18: Treatment Plant Lift Station Service Area



Pump Run Times and Drawdown Tests

Pump run times at the Treatment Plant Lift Station indicate the pumps are sufficiently sized for the current flows. In addition, both pumps tested within acceptable tolerances.

Table 4-32: Treatment Plant Lift Station - Pump Drawdown Results

Pump	Avg Hours/Day of Operation	Rated Flow (gpm)	Test Flow Rate (gpm)	Tolerance from Rated Capacity
1	2.25	235	222	-6%
2	1.74	235	238	1%

Impact of Future Development on the Lift Station Infrastructure

The Treatment Plant Lift Station was designed to serve the neighborhood that encompasses its service area. At this time there are no plans for additional areas to be served by this station.

4.3.16 Lakemoor Lift Station 1

General Description

Lakemoor Lift Station 1 is located at 524 Wegner Road in Lakemoor. It uses a duplex submersible pumping system, is equipped with an on-site standby power generator, and is equipped with a Sensaphone alarm system. The lift station was installed in 1978 and rehabilitated in 2007. Site improvements were most recently completed in 2014.

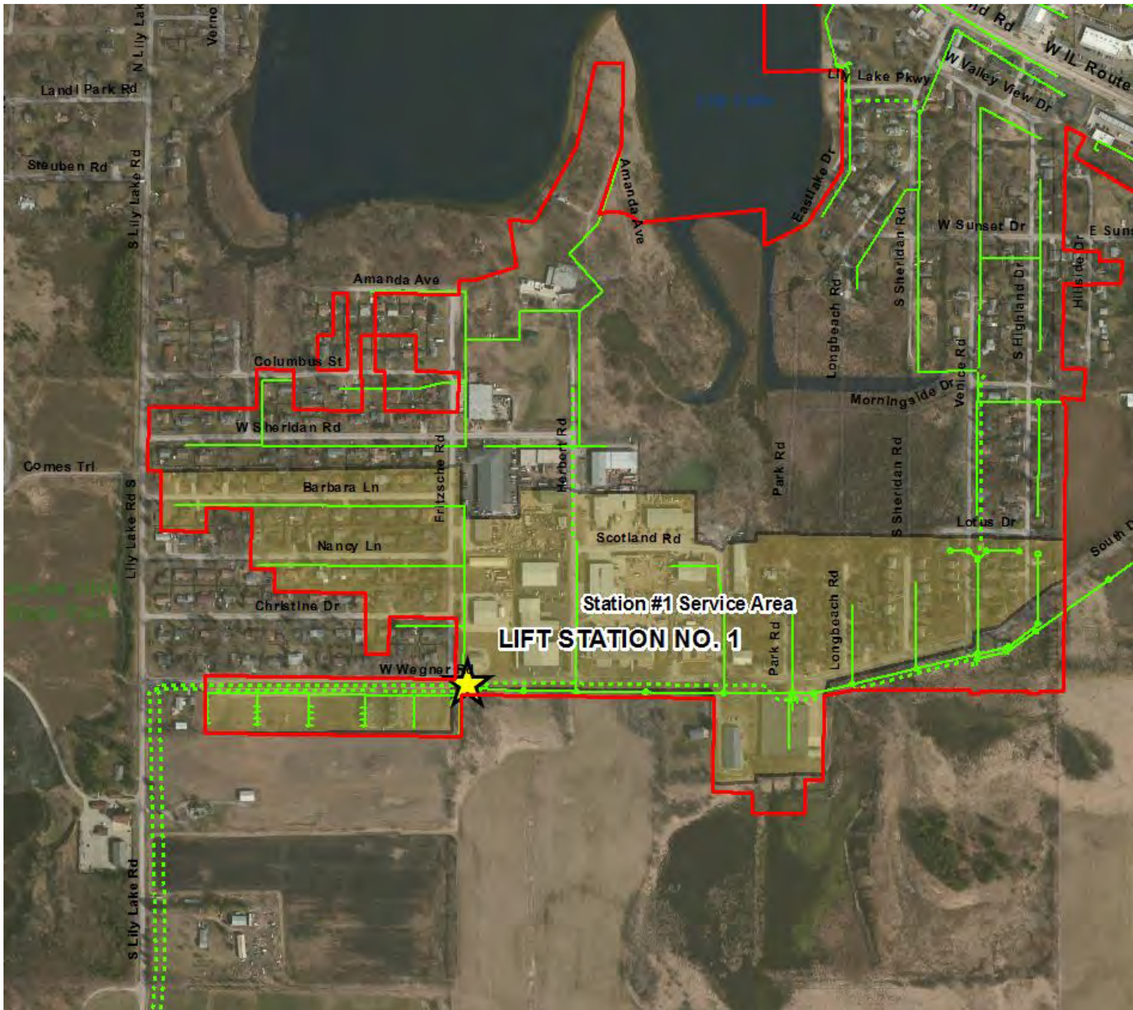
Site Improvements – November 2014

The lift station serves around 65 acres of development in the Northwestern Drainage Basin as well as flow pumped from Lakemoor Lift Stations 2, 3, 4, and 5. Wastewater is pumped south along Lily Lake and River Roads through an 8-inch force main to manhole C8NW57 in the 24-inch Route 176 West Interceptor.

Table 4-33: Lakemoor Lift Station 1 - Pump and Force Main Data

Lift Station	Install Date	Rehab Date	Pump Manufacturer	Pump Motors	Rated Capacity (gpm)	Force Main Diameter
Lakemoor 1	1978	2007	KJI 4-inch SKE-5-4T-001	15 HP	450	8-inch

Exhibit 4-19: Lakemoor Lift Station 1 Service Area



Pump Run Times and Drawdown Tests

Pump run times at Lakemoor Lift Station 1 indicate the pumps are sufficiently sized for the current flows. In addition, both pumps tested within or very close to acceptable tolerances.

Table 4-34: Lakemoor Lift Station 1 - Pump Drawdown Test Results

Pump	Avg Operation (hours/day)	Rated Capacity (gpm)	Test Pumping Rate (gpm)	Tolerance from Rated Capacity
1	3.00	450	497	10%
2	2.72	450	461	2%

Impact of Future Development

Future growth will have a significant impact on Lakemoor Lift Station 1. These impacts are discussed at length in Section 4.4.

4.3.17 Lakemoor Lift Station 2

General Description

Lakemoor Lift Station 2 is located at 349 Herbert Road in Lakemoor. It uses a duplex submersible pumping system, is equipped to accept a portable generator, and is equipped with a Sensaphone alarm system.

The lift station serves around 30 acres of development in the Northwestern Drainage Basin and discharges through 680 feet of 4-inch force main to manhole C5NE25.

Lakemoor Lift Station 2 was installed in 1978 and most recently rehabilitated in 2007.

Table 4-35: Lakemoor Lift Station 2 - Pump and Force Main Data

Lift Station	Install Date	Rehab Date	Pump Manufacturer	Pump Motors	Rated Capacity (gpm)	Force Main Diameter
Lakemoor 2	1978	2007	KJI 4-inch SKE-5-4T-001	3 HP	270	4-inch

Exhibit 4-20: Lakemoor Lift Station 2 Service Area

Pump Run Times and Drawdown Tests

Pump run times at Lakemoor Lift Station 2 indicate the pumps are sufficiently sized for the current flows. However, both pumps were tested to output (almost identically) significantly less than their rated capacity and outside acceptable tolerances. This is indicative that any clogging is effecting both pumps and would thus be located in the force main. On the other hand, these pumps may have simply been under-designed. It is recommended that the drawdown tests be verified, and that hydraulic computations be performed for this system.

Table 4-36: Lakemoor Lift Station 2 - Pump Drawdown Test Results

Pump	Avg Hours/Day of Operation	Rated Flow (gpm)	Test Flow Rate (gpm)	Tolerance from Rated Capacity
1	0.22	270	212	-21%
2	0.21	270	212	-21%

Impact of Future Development

Lakemoor Lift Station 2 was designed to serve the neighborhood and commercial development that encompasses its service area. At this time there are no plans for additional areas to be served by this station.

4.3.18 Lakemoor Lift Station 3

General Description

Lakemoor Lift Station 3 is located at 316 Venice Road in Lakemoor. The lit station was installed in 1978 and most recently rehabilitated in 2007.

This station utilizes a duplex submersible pumping system, is equipped to accept a portable generator, and is equipped with a Sensaphone alarm system. The pumps discharge through 775 feet of 6-inch force main to manhole C5NE45.

Lakemoor Lift Station 3 serves about 35 acres of development in the Northwestern Drainage Basin as well as flow pumped from Lakemoor Lift Stations 4 and 5.

Table 4-37: Lakemoor Lift Station 3 - Pump and Force Main Data

Lift Station	Install Date	Rehab Date	Pump Manufacturer	Pump Motors	Rated Capacity (gpm)	Force Main Diameter
Lakemoor 3	1978	2007	KJI 4-inch SKE-5-4T-001	3 HP	270	6-inch

Exhibit 4-21: Lakemoor Lift Station 3 Service Area

Pump Run Times and Drawdown Tests

Pump run times at Lakemoor Lift Station 3 indicate the pumps are sufficiently sized for the current flows. In addition, both pumps tested within or very close to acceptable tolerances.

Table 4-38: Lakemoor Lift Station 3 - Pump Drawdown Test Results

Pump	Avg Operation (hours/day)	Rated Capacity (gpm)	Test Pumping Rate (gpm)	Tolerance from Rated Capacity
No. 1	2.41	270	294	9%
No. 2	2.55	270	283	5%

Impact of Future Development on the Lift Station Infrastructure

Lakemoor Lift Station 3 was designed to serve the neighborhood and commercial development that encompasses its service area and tributary flow from Lakemoor Stations #4 and #5. At this time there are no plans for additional areas to be served by this station.

4.3.19 Lakemoor Lift Station 4

General Description

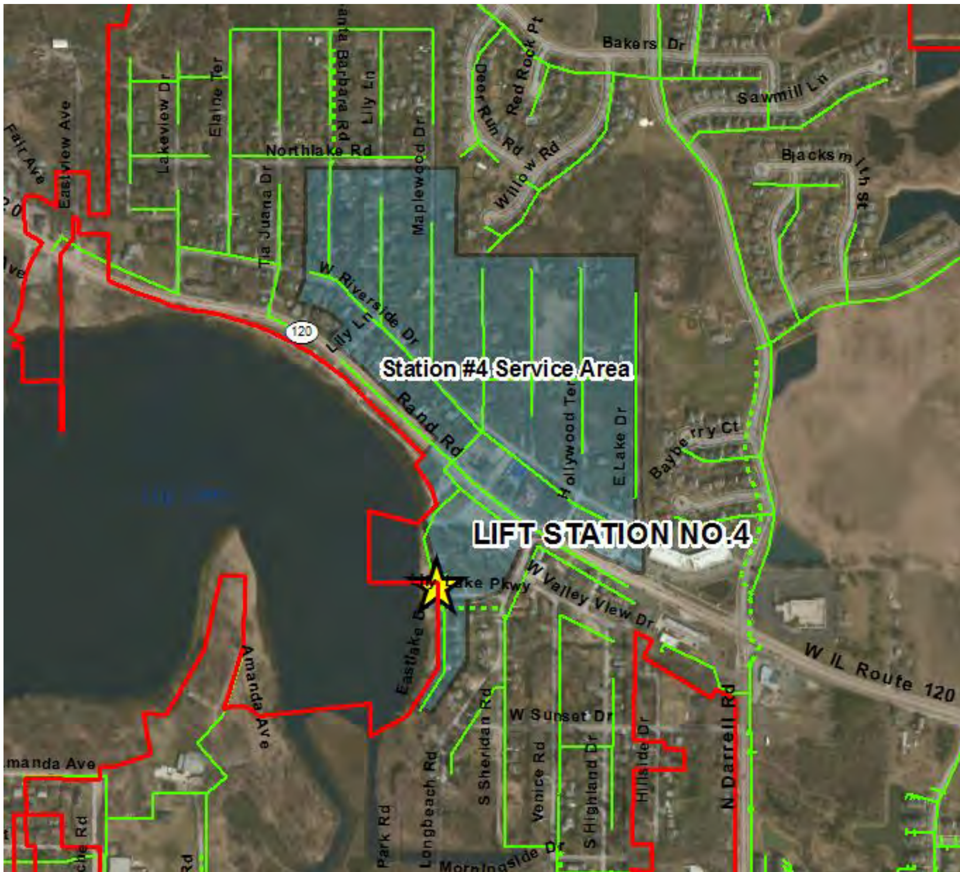
Lakemoor Lift Station 4 is located at 102 S Lakeshore Drive in Lakemoor. It uses a duplex submersible pumping system, is equipped to accept a portable generator, and is equipped with a Sensaphone alarm system. The station was installed in 1978 and most recently rehabilitated in 2007.

Lakemoor Lift Station 4 serves around 55 acres of development in the Northwestern Drainage Basin as well as pumped flow from Lakemoor Lift Station 5, and discharges through 490 feet of 6-inch force main to manhole C5NE69.

Table 4-39: Lakemoor Lift Station 4 - Pump and Force Main Data

Lift Station	Install Date	Rehab Date	Pump Manufacturer	Pump Motors	Rated Capactiy (gpm)	Force Main Diameter
Lakemoor 4	1978	2007	KJI 4-inch SKE-5-4T-001	3 HP	270	6-inch

Exhibit 4-22: Lakemoor Lift Station 4 Service Area



Pump Run Times and Drawdown Tests

Pump run times at Lakemoor Lift Station 4 indicate the pumps are sufficiently sized for the current flows. Pump 1 was observed to discharge very close to its rated capacity, while Pump 2 tested well below its rated capacity and outside acceptable tolerances. Reduced output from Pump 2 would result in increased run times which is opposite to that recorded. Because of this it is recommended that the Pump 2 drawdown test be verified.

Table 4-40: Lakemoor Lift Station 4 - Pump Drawdown Test Results

Pump	Avg Operation (hours/day)	Rated Capacity (gpm)	Test Pumping Rate (gpm)	Tolerance from Rated Capacity
1	2.70	270	278	3%
2	2.11	270	214	-21%

Impact of Future Development

Lakemoor Lift Station 4 was designed to serve the neighborhood and commercial development that encompasses its service area and tributary flow from Lakemoor Lift Station 5. At this time there are no plans for additional areas to be served by this station.

4.3.20 Lakemoor Lift Station 5

General Description

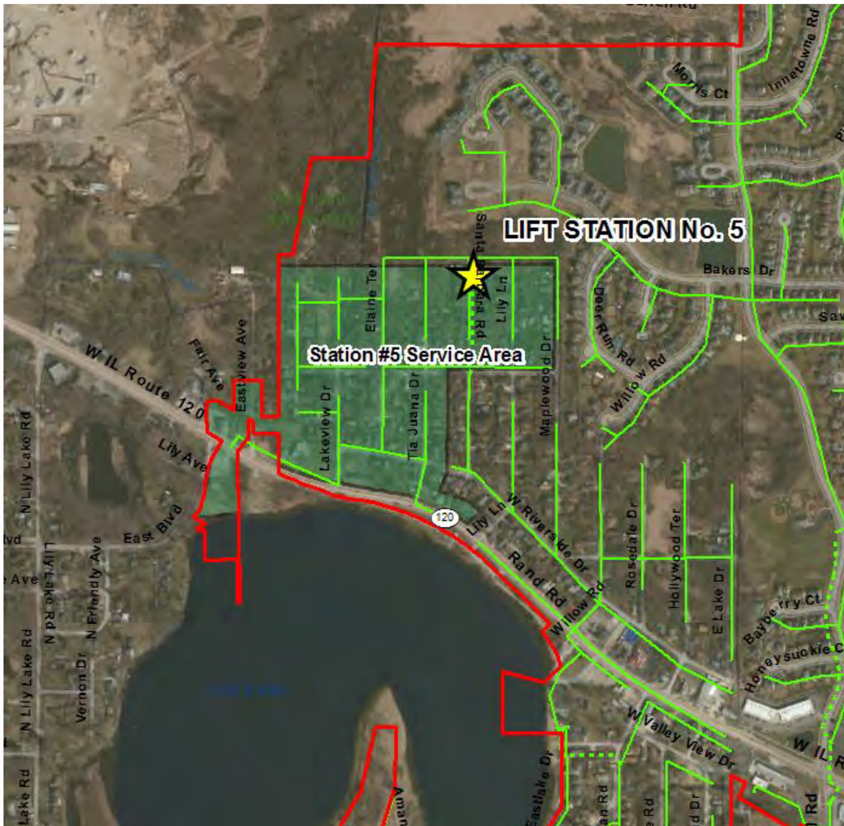
Lakemoor Lift Station 5 is located at 532 Santa Barbara Lane in Lakemoor. The lift station was installed in 1978 and most recently rehabilitated in 2007. It uses a duplex submersible pumping system, is equipped to accept a portable generator, and is equipped with a Sensaphone alarm system.

Lakemoor Lift Station 5 serves around 45 acres of development in the Northwestern Drainage Basin and discharges through 535 feet of 4-inch force main to manhole C4SE24.

Table 4-41: Lakemoor Lift Station 5 - Pump and Force Main Data

Lift Station	Install Date	Rehab Date	Pump Manufacturer	Pump Motors	Rated Capactiy (gpm)	Force Main Diameter
Lakemoor 5	1978	2007	KJI 4-inch SKE-5-4T-001	3 HP	200	4-inch

Exhibit 4-23: Lakemoor Lift Station 5 Service Area



Pump Run Times and Drawdown Tests

Pump run times at Lakemoor Lift Station 5 indicate the pumps are sufficiently sized for the current flows. However, both pumps were tested to similarly output significantly less than their rated capacity and outside acceptable tolerances. This is indicative that any clogging is affecting both pumps and would thus be located in the force main. On the other hand, these pumps may have simply been under-designed. It is recommended that the drawdown tests be verified, and that hydraulic computations be performed for this system.

Table 4-42: Lakemoor Lift Station 5 - Pump Drawdown Test Results

Pump	Avg Operation (hours/day)	Rated Capacity (gpm)	Test Pumping Rate (gpm)	Tolerance from Rated Capacity
1	1.48	200	167	-17%
2	1.42	200	163	-19%

Impact of Future Development

Lakemoor Lift Station 5 was designed to serve the neighborhood and commercial development that encompasses its service area. At this time there are no plans for additional areas to be served by this station.

4.3.21 Lakemoor Lift Station 6

General Description

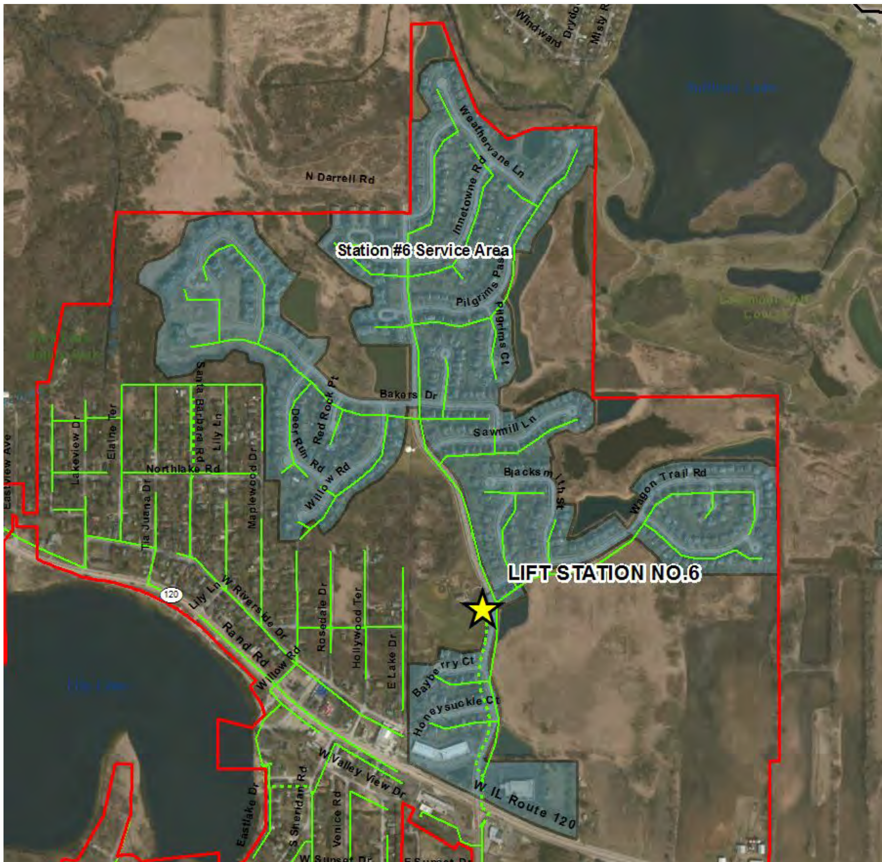
Lakemoor Lift Station 6 is located at 32250 Darrell Road in Lakemoor at the corner of Darrell Road and Wagon Trail. It uses a duplex submersible pumping system, is equipped to connect a portable generator, and has a Sensaphone alarm system.

The lift station serves around 160 acres of development in both the Northwestern and Northeastern Drainage Basins and discharges through 1,470 feet of 6-inch force main to manhole D5NW41.

Table 4-43: Lakemoor Lift Station 6 - Pump and Force Main Data

Lift Station	Install Date	Pump Manufacturer	Pump Motors	Rated Capacity (gpm)	Force Main Diameter
Lakemoor 6	2000	(manufacturer undocumented)	15 HP	502	6-inch

Exhibit 4-24: Lakemoor Lift Station 6 Service Area



Pump Run Times and Drawdown Tests

Pump run times at Lakemoor Lift Station 6 indicate the pumps are sufficiently sized for the current flows, although Pump 2 is beginning to run excessively. However, both pumps tested outside acceptable tolerances, with Pump 1 discharging above its rated capacity and Pump 2 operating well below rated capacity. In either case, it is recommended that the drawdown tests be verified, and that hydraulic computations be performed for this system.

Table 4-44: Lakemoor Lift Station 6 - Pump Drawdown Test Results

Pump	Avg Hours/Day of Operation	Rated Flow (gpm)	Test Flow Rate (gpm)	Tolerance from Rated Capacity
1	1.45	502	583	16%
2	4.04	502	305	-39%

Impact of Future Development

Lakemoor Lift Station 6 was designed to serve the neighborhood and commercial development that encompasses its service area. At this time there are no plans for additional areas to be served by this station.

4.3.22 Lakemoor Lift Station 7

General Description

Lakemoor Lift Station 7 is located at 127 South Drive in Lakemoor. It uses a duplex submersible pumping system and has room to accommodate the installation of a third pump.

This lift station is equipped with a permanent natural gas generator for auxiliary power, as well as a Sensaphone alarm system.

Lakemoor Lift Station 7 serves around 75 acres of development as well as the entire Northeastern Drainage Basin via Lakemoor Lift Station 6, and discharges south through a 12-inch force main to manhole C8NW56 in the 24-inch Route 176 West Interceptor. The capability of adding a third pump will be crucial in the evaluation of future flow from the Northeastern Basin which is proposed to be diverted to the future Darrell Road Collection System.

Table 4-45: Lakemoor Lift Station 7 - Pump and Force Main Data

Lift Station	Install Date	Pump Manufacturer	Pump Motors	Rated Capacity (gpm)	Force Main Diameter
Lakemoor 7	1997	Hydromatic (model undocumented)	25 HP	800	12-inch

Exhibit 4-25: Lakemoor Lift Station 7 Service Area

Pump Run Times and Drawdown Tests

Pump run times at Lakemoor Lift Station 7 indicate the pumps are sufficiently sized for the current flows. However, both pumps tested outside but close to acceptable tolerances, with Pump 1 discharging above its rated capacity and Pump 2 operating well below rated capacity. In either case, it is recommended that the drawdown tests be verified, and that hydraulic computations be performed for this system.

Table 4-46: Lakemoor Lift Station 7 - Pump Drawdown Test Results

Pump	Avg Operation (hours/day)	Rated Capacity (gpm)	Test Pumping Rate (gpm)	Tolerance from Rated Capacity
No. 1	1.82	800	869	9%
No. 2	2.19	800	704	-12%

Impact of Future Development

Future growth will have a significant impact on Lakemoor Lift Station 7. These impacts are discussed at length in Section 4.4.

4.4 LAKEMOOR LIFT STATIONS 1 AND 7

Lakemoor Lift Stations 1 and 7 are unique from the other 20 existing lift stations in the NMWRD collection system because:

- Each pumps through long (parallel) force mains extending from the Village of Lakemoor south along Lily Lake and River Roads all of the way to the Village of Island Lake.
- Each will receive a significant increase in flow over time as the Northwestern and Northeastern Basins develop.
- Each discharges to the 24-inch Route 176 West Interceptor which, while having sufficient capacity to convey current peak flows, will become overloaded as growth occurs in the Waterford, Central, Northeastern and Northwestern Drainage Basins.

One of the benefits of constructing the proposed Darrell Road Collection System (see Section 3) is that the pumped flow from Lakemoor Lift Station 7 would be redirected to the upstream end of the Darrell Road Interceptor. This reconfiguration would vacate the existing 12-inch force main on Lily Lake and River Roads, and in that way free up capacity in the existing pipe to support continued growth in the northwest.

Table 4-47 shows that the existing pumps at Lakemoor Lift Stations 1 and 7 have sufficient firm capacity for the current conditions, but both are projected to become overloaded by growth. Larger pumps would be required to pump the increased flow.

Table 4-47: Lakemoor Lift Stations 1 and 7 - Current and Future Pumping Requirements

Lift Station	Tributary PE	Average Flow (MGD)	Peak Flow (gpm)	Current Firm Capacity (gpm)	Surplus (Deficiency) (gpm)	Surplus (Deficiency) (PE)
Current Conditions						
Lift Station 1	1,618	0.16	411	450	39	**170**
Lift Station 7	1,912	0.19	478	800	322	**1,480**
Projected Conditions without Darrell Road Collection System						
Lift Station 1	7,243	0.72	1,555	450	(1,105)	**(5,455)**
Lift Station 7	14,587	1.46	2,826	800	(2,026)	**(11,195)**

Lakemoor Lift Station 1 currently has surplus capacity to connect an additional 170 PE, equivalent to 62 residential units at 2.74 PE/unit.

Lakemoor Lift Station 7 currently has surplus capacity to connect an additional 1,480 PE, equivalent to 540 residential units at 2.74 PE/unit.

In addition, the wet well at Lakemoor Lift Station 7 has space to add a third pump. If two 800 gpm pumps operating in parallel in a triplex arrangement were capable of pumping 1,200 gpm, the capacity gained would allow for a total of 3,460 PE to be connected to the lift station.

The ability to expand either Lakemoor Lift Station 1 or 7 is limited by the capabilities of their associated force mains. Table 4-48 summarizes conditions within the existing parallel 8-inch and 12-inch force mains along Lily Lake and River Roads associated with Lakemoor Lift Stations 1 and 7, respectively.

Table 4-48: Lakemoor Lift Stations 1 and 7 – Force Main Hydraulics

Force Main	Tributary PE	Peak Flow (gpm)	Diameter (inch)	Velocity (fps)	Length (feet)	System Head (feet)	Required Hydraulic Hosepower (hp)
Current Conditions							
LS 1 Force Main	1,618	411	8	2.62	16,500	78	8
LS 7 Force Main	1,912	478	12	1.36	19,000	34	4
Projected Conditions without Darrell Road Collection System							
LS 1 Force Main	7,243	1,555	8	9.93	16,500	805	316
LS 7 Force Main	14,587	2,826	12	8.02	19,000	405	289

Solids handling wastewater pumps are only available for total pumping heads of approximately 200 to 220 feet maximum. Because of the high flows and velocities within, and the relatively long lengths of these pipelines, it can be seen that projected system heads at ultimate build-out will be far in excess that which can be handled at a single lift station within each force main. Intermediate booster lift stations would be required.

If higher capacity, higher head pumping units were installed and the electrical service and standby power equipment upsized accordingly, the capacity of Lift Station 1 (pumping through the existing 8-inch force main) could be increased to as high as 1,025 gpm before a booster station would be required, which would allow for the connection of an additional 2,870 PE (1,047 residential units) in the Northwestern Basin. However, as discussed in Section 3 growth in the Northwestern Basin would flow directly to proposed Lift Station A which would pump directly into the existing force main(s), and not impacting Lift Station 1.

On the other hand, if the pumps and power service were increased at Lift Station 7 (pumping through the existing 12-inch force main), the capacity could be increased to as high as 2,000 gpm before a booster station would be required, which would allow for the connection of an additional 4,335 PE (1,582 residential units) in the Northeastern Basin.

It is important to note that the downstream 24-inch Route 176 West Interceptor also limits the degree to which Lakemoor Lift Stations 1 and 7 can be expanded. The interceptor currently has excess capacity of approximately 4,425 PE. Any combination of increased pumping capacity at the lift stations to serve the Northeastern and Northwestern Basins, along with growth in the Central and Waterford Basins, must not exceed this downstream limiting criteria.

If the Northwestern and Northeastern Basins were to be solely served with pumping service in the current manner, the District would need to either install additional parallel force mains or install new booster lift stations along the route of the existing force mains. In the case of Lift Station 1, three new booster stations would be required, while two booster stations would be required for Lift Station 7. While booster pumping stations could be used to provide increased pumping capacity, these booster stations are not recommended because of the high velocities (energy costs increase exponentially with velocity) and increased operation and maintenance requirements.

Table 4-49 illustrates the impact of constructing the proposed Darrell Road Collection System on the existing parallel 8-inch and 12-inch force mains along Lily Lake and River Roads. With the force mains interconnected, pumped flow will take the path of least resistance such that the head loss throughout the length of each force main is identical, in this case 77 percent carried by the 12-inch force main. Table 4-49 shows that the combined force mains are sufficient to convey the entire flow from the Northwestern Basin.

Table 4-49: Lakemoor Lift Station 1 and 7 – Force Main Load Sharing

Force Main	Tributary PE	Peak Flow (gpm)	Diameter (inch)	Velocity (fps)	Length (feet)	System Head (feet)	Required Hydraulic Horsepower (hp)
Current Conditions							
LS 1 Force Main	1,618	411	8	2.62	16,500	78	8
LS 7 Force Main	1,912	478	12	1.36	19,000	34	4
Projected Conditions with Darrell Road Collection System							
LS 1 Force Main	1,675	424	8	2.71	16,500	82	9
LS 7 Force Main	5,568	1,238	12	3.51	16,500	82	26

The timing for construction of the Darrell Road Collection System is driven by the limitations of the existing conveyance systems associated with Lakemoor Lift Stations 1 and 7. It was shown above that the existing parallel force mains will limit growth and eventually necessitate the off-loading of the force mains. Since both force mains discharge to the 24-inch Route 176 West Interceptor, limitations on available capacity in that interceptor will also eventually necessitate diversion of flow from the Northeastern Basin to the Darrell Road Collection System.

The existing systems will reach capacity when any combination of growth in the Northwestern, Northeastern, Waterford, or Central Basins reaches 4,425 PE (1,615 residential units).

4.5 MUTTON CREEK LIFT STATION

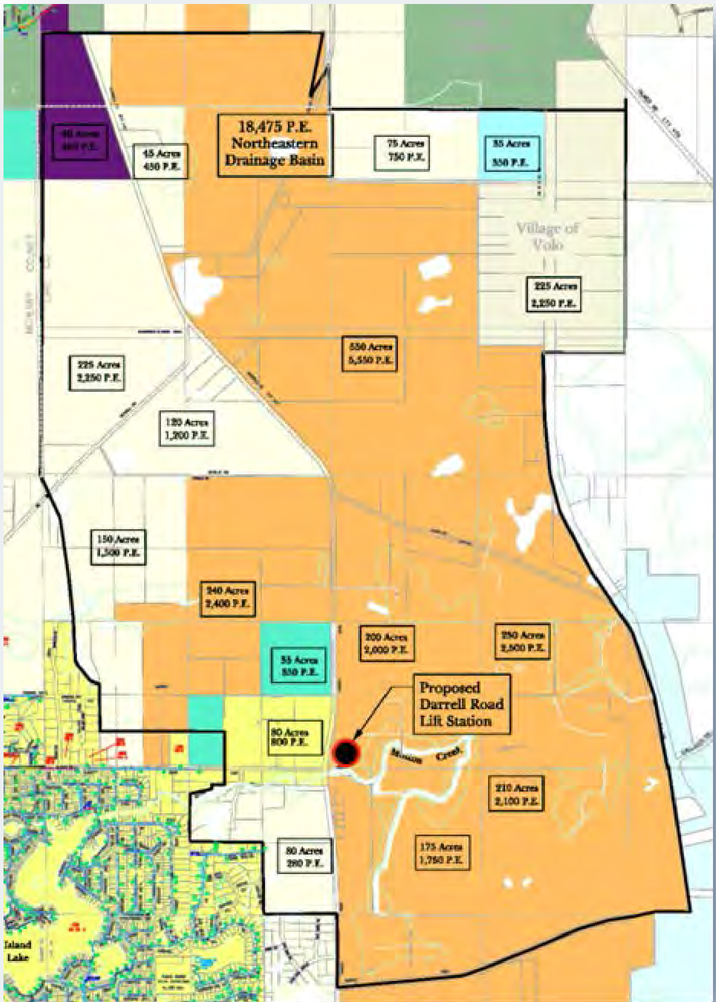
It has been shown that continued development of the Eastern and Northeastern Drainage Basins will eventually necessitate the construction of the Darrell Road Collection System, which will include a new regional lift station along Darrell Road north of Mutton Creek (see Section 3).

The proposed Mutton Creek Lift Station would enable the District to serve properties along Darrell Road north of Bonner Road to the Village of Volo. This installation would also serve the entire Northeastern Drainage Basin.

This new regional lift station would be installed after the construction of Phase I and Phase II of the Darrell Road Collection System, including the Mutton Creek Force Main and the downstream reach of the Darrell Road Interceptor Sewer. It is envisioned that the lift station design would initially include a duplex pumping system with provisions for the future installation of two additional pumps to accommodate increased future flows.

The Mutton Creek Lift Station and its force main are encompassed in Phases 2A and 2B of the Darrell Road Collection System. Probable capital costs for constructing this lift station and the all other components of the proposed collection system were presented in Section 3.

Exhibit 4-26: Mutton Creek Lift Station Service Area



4.6 CONCLUSIONS AND RECOMMENDATIONS

4.6.1 Rehabilitation

Northern Moraine WRD owns and operates 22 lift stations throughout the District. They have been constructed over the past thirty years and are generally in good condition. It is recommended that the District reinvest approximately $324,800 per year in replacement and upgrades towards these lift stations.

4.6.2 Construction of Regional Lift Stations

Lakemoor Lift Stations 1 and 7 currently have surplus capacity to convey additional flow from Lakemoor. These lift stations could be improved to increase capacity through the installation of larger pumping units and upsizing of power services. However, the degree to which these pump stations can be expanded is limited by the capacity of the downstream 24-inch Route 176 West Interceptor. At that time, the Darrell Road Collection System will be required unless parallel force mains or booster lift stations are constructed.

It is highly recommended that the District adhere to its policy of not allowing individual developers to install lift stations sized only to serve his individual property. Lift stations should be constructed regionally rather than locally, which will enable District Staff to maintain and monitor these installations more practically.

The proposed Mutton Creek Lift Station is to be constructed only after the second phase of the Darrell Road Interceptor Sewer has been constructed as detailed in Section 3. This installation will allow for the development of the Darrell Road Corridor, along with the remainder of the Eastern and Northeastern Drainage Basins.

4.6.3 Drawdown Testing

The District should conduct annual pump drawdown testing at all of its lift stations to determine and monitor the performance of the pumps. Provisions to attach pressure gauges during the pump drawdown tests would allow pressures to be recorded during the drawdown tests which would provide more meaningful results.

4.6.4 SCADA and Controls

Generally, the controls for the District's lift stations are antiquated and should be updated with modern technology. Future lift stations should be equipped with pressure transducers, float backups and PLC-based controls. The lift stations should also be integrated into a future SCADA system to allow for off-site monitoring of the stations.

4.6.5 Security Features

The existing lift stations should include features to secure them from entry by unauthorized personnel and to minimize the risk of vandalism. This would entail that all the lift stations and controls are enclosed in either a fence or within a building and have exterior lighting. Vandalism has been minimal, but in the event that it becomes a problem, the District can add motion sensors, cameras or intrusion alarms to the lift stations as needed.

SECTION 5

WASTEWATER TREATMENT FACILITY

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5. WASTEWATER TREATMENT FACILITY

5.1 General Background and Expansion History

The Northern Moraine WRD serves the Villages of Lakemoor, Island Lake, and Port Barrington. The District owns and operates the collection system and wastewater treatment facility that serves these communities. The facility discharges to the Fox River through a 30-inch outfall that extends from the facility through an existing wetland to the Fox River.

The original NMWRD wastewater treatment facility was constructed in 1978. The facilities included a raw sewage pump station, control building, garage, two package treatment plants, and drying beds. The biological processes included contact stabilization in conjunction with aerobic digestion. Biosolids were dewatered utilizing the drying beds and applied to agricultural ground.

NMWRD Treatment Facility

In 1991, new chlorination/dechlorination facilities were constructed to comply with NPDES permit revisions. In 1992, additional upgrades included raw sewage flow metering and replacement of diffusers within the biological process and aerobic digesters. In 1993, the District replaced the original comminuter (grinder) with a small mechanical fine screen. In 1997 the blowers for the contact stabilization package plants were replaced.

The District completed a Facility Plan Amendment in April 1998. At that time, the District was serving roughly 10,000 PE. The recommendations of that amendment were based on a projected 20-year design capacity of 30,000 PE. The recommended plan included a phased expansion of the treatment facility. The Phase I improvements expanded the facility's design capacity from 1.2 MGD to 2.0 MGD. It was intended that Phase II would increase capacity to 3.0 MGD.

The Phase I Expansion was completed in 1999. The improvements included installation of a second mechanical fine screen, replacement of the raw sewage pumps, and construction of a two-ring oxidation ditch, two new final clarifiers, a sludge dewatering building, and a chemical feed building. The existing package treatment plants were converted to provide for aerobic digestion and sludge storage.

In 2004, the communities within the facility planning area were experiencing an unprecedented rate of development. In response to the area's growing needs, the District completed a Facility Plan Update which included a thorough review of each community's comprehensive plan and land use plan. Based on the needs demonstrated in those comprehensive plans, the District developed a phased expansion plan to increase the treatment facility's capacity from 2.0 MGD to 10.0 MGD.

The District considered several alternatives for rehabilitation/expansion of the aerobic digestion system. The Phase I Expansion completed in 1998 set precedence for the future expansion of the facility. While that particular phased expansion plan only anticipated one additional phase to increase the treatment facility's capacity from 2.0 to 3.0 MGD, it was concluded that future phases should be designed to parallel the processes as much as practical. It was also agreed that the expansion from 2.0 MGD to 10.0 MGD should be completed in a minimum of four phases. The recommended phasing of future expansion was as follows:

- Phase I – 2.0 MGD - Completed 1998
- Phase II – 3.0 MGD
- Phase III – 4.5 MGD
- Phase IV – 6.0 MGD
- Phase V – 10.0 MGD

Since the completion of the 2004 Facility Plan, the Northern Moraine WRD acquired the Village of Lakemoor collection system. In addition, several improvements were made to the treatment facility.

In 2013, the aerobic digesters were rehabilitated. The project included removal of the interior steel walls, replacement of the diffused aeration system, and minor modifications to waste activated sludge, decant, and digested sludge piping. The improvements also included aluminum covers to maintain temperature during winter operation. Under a separate project, a centrifuge was added to the dewatering facility, and two covers were installed over 5 of the 14 sludge drying beds to provide for dewatered sludge storage.

In 2014 a new influent screen was installed, along with a new blower for the aerobic digesters. Additional improvements included implementation of variable speed aeration capabilities to more precisely control dissolved oxygen (DO) levels in the biological treatment process.

Treatment Facility Laboratory

The current facility has an average flow design capacity of 2.0 MGD and consists of influent screening, raw sewage pumping, extended aeration/biological treatment, tertiary clarification, and chlorine disinfection. The waste activated sludge is aerobically digested, mechanically thickened, dewatered, and land applied as fertilizer.

Influent and effluent flows and loadings at the NMWRD treatment facility are monitored by flow meters, wastewater sampling units, and on-site laboratory testing.

5.2 NPDES EFFLUENT LIMITS

Effluent water quality requirements as contained in the Northern Moraine WRD NPDES discharge permit are summarized in Table 5-1. The permit includes limits for the existing 2.0 MGD facility and also for the proposed 3.0 MGD treatment facility. The phosphorus limits do not take effect for 54 months; alternatives for compliance are discussed in Section 6.

Table 5-1: Current NPDES Effluent Limits

Parameter	Phase I (Existing)		Phase II (Proposed)	
Flow				
Design Average Flow	2.0	MGD	3.0	MGD
Design Maximum Flow	5.0	MGD	6.0	MGD
BOD_5				
Annual Average	-		10	mg/L
	-		250	lbs/day
Monthly Average	20	mg/L	20	mg/L
	334	lbs/day	500	lbs/day
Weekly Average	40	mg/L	40	mg/L
	667	lbs/day	1001	lbs/day
Suspended Solids				
Annual Average	-		12	mg/L
	-		300	lbs/day
Monthly Average	25	mg/L	25	mg/L
	417	lbs/day	600	lbs/day
Weekly Average	45	mg/L	45	mg/L
	751	lbs/day	1126	lbs/day
Fecal coliform				
Monthly Geometric Mean	200 per 100	ml	200 per 100	ml
Chlorine residual				
Daily maximum	0.05	mg/L	0.05	mg/L
pH				
Range	6 - 9	s.u	6 - 9	s.u.
Ammonia nitrogen				
April thru October				
Daily Maximum	2.5	mg/L	2.5	mg/L
Monthly Average	1.5	mg/L	1.5	mg/L
November thru February				
Daily Maximum	4.9	mg/L	4.9	mg/L
Monthly Average	3.7	mg/L	3.7	mg/L
March				
Daily Maximum	3.6	mg/L	3.6	mg/L
Monthly Average	1.5	mg/L	1.5	mg/L
Phosphorus				
Annual Average	1.0	mg/L	1.0	mg/L
	17	lbs/day	25	lbs/day

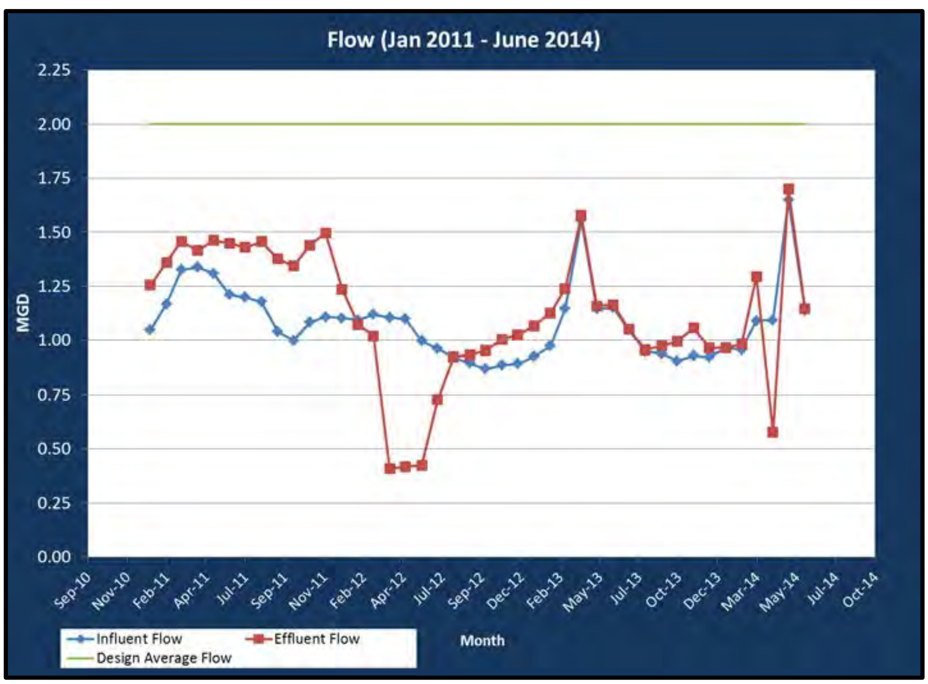
5.3 INFLUENT AND EFFLUENT WASTEWATER FLOWS

The Illinois EPA determines a treatment facility's remaining hydraulic capacity based on the average of the three low flow months. Influent and effluent flows at the NMWRD treatment facility for the years 2011 through 2013 are listed in Table 5-2. Monthly influent and effluent flows are shown on Figure 5-1, which shows that average monthly flows from January 2011 to June 2014 were well below the treatment facility's design capacity of 2.0 MGD.

Table 5-2: Treatment Facility Influent and Effluent Flows

Year	3-month Average Low Flow (MGD)	Months Occurring	Annual Average Flow (MGD)	Peak Month Flow (MGD)
Influent Flows				
2011	1.03	January / September / October	1.17	1.34
2012	0.88	October / November / December	1.00	1.12
2013	0.92	January / October / December	1.05	1.56
Effluent Flows				
2011	1.32	January / February / October	1.41	1.50
2012	0.42 (1)	April / May / June	0.85	1.24
2013	0.97	August / September / December	1.11	1.58
(1) Inaccurate 3-month low flow data due to effluent flow meter issues.				

Figure 5-1: Monthly Average Influent and Effluent Flows



The recorded treatment facility influent and effluent flows have not always been consistent with each other, although this phenomena has improved over the past few years. It would be expected that the influent flow would typically be higher than the effluent flow because waste activated sludge is removed from the system, however Figure 5-1 shows the opposite has typically been the case.

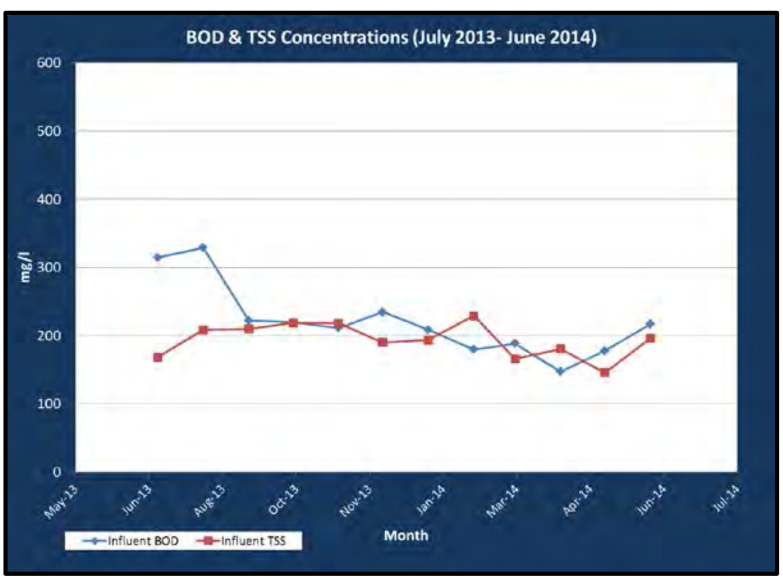
The other explanation for the deviations between influent and effluent flow recordings are the inherent inaccuracy of the flow meters. A review of the existing 18-inch influent magnetic flow meter design and sizing indicates that insufficient velocities exist at reduced pumping speeds and flows to produce highly accurate flow recording; this meter would be properly sized at 12-inch.

The effluent Parshall flume flow meter was also reviewed. There have at times been obvious issues with the flume's flow recordings, as can be seen on Figure 5-1. The existing 18-inch throat flume has a maximum capacity of 15.9 MGD, well in excess of current peak flow rates. The flume inlet configuration is also less than ideal with sharp edges and no means of ensuring normal subcritical flow entering the flume under low flow conditions. Nesting of a smaller flume inside the existing 18-inch flume would provide some opportunity for improving flume inlet conditions (if treatment facility hydraulics allow). The maximum capacity of a 9-inch flume is 5.7 MGD which would be insufficient for the Phase II peak flows. However, a 12-inch nested flume would be capable of measuring flows up to 10.4 MGD. The nested flume would be removed for the Phase IV (6.0 MGD) Expansion. Little else can be done to improve the accuracy of the flume other than to regularly calibrate and maintain the level measuring electronics.

5.4 INFLUENT WASTEWATER CHARACTERISTICS

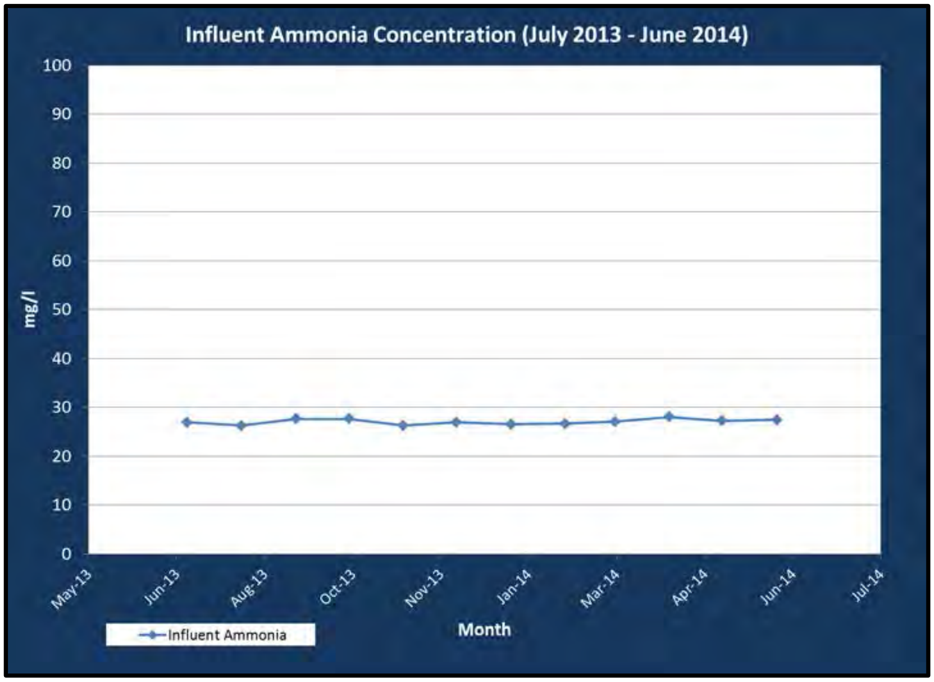
Monthly average concentrations of BOD_5 and TSS in the influent flow between July 2013 and June 2014 are shown on Figure 5-2. The average influent BOD concentration during this period was 221 mg/L, while the average influent TSS concentration was 194 mg/L.

Figure 5-2: Influent BOD_5 and TSS Concentrations



Monthly average influent ammonia concentration between July 2013 and June 2014 is shown on Figure 5-3. The average ammonia concentration for the year was approximately 26 mg/L or 217 lbs/day or roughly 54 percent of design (417 lbs/day).

Figure 5-3: Influent Ammonia Concentrations



Current influent flows and loadings as compared to the existing facility's design are summarized in Table 5-3. In general, the treatment facility is on average operating at or slightly above half capacity.

Table 5-3: Influent Wastewater Flows and Loadings (2013-2014)

Condition	Population Equivalent PE	Average Flow (MGD)	Per Capita Flow (gcd)	BOD_5 (lbs/day)	Per Capita BOD_5 (lbs/PE/d)	TSS (lbs/day)	Per Capita TSS (lbs/PE/d)	NH_3-N (lbs/day)
Current	13,695	1.05	77	1,793	0.13	1,595	0.12	227
Design	20,000	2.00	100	2,800	0.14	3,370	0.17	417
Percent of Design	68%	53%	77%	64%	93%	47%	71%	54%

5.5 EFFLUENT WATER QUALITY

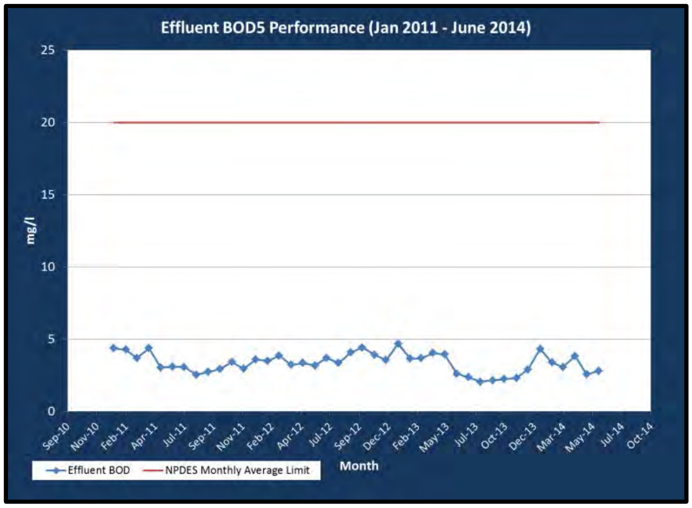
Effluent data indicates that the NMWRD treatment facility consistently produced an effluent water quality in compliance with the effluent limits set forth in the NPDES permit over the entire 42 month evaluation period from January 2011 through June 2014.

Monthly average BOD_5 concentrations in the treatment facility effluent are shown along with the current NPDES effluent limit on Figure 5-4.

The existing biological process has provided continuous BOD_5 reduction to meet the NPDES Permit standards. The monthly average effluent CBOD concentration ranged from 2.04 mg/L to a maximum of 4.68 mg/L with an average effluent concentration 3.35 mg/L. With an average influent concentration of 221 mg/L the treatment facility is removing 98.5 percent of the influent BOD_5.

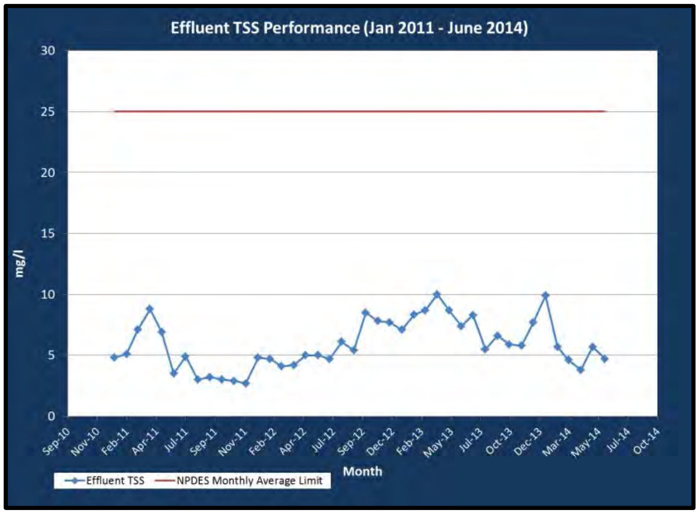
The monthly average limit for CBOD in the NPDES permit is 20 mg/L. Historically the treatment facility effluent has easily met this criteria, which is not surprising since the facility is designed to nitrify (remove ammonia).

Figure 5-4: Effluent BOD Concentrations



The treatment facility's performance for suspended solids removal has been equally as effective. Monthly average TSS concentrations in the treatment facility effluent are shown along with the current NPDES effluent limit on Figure 5-5.

Figure 5-5: Effluent TSS Concentrations



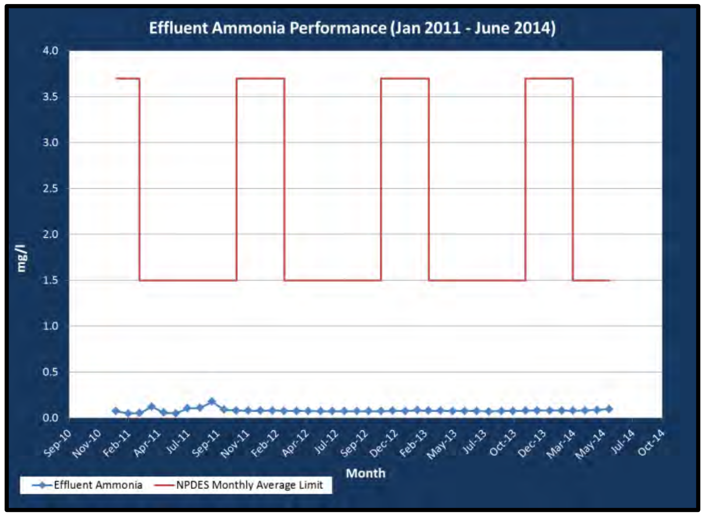
The monthly average effluent Suspended Solids concentration ranged from 2.70 mg/L to 10.0 mg/L with an average concentration of 5.95 mg/L. With an average influent concentration of 194 mg/L the treatment facility is removing 97 percent of the influent TSS.

The monthly average limit for TSS in the NPDES permit is 25 mg/L. Historically the treatment facility effluent has easily met this criteria, which is not surprising because the secondary clarifiers are lightly loaded under the current conditions.

Monthly average ammonia nitrogen concentrations are shown along with the current NPDES seasonal effluent limits on Figure 5-6. The average effluent ammonia over the 2011-2014 period was 0.08 mg/l. With an average influent concentration of 26 mg/L the treatment facility is removing 99.7 percent of the influent NH_3-N.

The current effluent ammonia limits vary by season as shown on Figure 5-6. Historically the treatment facility effluent has easily met these criteria, which is not surprising because the treatment facility currently operates well below it design capabilities.

Figure 5-6: Effluent NH_3-N Concentrations



5.6 PROJECTED WASTEWATER FLOWS AND POLLUTANT LOADS

Population equivalent and wastewater flow projections were discussed in Section 2. Projected flows and loadings at build-out of the incorporated areas, for the 20-year planning period, and at complete build-out of the Northern Moraine FPA, are summarized in Table 5-4.

Table 5-4: Projected Influent Wastewater Flows and Loadings

Condition	Projected PE	Average Flow (MGD)	Peak Flow (MGD)	BOD_5 (lbs/day)	TSS (lbs/day)	NH_3-N (lbs/day)
Design	20,000	2.0	5.0	2,800	3,370	417
Projected Conditions at Build-out of NMWRD Incorporated Areas						
Projected	16,338	1.3	3.6	2,242	2,124	282
Percent of Design	82%	66%	72%	80%	63%	68%
Projected 20-year Conditions within NMWRD FPA – HIGH RATE GROWTH						
Projected	27,921	2.47	6.20	4,211	4,440	524
Percent of Design	140%	124%	124%	150%	132%	126%
Projected 20-year Conditions within NMWRD FPA – LOW RATE GROWTH						
Projected	19,641	1.64	4.36	2,804	2,784	351
Percent of Design	98%	82%	87%	100%	83%	84%
Projected Ultimate Conditions at Build-out of Northern Moraine FPA						
Projected	56,460	5.3	11.8	9,063	10,148	1,125
Percent of Design	282%	266%	236%	324%	301%	270%

5.7 HEADWORKS

Fine Screens

The Headworks includes two mechanical fine screens installed in parallel channels adjacent to the Operations Building and over the raw sewage wet well.

A 31-inch diameter rotating screen was installed in 1993 and had a peak hydraulic capacity of 2.0 MGD. This screen was replaced in 2014 with a 36-inch fine screen (3 mm openings).

The 40-inch diameter rotating fine screen was installed in 1999 during the Phase I Expansion and has peak hydraulic capacity of 4.8 MGD. This screen is nearly 16 years old, the District should plan to replace the screen within 5 years.

The older of the fine screens is designed to remove particles greater than ¼-inch while the newer fine screen removes particles larger than 3 mm.

Design Data

Pertinent design criteria related to the Headworks are listed below.

Number of channels	2 each
Capacity	6.8 MGD +
Number of Mechanical Fine Screens	2 each
Screen openings on 40-inch Fine Screen	¼-inch
Screen Openings on 36-inch Fine Screen	3 mm

Performance and Deficiencies

The District had in the past experienced operational issues related to freezing of the screen components. A shelter was constructed to protect the screens from the elements. Otherwise the fine screens have provided reliable and effective screenings removal.

The newly installed 36-inch screen has a service life of 20 years. The 40-inch screen is showing its age and at most has a remaining service life of approximately 5 years and is currently used as a standby unit. Replacement of this screen should be included in capital improvements planning.

The peak hourly design flow for the Phase I Expansion was 5.0 MGD. The projected 20-year peak flow is expected to vary between 4.36 MGD and 6.20 MGD for low and high rate growth, respectively, as shown in Table 5-4. As such, replacement of the 40-inch screen in kind will provide sufficient hydraulic capacity for the currently projected future conditions.

There are no major operational issues with the fine screens, but regular spraying of the screen should be done to remove any clogs.

5.8 RAW SEWAGE PUMPING

The screened influent wastewater flow is discharged from the Headworks channel and enters the raw sewage pump station wet well. The basement of the operations building includes a drywell with four centrifugal pumps, which were installed during the Phase I Expansion.

Raw Sewage Pumps

Two of the four pumps are equipped with a variable frequency drive to provide flow-pacing capabilities. The maximum capacity of the lift station with the largest pump out of service is 5.0 MGD.

Design Data

Pertinent design criteria related to the Raw Sewage Pumping Station are listed below.

Number of pumping units	4 each
Type	Centrifugal Solids Handling
Rated Pump Capacity, each	4 @ 1,160 gpm
Firm Pumping Capacity	3,500 gpm
Force Main Diameter	12-inch and 18-inch

Performance and Deficiencies

The existing Raw Sewage Pumping Station has adequate capacity to meet both the current and projected future peak hydraulic flows.

Since the Phase I Expansion was completed, only two of the four pumps have been regularly used. The remaining two pumps are exercised on a weekly basis to ensure proper operation, and occasionally during extreme wet weather events. This mode of operation is due to the fact that variable speed controls have only been provided for the two lead pumps.

The two lead raw sewage pumps have a remaining service life of approximately 5 years. The two lag pumps likely have a remaining service life of about 15 years due to their inactivity.

The replacement of the two lead pumps should be included in capital improvements planning. The District should also plan to install VFD’s on the two standby pumps in order to more equally wear the pumps.

5.9 BIOLOGICAL PROCESS

The NMWRD treatment facility employs a single stage nitrification suspended growth biological treatment process. The single stage nitrification process is a version of activated sludge that creates an environment to promote BOD reduction and nitrification of ammonia within the same process. The environment promotes the growth of microorganisms, which convert the influent BOD and nutrients to energy and biomass. The process can be expanded to incorporate biological phosphorus removal and denitrification (conversion of nitrite & nitrate to nitrogen gas) by constructing additional cells and increasing detention time.

The existing aeration tanks include a two-ring oxidation ditch having a volume split of 60 percent in the outer ring and 40 percent in the inner ring. Mechanical surface disc aerators are used to aerate the mixed liquor. The oxidation ditch design allows for the aeration channels to be operated in series or parallel mode.

The phasing plan described in the 1998 Facility Plan proposed the construction of a third ring to increase the facility capacity from 2.0 MGD to 3.0 MGD. **The 20-year flow projections provide for a future average flow of between 1.64 MGD and 2.47 MGD for low and high rate growth, respectively. As such, the construction of the third ring is not anticipated to be necessary for the foreseeable future except under the most optimistic of projections.**

Design Data

Pertinent design criteria related to the biological process (oxidation ditch) are listed below.

Number of Units		2-channels
Design		Oxidation Ditch
Design Average Flow	2.0	MGD
Peak Hourly Flow	5.0	MGD
BOD_5	168	mg/L
	2,800	lbs/day
TSS	202	mg/L
	3,370	lbs/day
NH_3-N	25	mg/L
	210	lbs/day
Number of channels	2	each
Side Water Depth	14	feet
Channel Width	20	feet
Detention Time	16.1	hours
Total Volume	1,346,000	gallons
Solid Production	2,000	lbs/day
Solids Inventory	45,000	lbs
Sludge Age	22	days
Oxygen Required	342	lbs/hr
Oxygen Supplied	450	lbs/hr

Performance and Deficiencies

The process was designed in accordance with the Illinois EPA Design Standards for ammonia removal, which limits the applied BOD loading to 15 lbs BOD per 1,000 ft^3 of aeration tank volume per day. The process has performed exceptionally well and is resilient to changes in MLSS concentration as well as shock loading from the collection system.

The latest improvements to the oxidation ditch were implemented in 2014 and included the addition of four new VFD's to provide variable speed control of the surface disc aerators, and two new dissolved oxygen (DO) sensors.

2-ring Orbal Oxidation Ditch

The original design of the facility anticipated lower BOD_5 and ammonia concentrations than the influent concentrations currently received by the District. The actual concentrations are closer to the peak concentrations contemplated during design. This is attributed to the relatively low, non-excessive levels of I/I in the NMWRD collection system. As the treatment facility approaches design loadings, the disc aerators will need to be operated at increased submergence to meet the increasing oxygen demand.

The design of the oxidation ditch provides a great deal of flexibility. The existing 18-inch raw sewage influent pipe is equipped with sluice gates to direct flow to either or both channels. The existing 14-inch RAS piping is similarly equipped with sluice gates to allow RAS to be returned to either or both channels. Mixed liquor is withdrawn from the oxidation ditch through a 30-inch pipe which also is equipped with sluice gates allowing flow to be drawn from either or both channels. A splitter box located in the center island distributes flow evenly from the biological process to the final clarifiers.

The two oxidation ditch channels are connected with a 24-inch wall mounted sluice gate. This configuration allows the two channels to operate in series or parallel. The piping configuration allows each channel to be taken out of service for maintenance if necessary.

The maintenance requirements for the system are simple, requiring routine bearing lubrication and oil change in the drives. The moving parts operate at relatively low speeds and the expected life of the equipment is in excess of 20 years. The aerators were rehabilitated in 2010 including the replacement of the shaft bearings.

5.10 FINAL CLARIFIERS

The existing treatment facility includes two covered, 85-foot diameter clarifiers. The clarifiers were oversized to accommodate future expansion. The MLSS from the diversion structure is fed through the center pier into the influent center well. The suspended solids form a flock and begin settling to the floor of the clarifier. The clean water travels to the periphery of the clarifier and exits through the effluent weirs and launders. The settled sludge is collected from the bottom by a rotating collection header. The sludge is piped to the RAS/WAS Pump Station. Scum from the clarifier surface is collected on scum beaches and is also tributary to the RAS/WAS Pump Station.

Design Data

Pertinent design criteria related to the final clarifiers are listed below.

Number of clarifiers	2 each
Design	Circular
Average Flow	2.0 MGD
Peak Hourly Flow,	5.0 MGD
Diameter	85 feet
Sidewater Depth	12 feet
Surface Area, each	5,675 ft^2
Surface Area, total	11,350 ft^2
Weir Length, each	247 feet
Weir Length, total	494 feet
Surface Loading Rate at PHF	440 gpd/ft^2
Solids Loading Rate at PHF	21 $lbs/day/ft^2$
Weir Loading Rate	10,121 gal/day/ft

Performance and Deficiencies

After initially suffering from freezing during the winter, the final clarifiers have performed exceptionally since being covered in 2001. Effluent suspended solids concentrations have averaged 5.94 mg/L, which represents a removal efficiency of 97 percent.

The final clarifiers were rehabbed in 2010 and do not exhibit any pressing concerns.

Covered Final Clarifiers

5.11 CHLORINE CONTACT TANK

The chlorine contact tanks were constructed in 1999 as part of the Phase I Expansion. Chlorine is an oxidizing agent, which is commonly used to kill remaining micro-organisms in the effluent prior to discharge. The District converted the system in 2013 from a chlorine gas system to one that uses hypochlorite as the source of chlorine via a chemical feed system. Dechlorination is similarly accomplished by feeding bisulfate. To protect the environment, the NPDES Permit limits the concentration of chlorine in the final effluent. Dechlorination is accomplished by feeding bisulfate at the tail end of the contact tanks.

Design Data

Pertinent design criteria related to the chlorine contact tanks are listed below.

Number of Tanks	2 each
Channel Width	12 feet
Channel Length	56 feet
Sidewater Depth	8.5 feet
Total Volume	11,425 ft^3
Total Volume	85,460 gallons
Design Average Flow	2.0 MGD
Peak Hourly Flow	5.0 MGD
Detention Time @ Peak Flow	24.6 minutes

Performance & Deficiencies

The operational staff has stated that the disinfection facility meets their expectations. Maintenance is very low and the District has not received any violations since the facility was placed in service. The chlorine contact tank was constructed with sufficient volume to provide the required detention time for the Phase II Expansion to 3.0 MGD.

Both chemical feeds, both hypochlorite and bisulphate, are manually operated and are not flow-paced. By implementing flow-paced operation, the District could improve the efficiency of the existing disinfection process. Over the long term, the District may consider conversion of the process to ultraviolet disinfection.

Chlorine Contact Tanks

5.12 RAS/WAS PUMP STATION

Each final clarifier is equipped with a telescoping valve to withdraw return activated sludge (RAS). The telescoping valves are located in the withdrawal box attached to each clarifier, and can be lowered or raised to increase or decrease the RAS withdrawal rate, respectively. The 1998 design anticipated a RAS withdrawal rate at each clarifier ranging from 100 to 700 gpm. The RAS then flows by gravity to the RAS/WAS Pump Station.

The RAS/WAS Pump Station is a duplex submersible pump station, each pump having a rated capacity of 1,400 gpm. The pumps are installed in a 10-foot diameter precast wet well. Pump check and isolation valves are located in an adjacent precast vault. The RAS pumps are float controlled and operate in an on/off mode, pumping RAS through a 14-inch force main. Waste Activated Sludge (WAS) is withdrawn from the RAS force main via an 8-inch WAS force main, and is controlled by a manually operated valve located on the south side of the oxidation ditch.

Design Data

Pertinent design criteria related to the RAS/WAS Pump Station are listed below.

Number of RAS Pumps	2 each
Pump style	Submersible
Rated RAS Pump Capacity, each	1,400 gpm
RAS Force Main Diameter	14-inch
WAS Force Main Diameter	8-inch

Performance and Deficiencies

The RAS/WAS Pump Station operates reliably, although there are a few operational issues that could be addressed. The current pump station includes two constant speed pumps. Since the wet well volume between PUMP ON and PUMP OFF levels is only 1,400 gallons, at a RAS withdrawal rate of 700 gpm, a single 1,400 gpm pump would cycle every 4 minutes or 15 times per hour. Automatic alternation of the pumps reduces the starts per hour of each motor to less than 8 starts, which is below the recommended maximum of 10 starts per hour. Although the frequency of motor starts could be decreased further by raising the PUMP ON level, it is recommended that the District evaluate installing VFD's so that the pumps could more closely pace to the actual RAS flow rate. A level transducer would be required in the wet well to pace the pump speed, and further hydraulic computations need to be performed to properly assess the potential benefits and drawbacks to variable speed pumping.

Operational control could be enhanced by installing flow meters on the RAS and WAS force mains. The operational staff would be able to confirm the volume and pounds of solids wasted from the system. This feature would provide for improved control over the biological process as well as assist in managing the digestion process and solids disposal. In addition, the wasting procedure could be automated to provide enhanced control over the processes and lower the manual labor requirements of the operational staff.

5.13 SLUDGE TREATMENT AND HANDLING

During the Phase I Expansion, the packaged contact stabilization tanks were converted to serve as aerobic digesters. The contact tank in the north digester was converted to two gravity thickening tanks. The design calculations provided in the 1999 Facility Plan identify the volume of each gravity thickening tank as 5,150 cubic feet. Each gravity thickener was retrofitted with a telescoping valve for decant and a submersible thickened WAS transfer pump. The remaining compartments were converted to aerobic digestion and sludge holding.

Covered Aerobic Digester

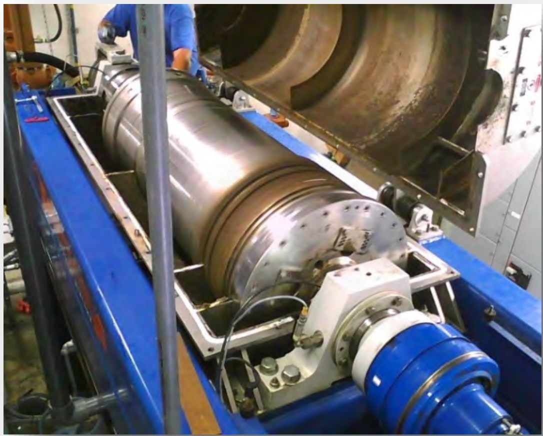
In 2013, the two 78-foot diameter aerobic digesters were rehabilitated. The project included removal of the interior steel walls, replacement of the diffused aeration system, and minor modifications to waste activated sludge, decant, and digested sludge piping. The improvements also included aluminum covers to maintain temperature during winter operation. Under a separate project, a centrifuge was added to the dewatering facility, and two covers were installed over 5 of the sludge drying beds to provide for dewatered sludge storage. A new blower was also installed for the aerobic digesters.

Sludge Dewatering Belt Filter Press

Sludge dewatering is accomplished using either a belt filter press or a centrifuge, both of which are located in the Sludge Thickening Building. The 1.5 meter belt filter press was installed in 1999 during the Phase I Expansion.

The centrifuge was installed in 2013. The centrifuge is capable of achieving both mechanical sludge thickening and dewatering.

Sludge Thickening / Dewatering Centrifuge



Dewatered sludge is conveyed to a small dump truck. The District typically stores the dewatered sludge on the adjacent drying beds and contracts for its disposal with a local land application firm. Two sludge storage covers were installed in 2012, covering 5 of the 14 drying beds. Additionally, the sludge pumping system was replaced in 2013 and has a service life of 12 years.

Progressing Cavity Sludge Transfer Pumps

Design Data

Pertinent design criteria related to the sludge handling facilities are listed in Table 5-5.

Performance and Deficiencies

The Illinois EPA standards require that the facility provide 4.5 cubic feet of digestion capacity per PE served, assuming a sludge solids concentration of 2%. In addition, Illinois EPA standards require 0.06 ft^3 of sludge storage per P.E. per day and require 150 days storage, unless alternative disposal options are provided.

Covered Sludge Drying Beds

Over the last several months of operation, operational staff has reported that the sludge stabilization process has been operating effectively. The installation of covers on the digesters has mitigated previous problems with limited biological activity during winter operations. Digestion time and temperature requirements are now being consistently met.

Table 5-5: Sludge Digestion, Dewatering, and Storage Computations

Description	Design Conditions	
Population Equivalent	20,000	PE
WAS Production		
BOD loading	3,370	lbs/day
Sludge yield	0.80	
WAS production	2,695	lbs/day
WAS solids concentration	2.0	%
WAS production	16,157	gal/day
Aerobic Digestion		
Number of units	2	each
Total volume	133,848	cu ft
	6.7	cu ft/PE
	1,001,183	gal
Sludge solids	2.0	%
Detention time	62	days
Belt Filter Press		
Number of units	1	each
BFP Capacity	100	gpm
Feed sludge solids	2.0	%
BFP Capacity	1,501	lbs/hr
BFP weekly operations	18.8	hr/wk
Centrifuge		
Number of units	1	each
Capacity	80	gpm
	1,109	lbs/hr
	23.5	hrs/wk
Dewatered Sludge Storage		
Number of drying beds	14	each
Drying bed volume	175	cu yd
Total volume provided	2,450	cu yd
Volume produced @ 15% solids	10.66	cu yd/day
Storage provided (sludge based)	230	days

5.14 FINDINGS AND RECOMMENDATIONS

The existing wastewater treatment facility consistently provides a permit-compliant quality effluent. In general, the treatment facility is very well maintained and efficiently operated. Some of the equipment is approaching the end of its useful service life and should be planned for replacement. Probable costs for those improvements are presented in Section 6.

5.14.1 Treatment Capacity

Current and projected wastewater flows and influent loadings are summarized in Table 5-6 for both build-out of the currently incorporated areas of the District and also for build-out throughout the entire Northern Moraine FPA.

As previously shown (Figure 2-15), it is projected that the design capacity will be reached within the 20-year planning period for the most optimistic growth projections, but that under the CMAP lower growth projections the treatment facility will not reach design capacity during the 20-year period. Planning for the Phase II Expansion of the NMWRD treatment facility should begin when the facility reaches 80 percent (1.6 MGD) of its hydraulic capacity based on the 3-month low average flow which corresponds to a population equivalent of 19,195 PE.

Table 5-6: Current and Projected Influent Wastewater Flows and Loadings

Condition	PE	Average Flow (MGD)	Per Capita Flow (gcd)	BOD_5 (lbs/day)	Per Capita BOD_5 (lbs/PE/d)	TSS (lbs/day)	Per Capita TSS (lbs/PE/d)	NH_3-N (lbs/day)
Design	20,000	2.0	100	2,800	0.14	3,370	0.17	417
Current Conditions (2013– 2014)								
Current	13,695	1.05	77	1,793	0.13	1,595	0.12	227
Percent of Design	68%	53%	77%	64%	93%	47%	71%	54%
Projected Conditions at Build-out of NMWRD Incorporated Area								
Projected	16,338	1.3	80	2,242	0.14	2,124	0.13	282
Percent of Design	82%	66%	80%	80%	100%	63%	76%	68%
Projected 20-year Conditions within NMWRD FPA – HIGH RATE GROWTH								
Projected	27,921	2.47	88	4,211	0.15	4,440	0.16	524
Percent of Design	140%	124%	88%	150%	107%	132%	94%	126%
Projected 20-year Conditions within NMWRD FPA – LOW RATE GROWTH								
Projected	19,641	1.64	4.36	2,804	0.14	2,784	0.14	351
Percent of Design	98%	82%	87%	100%	100%	83%	82%	84%
Projected Conditions at Build-out of NMWRD FPA								
Projected	56,460	5.3	94	9,063	0.16	10,148	0.18	1,125
Percent of Design	282%	266%	94%	324%	114%	301%	106%	270%

The existing treatment facility has sufficient capacity to support all projected growth within the current boundaries of the NMWRD incorporated areas. The current conditions indicate that BOD loading of the aeration tanks will likely be the limiting parameter as the population served continues to grow. The remaining 558 lbs BOD/day of available capacity will allow for an additional 3,282 PE (at 0.17 lbs/PE) to be connected to the NMWRD system.

The next major improvement at the NMWRD treatment facility will be necessitated by the changing regulatory environment, not by growth. The District's new NPDES permit requires that construction of the phosphorus removal systems be completed by May 1, 2018 (42 months from the effective date of the permit) and that full compliance with the phosphorus limit be achieved by May 1, 2019 (54 months from the effective date of the permit).

Any improvements made to the treatment facility should also consider the long-range capacity needs of the District. By making maximum use of existing treatment infrastructure as part of the phased improvements, the District will realize substantial savings.

There are some other immediate needs at the NMWRD treatment facility. Within the next five years it is recommended that the District plan on implementing the following improvements.

5.14.2 Phosphorus Removal

Northern Moraine's NPDES permit requires that a Phosphorus Removal Feasibility Report be submitted within 12 months of the effective date of the permit (by October 2015). An interim report is due by May 1, 2015. Plans and specifications are to be submitted by May 1, 2017 and construction is to be completed by May 1, 2018. Full compliance with the 1.0 mg/L phosphorus limit is required by May 1, 2019.

The phosphorus feasibility study must assess the method, time frame and costs for reducing phosphorus loadings to levels equivalent to monthly average discharges of 1 mg/L and 0.5 mg/L. Ultimate phosphorus limits may be significantly lower (0.075-0.10 mg/L) and will be based on the Fox River Study Group's findings as presented in their Implementation Plan that is due to be submitted to the IEPA by December 2015.

Initial improvements to meet the 0.5 mg/L to 1.0 mg/L range will likely include incorporation of chemical phosphorus removal and/or some degree of biological phosphorus removal. The removal of phosphorus will increase sludge production, but the computations in Table 5-4 suggest that sufficient sludge storage exists to store the additional phosphorus removal sludge. These options are evaluated on a cost-effective basis in Section 6.

Chemical phosphorus removal would comprise the construction of a new Chemical Feed Building to house a large chemical storage tank and the chemical feed pumping system. The storage tank would need to be contained in a recessed pit, would likely be made of polyethylene, and would be sized at approximately 6,000 gallons. Two smaller tanks could alternatively be implemented. The phosphorus removal chemical would be introduced to the mixed liquor flow in the center island withdrawal box upstream of the final clarifier weir gates.

If and when flows require the NMWRD treatment facility to be expanded from 2.0 MGD to 3.0 MGD capacity, biological phosphorus removal would be incorporated by constructing a third ring to the existing 2-ring oxidation ditch. In the meantime, biological phosphorus removal could be enhanced by constructing a separate anaerobic selector tank, along with installation of internal recycle pumps and piping.

5.14.3 Raw Sewage Pumping Station

The raw sewage pumps will be approaching the end of their useful lives within the next five years, and the District should plan for their replacement. Based on current flow projections, there is no need to upsize the pumps at this time. Therefore, power requirements will remain unchanged which will facilitate the pump replacement. It is recommended that either the replacement pumps be equipped with dry-pit immersible motors, or that the building be flood-proofed.

5.14.4 Influent Flow Meter

The existing influent magnetic flow meter is full line-sized at 18-inch diameter. Velocities through the meter at reduced pumping rates are well below recommended minimums which adversely impacts the accuracy of this flow meter. It is recommended that the meter be replaced with a 12-inch magnetic flow meter, and that a bypass around the meter be provided at that time.

5.14.5 Fine Screen

The existing 40-inch fine screen was installed in 1999 and is reaching the end of its useful life. It is also showing its age in the harsh environment. It is recommended that the District include replacement of the screen with a similarly sized screen in their 5-year planning horizon.

5.14.6 RAS/WAS Pumping Station

Both of RAS/WAS pumps were replaced in 2014. As discussed earlier, the existing constant speed RAS pumps cycle frequently. To reduce and more reasonably control pump motor wear, it is recommended that variable speed pumping capabilities be provided. This would involve the installation of VFD's and a level transducer in the wet well. It is also recommended that flow meters be provided for both the RAS and WAS flows.

5.14.7 Other Near Term Improvements

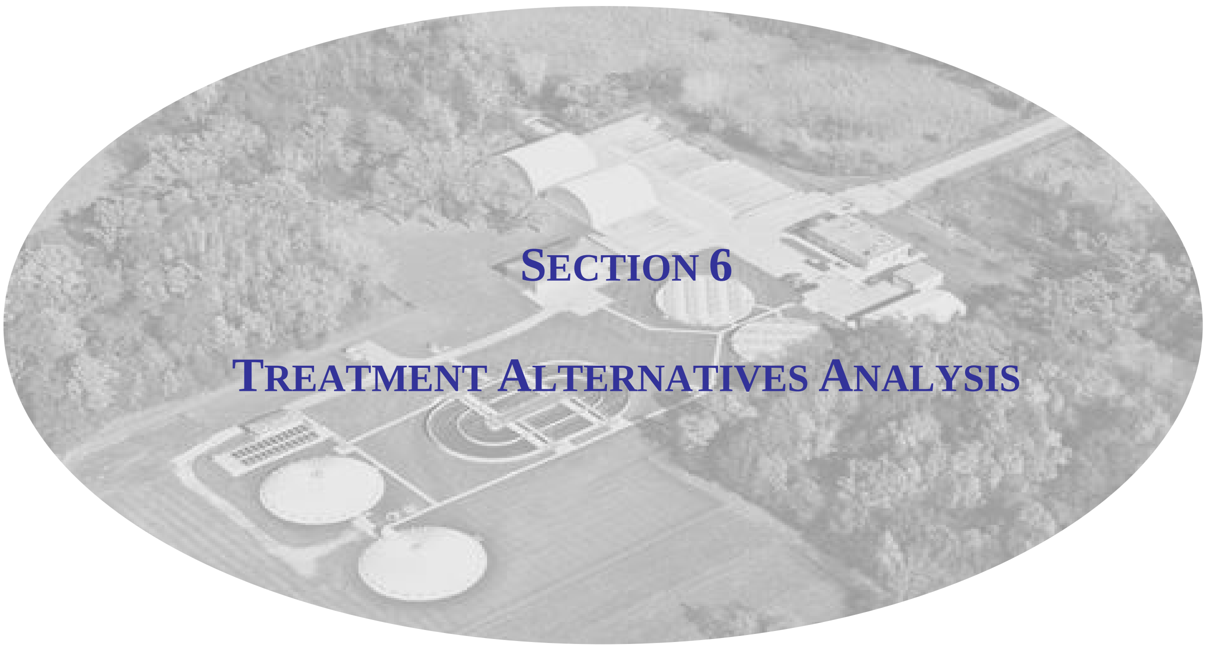
The various improvements described above are all within a 5-year planning horizon. Over the longer 10-year planning period the District should also plan on replacing the bearings on the oxidation ditch and aerator shafts if wear becomes evident. The final clarifier equipment should also be slated for replacement in the longer-term. These equipment replacements could possibly be delayed and made part of the Phase II Expansion.

5.14.8 Long Term Improvements

As the areas within the Northern Moraine FPA are developed, flows will increase and eventually the treatment facility will need to be expanded. It is projected that the existing NMWRD treatment facility will not reach capacity within the 20-year planning period except under the most optimistic high rate growth assumptions.

When influent flow requires expansion of the Northern Moraine treatment facility, it is envisioned that the expansion from 2.0 MGD to 3.0 MGD would comprise the addition of a third ring to the existing oxidation ditch, and upgrades to the aerobic digesters. Additional pumping capacity would also be required. The existing final clarifiers are oversized and should be capable of handling the peak flows associated with the Phase II Expansion to 3.0 MGD. The District may also desire to replace the existing chlorination/dechlorination disinfection process with ultraviolet disinfection.

The ultimate expansion of the NMWRD treatment facilities from 3.0 MGD to 6.0 MGD would be accomplished through the construction of a second oxidation ditch along with additional final clarifiers. Additional capacity would also be required throughout the treatment facility to increase the capacity of each individual process. This expansion is not anticipated to be necessary within the 20-year facilities planning period even under the most optimistic growth assumptions.



SECTION 6

TREATMENT ALTERNATIVES ANALYSIS

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6. EXPANSION ALTERNATIVES

6.1 GENERAL DISCUSSION

The last major upgrade to the NMWRD wastewater treatment facility was completed in 1998. The project addressed three major issues - rehabilitation, expansion needs, and regulatory requirements. The rehabilitation effort concentrated on equipment and structures that had reached the end of their design life. The expansion increased the treatment facility capacity from 1.2 MGD to 2.0 MGD to address anticipated growth within the community. The regulatory compliance required that the biological process be expanded to incorporate nitrification of ammonia.

Section 5 of this study provided an analysis of the wastewater treatment facility's operational performance, an assessment of the existing infrastructure, and prioritization for rehabilitation needs. The facility's operational performance has been within permit limits. The recommendations included a short list of items that are approaching the end of their service life. It was determined that these items could be addressed over the next 36 to 48 months as part of a comprehensive solution to the District's long-term needs.

The communities within the FPA have updated their comprehensive plans which identify land use and anticipated population density. Section 2 establishes population equivalents and future capacity requirements. The District anticipates that the facility will require capacity to treat 6.0 MGD at ultimate build-out of the FPA.

Although the natural environment is able to assimilate some pollution, the environment becomes unable to convert pollutants as the surrounding population continues to increase. This leads to degradation of the water quality and wildlife habitat. In order to ensure stability within the environment, governmental agencies on the federal, state, and local levels are continuously evaluating the effectiveness of wastewater regulations. The regulatory issues that will be addressed within this section are limited to nutrient removal and biosolids stabilization.

6.1.1 Regionalization

Sections 3 and 4 have demonstrated that extension of the collection system to serve each community is a practical solution. A second solution could include construction of remote treatment facilities. To date, the collection system and treatment facility have been constructed to provide for regionalization. As such, it was the original intent of the Water Quality Management Plan that only one treatment facility would be required to serve the Planning Area.

Regionalization provides both economic and non-economic benefits over satellite treatment facilities. The greatest benefit is simply due to economy of scale. All treatment facilities have the same basic physical, biological and chemical process requirements. The appropriate processes must be constructed at each facility and expanded as required. As a result, a regional treatment facility can typically be constructed at a lower price per gallon treated.

A second economic factor to be considered is the ability to phase construction. Smaller facilities are traditionally completed in few phases because their service area is limited. In comparison, a regional facility can be readily phased to accommodate the needs of growth as it occurs. The existing users should not be required to pay for expansion. As land within the Facility Planning Area is served, a connection fee is paid. The amount of the fee is applied to constructing additional capacity.

A third economic factor is the cost of operation of a regional facility. Fewer staff are required per million gallons treated, automation is more cost effective and supplies are bought at reduced bulk rates.

Some non-economic factors include:

- Effluent quality
- Limitation of impacts on surrounding land values
- Limitation of environmental impacts
- Retention of trained professional staff

The largest drawback to regionalization will be the cost of constructing the collection system extensions that serve development, because properties generally cannot be forced to develop in sequence with construction of the collection system.

6.1.2 Biological and Hydraulic Expansion

It is evident that the existing processes are performing well and meeting all current treatment requirements. Future expansions of the biological processes and hydraulic capacity of the facilities should provide continuity and maintain the reliability and simplicity of operation established in the first phase. Future considerations should be incorporated into earlier designs (such as the phosphorus removal design facilitating the addition of the third ring to the oxidation ditch) to minimize pumping requirements, maintain reliability, and ensure continued compliance with NPDES permit conditions and effluent limits.

6.1.3 Biosolids Stabilization

The biosolids or sludge produced in the treatment process must be stabilized prior to disposal. The District currently utilizes aerobic digestion to meet the Class B requirements for land application. The aerobic digesters will need to be upgraded when the NMWRD treatment facility is expanded from 2 MGD to 3 MGD.

6.1.4 Excess Flow Treatment

The USEPA and Illinois EPA are contemplating elimination of wet weather permit standards. Unlike most communities along the Fox River, the District is capable of providing secondary treatment to 100% of the influent flow even during wet weather periods. The phased improvements plan outlined herein will maintain the robust design of the treatment facilities with respect to the capacity to treat infiltration and inflow. In doing so, the District will be able to avoid future expenses related to compliance with this potential regulatory change.

6.1.5 Nutrient Removal

The Northern Moraine NPDES permit now includes provisions for phosphorus removal. The permit now also requires NMWRD to sample and measure total nitrogen in the effluent. The District should expect that total nitrogen removal will be required in the future.

Phosphorous removal can be accomplished through chemical precipitation or biologically by adding a step to the denitrification process. Each of these alternatives are evaluated later in this section and compared with the District's long-range biosolids stabilization plans, as each will produce a differing volume of sludge, have differing aeration requirements in the aerobic digesters, require differing amounts of electric energy, and require the construction of differing facilities.

Total Nitrogen can be removed by biological means through a denitrification process. Provisions for the denitrification process will need to be incorporated into the design of the expanded oxidation ditch process. This enhancement will address concerns regarding changes in the regulatory requirements with respect to this nutrient.

6.2 REHABILITATION OF EXISTING WASTEWATER TREATMENT FACILITY

Section 5 provided an overview of the existing wastewater treatment facility's strengths and limitations. The District's goal is to reuse as much of the existing infrastructure as practical and still meet the long-term needs of the community and the environment. In order to reuse the existing infrastructure, mechanical components will need to be replaced as they reach the end of their service life.

Table 6-1 summarizes probable capital costs associated with the near-term equipment replacements discussed in Section 5.

Table 6-1: Probable Capital Costs – Treatment Facility Rehabilitation

Item	Probable Cost
GENERAL CONDITIONS	$ 93,750
RAW SEWAGE PUMP REPLACEMENT	250,000
INFLUENT FORCE MAIN AND METER REPLACEMENT	100,000
FINE SCREEN REPLACEMENT	250,000
SUBTOTAL CONSTRUCTION	$ 693,750
CONTINGENCY @ 25%	173,440
CONSTRUCTION TOTAL	$ 867,190
ENGINEERING @ 15%	130,100
PROBABLE CAPITAL COST – TREATMENT FACILITY REHABILITATION	$ 997,290

6.3 REGULATORY ISSUES AND ANALYSIS

6.3.1 Nutrient Removal Criteria

The NMWRD facility discharges to the Fox River. According to the Illinois EPA Clean Water Act Section 303(d) List, the Fox River does not meet water quality standards for its intended use in the majority of the segments, including the segments immediately downstream of the treatment facility. The impairment on the river for aquatic life is based on a low dissolved oxygen concentration. This low dissolved oxygen content is due to algal growth and exacerbated by the presence of pools upstream of the low head dams along the river.

In 2001, the Illinois EPA was contemplating performing a Total Maximum Daily Load (TMDL) study on the Fox River in an attempt to address the impairment. At that time, there was insufficient data available to support a TMDL. Therefore, modeling would simply be an exercise which would not reflect actual environmental conditions. Many of the communities along the Fox River (including the District) joined forces with other stakeholders, including Friends of the Fox and Sierra Club, to form the Fox River Study Group (FRSG). The FRSG determined that it was in the best interest of all the stakeholders if a comprehensive solution was developed through a model that could be calibrated based on extensive river monitoring data. The FRSG, in concert with the POTWs along the river, have monitored the river for numerous constituents including phosphorus, nitrogen, fecal coliform and chlorophyll a. This water quality data provided the basis for development of QUAL2K and HSPF models.

In 2004, the Illinois EPA implemented statewide nutrient removal criteria for wastewater treatment facilities that were proposing expansion of their hydraulic capacity. Two nutrients of concern were total nitrogen and phosphorus. The NPDES Permits issued for these facilities typically contained an interim phosphorus limit of 1.0 mg/L and a requirement to monitor total nitrogen.

In 2011, the Illinois EPA was receiving increased pressure by the USEPA and environmental stakeholders to address nutrient criteria on all POTWs, not only treatment facilities undergoing expansion. Several NPDES permits along the Fox River had expired and were due to be reissued by the Illinois EPA. However, the Illinois EPA elected to delay reissuance so the NPDES permits could incorporate language agreed upon in ongoing discussions on nutrient criteria.

In January 2012, in an attempt to build consensus among all stakeholders, the Illinois EPA presented the FRSG with special conditions in draft form for nutrient criteria. The FRSG had not yet completed the low flow monitoring required to calibrate the HSPF and QUAL2K models. Therefore, determination of a water quality based phosphorus limit could not be determined at that time. The FRSG in conjunction with the Illinois EPA worked to develop a schedule for completion of the modeling effort and determination of water quality based phosphorus standards. During the drought in the summer of 2012, the FRSG was able to obtain low flow monitoring for the Fox River and is on schedule to present a calibrated model by May 2013.

In January 2013, the Illinois EPA and FRSG were able to agree on special conditions for all dischargers greater than 1.0 MGD. These conditions included a 1.0 mg/L interim phosphorus standard and a schedule for completion of the water quality modeling for the development of permanent phosphorus criteria. Ultimately, the permit language requires the FRSG to complete analysis of the alternatives and provide recommendations by December 2015. The permit also requires the POTWs to perform a feasibility study and determine the cost for compliance. It is the intent of the special conditions that all dischargers along the Fox River will meet the recommended standards by 2030.

The NMWRD has received its updated NPDES permit which includes special conditions related to the FRSG implementation plan. The special conditions require the District to complete a phosphorus removal feasibility report within 12 months of the permit issuance date. This study will evaluate options and costs associated with reducing phosphorus concentrations in the effluent to 1.0 mg/L as well as 0.5 mg/L. The District must complete the improvements to achieve 1.0 mg/L within 54 months of the issuance date. The compliance schedule contained in the Northern Moraine NPDES is summarized in Table 6-2.

Table 6-2: Phosphorus Removal Compliance Schedule

Item	Required date of completion	Completion Date
Interim Report on Phosphorus Removal Feasibility Report	6 months from the effective date of Permit	May 1, 2015
Phosphorus Removal Feasibility Report submitted	12 months from the effective date of Permit	November 1, 2015
Progress Report on FRSG Phosphorus Input Reductions and Implementation Plan	18 months from the effective date of Permit	May 1, 2016
Progress Report on Recommendations of FRSG Implementation Plane	24 months from the effective date of Permit	November 1, 2016
Plans and specifications submitted	30 months from the effective date of Permit	May 1, 2017
Progress Report on Construction	36 months from the effective date of Permit	November 1, 2017
Complete Construction	42 months from the effective date of Permit	May 1, 2018
Progress Report on Optimizing Treatment System	48 months from the effective date of Permit	November 1, 2018
Achieve Annual Concentration and Loading Effluent Limitations for Total Phosphorus	54 months from the effective date of Permit	May 1, 2019

6.3.2 Nitrogen Removal Alternatives

Nitrogen in wastewater can be found in several forms including ammonia (NH_3), ammonium (NH_4^+), nitrate (NO_3^-) and nitrite (NO_2^-). In the past, limits were placed only on the levels of ammonia discharged from wastewater treatment facilities since that is the only form of nitrogen that is toxic to aquatic life. Even though they do not directly harm fish, nitrates and nitrites can contribute to algal bloom. Phosphorous, in the form of phosphates (PO_4^-), can also trigger algal growth if it is present in high enough concentrations. Limiting phosphorus and total nitrogen, the sum of all forms of soluble nitrogen, helps to preserve ecosystems in the surrounding watershed.

The District's new permit includes unchanged ammonia limits and requires monitoring of total nitrogen. If the District is required to remove total nitrogen in the future, the biological process will need to be modified. The removal of nitrogen is effected through the biological oxidation of nitrogen from ammonia to nitrate (nitrification), followed by denitrification, the reduction of nitrate to nitrogen gas. Nitrogen gas is released to the atmosphere and thus removed from the water.

During the next round of NPDES permits (in approximately five years) the District can anticipate that the permit limits for ammonia will be reduced by roughly 80% due to a modification within the calculation for toxicity from flathead minnows to mollusk.

The existing biological process was designed for conversion of soluble bio-degradable organic contaminants and nutrients, specifically ammonia nitrogen. Most aerobic biological processes are designed for the development of beneficial bacteria that are able to convert organic compounds and are capable of performing this task within a very short amount time. However, the conversion of ammonia nitrogen to nitrite (nitrification) is accomplished by *Nitrosomonas* bacteria. In order to develop and maintain a sufficient population of *Nitrosomonas* bacteria within the bio-mass, the process must maintain a low feed to mass ratio, with typical values ranging from 0.08 to 0.12. Since the facility cannot control the influent food source, operators control the bio-mass (MLSS) within the basins. There is a practical limit to the concentration of MLSS ranging from 2,000 to 3,000 mg/l. Therefore, the basins must be constructed large enough to allow the operators to develop a bio-mass population that is 10 to 12 times greater than the incoming food (soluble BOD). The operators maintain the ratio of food to mass by wasting the proper amount of solids from the process. The *Nitrosomonas* bacteria convert ammonia to nitrite, while *Nitrospira*, which are also present, convert the nitrite to nitrate.

Denitrification is a step in the biological process in which microorganisms utilize the oxygen from nitrite and nitrate for respiration during metabolization of organic matter. Denitrification is an alternative to respiration and is initiated by incorporating a zone that is rich in soluble BOD and operates at a dramatically low dissolved oxygen concentration, an anoxic zone. This zone is typically near the beginning of the biological process were the soluble BOD is plentiful. However, in order to convert the nitrate to nitrogen gas it must be first converted from ammonia to nitrate, which typically is near the end of the biological process. Therefore, most designs incorporate an internal loop, which brings the nitrate rich mixed liquor into contact with the high strength soluble organic matter. The NMWRD facility has a two-ring oxidation ditch which subjects the microorganisms to both oxic and anoxic conditions in a continuous loop. Typically these facilities include a three-ring design which includes independent drives for the external ring and interior two rings. This third ring provides increased flexibility for dissolved oxygen control which facilitates better operational performance with respect to denitrification and biological phosphorus removal.

6.3.3 Phosphorus Removal Alternatives

Phosphorous removal in wastewater treatment facilities was common in the 1970’s. The most widespread method of phosphorous removal used at that time was the addition of chemical coagulants that cause phosphate compounds to settle out of solution. Phosphorous removal is possible through biological processes, but the amount of phosphorous that can be removed through such processes is limited.

All life forms utilize a food source and a source of oxidative potential, usually oxygen or nitrite, to absorb phosphates into their bodies as the molecule adenosine tri-phosphate (ATP). This process is known as metabolism. Phosphorous is released from ATP to provide energy for cellular growth and activities. When activated sludge is produced and collected, phosphates absorbed within the cells of microorganisms as ATP and other cellular components are removed from the wastewater flow. This is the basis for biological phosphorous removal, a small amount of which occurs in all activated sludge processes.

Greater amounts of phosphorous can be removed through biological methods by creating an anaerobic zone, in which no oxygen or nitrate is available, within a treatment facility's suspended biological growth processes. Most microorganisms are not capable of storing large amounts of ATP and rely on a constant rate of metabolism to maintain cellular activity. Certain microorganisms known as Polyphosphate Accumulating Organisms (PAOs) can store significantly more phosphorous than other heterotrophic bacteria. PAOs are capable of survival in an anaerobic environment absent of nitrate and oxygen. As such, the percentage of PAOs within the microbiological community increases when the process includes an anaerobic zone. The larger PAO population ensures a higher concentration of phosphorus within the sludge wasted from the process.

Biological phosphorus removal requires rigid operational control in order to maximize the efficiency of the process. The process is sensitive to changes in temperature, flow and feed concentration. Biological phosphorous removal may not be able to continuously meet the 1.0 mg/L effluent standard set by the IEPA. Therefore, chemical polishing capabilities would be incorporated into a biological phosphorus removal design.

It is important to note that the phosphorous captured in biological phosphorous removal (Bio-P) process is simply stored in the bodies of microorganisms and can easily be returned to solution. The high phosphorus sludge is wasted from the biological process to a sludge stabilization process. Sludge production from a Bio-P facility is expected to have a yield of 0.8 pounds of solids per pounds of BOD applied. Once stabilized, the sludge is then dewatered and disposed of through land application or land filling operations.

Chemical precipitation can be accomplished within either the primary or secondary treatment process. The District does not have primary treatment and therefore is limited to implementing chemical phosphorus removal in the secondary process. The District has several options for chemical selection. Lime addition is effective but produces a considerable amount of sludge. Alum and iron salts are more commonly recommended. The locally available iron salts include ferric chloride ($FeCl_3$) and ferrous sulfate ($FeSO_4$). Both are highly corrosive and should be stored in a separate, well-ventilated area.

For planning purposes, the influent phosphorus concentration was assumed to be 6.0 mg/L (actual influent phosphorus concentration is currently 6.2 mg/L). The chemical precipitation required for phosphorus removal is estimated to be one mole of iron (Fe) for one mole phosphorus (P). However, an additional one to five moles of iron is required to satisfy competing reactions, such as hydroxide formation. For purposes of this evaluation, the estimate is based on three moles (1 mole + 2 moles additional) per mole of phosphorus for secondary treatment. The calculations for ferric chloride ($FeCl_3$) addition at 35% solution strength are shown below.

<u>$FeCl_3$ Dosage for Phosphorous Removal</u>

Molecular Weight of PO_4 = 95 g/mole

Moles / Pound of PO_4 = 453 g/lb / 95 g/mole = 4.768 moles of PO_4/ pound

Molecular Weight of $FeCl_3$ = 164 g /mole

Moles / Pound of $FeCl_3$ = 453 g/lb / 164 g/mole = 2.78 moles of $FeCl_3$/ pound

Weight of $FeCl_3$ per gal of solution = 11.23 lb/gal x 35% = 3.93 lb of $FeCl_3$/ gal

3.93 lb of $FeCl_3$/ gal x 2.78 moles / pound = 10.9 moles of $FeCl_3$ per gallon

<u>Determine $FeCl_3$ dosage for Secondary Treatment (use 3 moles $FeCl_3$ / per mole PO_4)</u>

3 mol $FeCl_3$ per mole of PO_4 x 4.768 mol PO_4/ lb PO_4 = 14.304 mol $FeCl_3$ /lb PO_4

14.304 mol $FeCl_3$ / lb PO_4 / 10.9 mol $FeCl_3$ / gal $FeCl_3$ = 1.31 gals $FeCl_3$/ lb PO_4

Use 1.5 gallons of $FeCl_3$ per pound PO_4

The ferric chloride and ferric sulfate are commodities and the price is determined by supply and demand. Ferric chloride is a by-product of the steel industry. In the 1960's, 70's, and 80's, ferric chloride and ferric sulfate compounds used for phosphorus removal were abundant. The US steel industry has been in a significant production decline and the industry has begun to recycle iron salt compounds. The reemergence of phosphorus removal will result in an increased demand for the iron salts, while the current market trends indicate less will be produced. The current price of ferric chloride is roughly $1.00 per gallon delivered. Table 6-3 includes the chemical cost analysis performed assuming an influent concentration of 6 mg/L and a dosage requirement of one and one half gallons per pound for secondary treatment.

Table 6-3: Chemical Costs – Phosphorus Removal

Phase	DAF (MGD)	Phosphorus (lbs/day)	Phosphorus (lbs/year)	$FeCl_3$ (gallons/year)	Estimated Annual Cost
I	2.0	0	-	-	-
2017 P Removal	2.0	100	36,500	54,750	$ 54,750
II	3.0	150	54,750	82,125	$ 82,125
III	4.5	225	82,125	123,188	$ 123,188
IV	6.0	300	109,500	164,250	$ 164,250

The NPDES permit requires that the District assess the method, time frame and costs specific to the NMWRD treatment facility for reducing its loading of phosphorus to not only 1.0 mg/L but also to 0.5 mg/L. A ferric to phosphorus molar ratio of 3:1 was assumed during the evaluation of chemically removing phosphorus to 1.0 mg/L. This is a conservative assumption used for planning purposes; some treatment facilities successfully remove phosphorus chemically at a molar ratio of 1.5:1.

The required molar ratio to remove phosphorus below 1 mg/L to 0.5 mg/L is greater. The typical molar ratio for these low levels increases by 2 to 6 times that for less stringent requirements; for planning purposes a molar ratio 4 times greater was assumed, or 12:1. Instead of assuming 1.5 gallons of ferric chloride per pound phosphorus removed, it was assumed that 6.0 gallons of ferric chloride per pound phosphorus removed would be required. Therefore, the cost per pound of phosphorus removed will increase as the concentration is lowered from 1 mg/L to 0.5 mg/L.

It is recommended that jar testing be conducted on the NMWRD mixed liquor to clarify actual chemical requirements for phosphorus removal at the NMWRD treatment facility.

A Chemical-P removal system will typically yield 0.65 pounds of solids per pound of BOD applied plus the chemical sludge produced which is estimated to be 3.77 pounds of solids per pound of phosphorus removed. As stated previously, a Bio-P process will yield 0.8 pounds of solids per pound of BOD applied. Table 6-4 summarizes the estimated sludge production from the secondary process under three conditions – current design, chemical phosphorus removal and biological phosphorus removal.

Table 6-4: WAS Production – Chemical-P Removal versus Biological-P Removal

Phase	DAF (MGD)	WAS Production Without P Removal (lbs/day)	WAS Production With Chemical P Removal (lbs/day)	WAS Production Using Biological P Removal (lbs/day)
I	2.0	1,600	-	-
2017 P Removal	2.0	1,600	1,978	1,970
II	3.0	2,400	2,966	2,954
III	4.5	3,601	4,450	4,432
IV	6.0	4,801	5,933	5,909

For planning purposes, a present-value cost analysis was performed to compare the initial capital costs of each alternative, along with the annual chemical and energy costs, to determine the most cost-effective option for providing phosphorus removal at the NMWRD treatment facility.

Both chemical and biological phosphorus removal require the construction of a chemical feed building to house and pump Ferric Chloride (Bio-P requires chemical for polishing). Probable capital costs for the Chemical Feed Building are summarized in Table 6-5.

Table 6-5: Chemical Feed Building – Probable Capital Costs

Description	Total
CONSTRUCTION SUBTOTAL	$ 358,015
GENERAL CONDITIONS	44,750
CONTINGENCY @ 25%	100,690
CONSTRUCTION TOTAL	503,455
DESIGN ENGINEERING @ 7.5%	37,760
CONSTRUCTION ENGINEERING @ 7.5%	37,760
TOTAL CAPITAL COSTS	**$ 578,975**
General Conditions	
Bond & Insurance @ 2.5%	$ 8,950
Overhead and Profit @ 10%	35,800
TOTAL GENERAL CONDITIONS	**$ 44,750**
Chemical Feed Building	
Excavation	$ 23,990
Concrete	100,000
Precast Concrete	20,625
Masonry	63,500
Metals	6,400
Caulking	1,000
Insulation	9,125
Roofing	12,280
Sheet Metal	4,170
Hollow Metal Door & Finish Hardware	13,500
Louver	1,500
Painting	25,325
Piping & ID	10,000
Valves	1,500
HVAC	13,950
Equipment	38,650
Plumbing	12,500
CONSTRUCTION SUBTOTAL	**$ 358,015**

For chemical phosphorus removal, the operating costs that are pertinent to the analysis include the chemical costs, and the energy costs associated with the aerobic digesters. Based on a BOD sludge yield of 0.65, volatile solids at 72%, and an aeration blower energy cost per pound of volatile solids applied of $0.07/lb, the daily energy cost to operate the aerobic digester blowers was calculated to be $112 per day.

In addition, the cost for chemical usage was calculated assuming an influent phosphorus concentration of 6 mg/L (100 lbs/day at 2.0 MGD) and a molar ratio of 3:1, which results in 1.5 gallons $FeCl_3$ dosage per pound of phosphorus. At $1 per gallon of ferric chloride, this equates to $150 per day for phosphorus removal chemicals. The total daily cost for chemical phosphorus removal was in this manner calculated to be $262 per day (including chemical costs and digester blower energy costs), or an annual cost of $95,637.

For biological phosphorus removal, capital costs include the construction of an anaerobic selector tank to be included upstream of the existing oxidation ditch. The anaerobic selector tank must provide 1.5 hours of detention time at the design flow of 2.0 MGD, which equates to 150,000 gallons. A 35 foot square tank with a side water depth of 14 feet was assumed. Probable costs associated with the selector tank are summarized in Table 6-6.

Table 6-6: Biological Phosphorus Removal – Probable Capital Costs

Description	Total
CONSTRUCTION SUBTOTAL	$ 589,800
GENERAL CONDITIONS	73,725
CONTINGENCY @ 25%	165,880
CONSTRUCTION TOTAL	829,405
DESIGN ENGINEERING @ 7.5%	62,205
CONSTRUCTION ENGINEERING @ 7.5%	62,205
TOTAL CAPITAL COSTS	**$ 953,815**
General Conditions	
Bond & Insurance @ 2.5%	$ 14,745
Overhead and Profit @ 10%	58,980
TOTAL GENERAL CONDITIONS	**$ 73,725**
Bio-P Anaerobic Selector Tank	
Excavation and Haul/Fill	$ 102,500
Concrete Chamber	200,000
Aggregate Base	4,800
12" DIP RAS Piping	50,000
18" DIP Raw Sewage Piping	70,000
Handrail	7,500
20 HP Mixers (2)	80,000
Sluice Gates	75,000
CONSTRUCTION SUBTOTAL	**$ 589,800**

When the Phase II (3.0 MGD) Expansion is constructed, the level of biological phosphorus removal would be further enhanced by the construction of the third outer ring of the oxidation ditch, which would be designed to operate in an anoxic condition. When the Phase III Expansion is constructed, the selector tank will remain in service or re-commissioned as a junction chamber/splitter box.

The Bio-P system would also have daily operating costs. These include the energy cost for digester blowers, the energy cost for mixing, and the chemical cost required for polishing. Two 20 HP mixers were assumed to run over a 24-hour period. Multiplying this by a cost of $0.07 per kWh, the energy cost required for mixing was calculated to be $50/day.

The chemical cost required for polishing in the Bio-P process was calculating using an assumption that 1/12 of the solids produced from phosphorus removal would require polishing. Using the aforementioned chemical cost of $1.00 per gallon of $FeCl_3$, the polishing cost equates to $32 per day.

Finally, the blower requirements for aerobic digestion of volatile solids produced by the Bio-P process was calculated to cost $138 per day based on an assumed sludge yield of 0.80 for Bio-P, 72% volatile solids, and an energy cost of $0.07 per pound of BOD sludge applied to digestion. The total daily operating costs for Bio-P treatment amounts to $220 per day, or an annual cost of $80,080. Based on a 20-year planning period and a discount rate of 4%, the present value of these operating costs is calculated to be $1.09 million.

Table 6-7 summarizes the findings of the cost effective analysis for Chem-P and Bio-P. Annual operating costs are estimated to be lower for biological phosphorus removal than those associated with chemical phosphorus removal. However, due to the high initial capital cost associated with biological phosphorus removal, the overall life-cycle present value cost for biological phosphorus removal is approximately $0.65 million greater than that associated with chemical phosphorus removal. Therefore, it is recommended that a chemical phosphorus removal system be implemented into the NWRWRD facility for the 2016 Upgrades.

Table 6-7: Cost Effective Analysis – Phosphorus Removal

	Chemical P Removal	Biological P Removal
Capital Cost	$ 578,975	$ 1,532,790
Annual Costs:		
Chemicals	$ 54,750	$ 11,467
Blower Energy	40,887	50,323
Mixing Energy	-	18,291
Subtotal – Annual Costs	$ 95,637	$ 80,080
Discount Rate	4%	4%
Planning Period (years)	20	20
Present Value – Annual Costs	$ 1,299,740	$ 1,088,317
Total Present Value	$ 1,878,715	$ 2,621,107

6.4 EXPANSION PHASING

The most economical approach to expanding the NMWRD regional treatment facility will be to develop a phasing program that meets the Facility Planning Area's long-range needs, while being constructed in small enough increments to avoid unnecessary debt.

The potential build-out of the Facility Planning Area equates to 56,000 PE. The existing facility is designed to serve 20,000 PE. Some basic factors to consider are land requirements, treatment process configuration, and ease of expansion for existing unit processes. The most direct approach to addressing the issues is to provide a conceptual design for the ultimate treatment facility, and then to segregate the improvements into logical phases. Other regional treatment facilities where this approach has not been employed have resulted in side stream or parallel processes that are either difficult to manage and operate or have units and/or treatment trains that sit idle. The phasing should capitalize on the strengths of the existing treatment facility, by incorporating the oxidation ditch, final clarifiers and sludge dewatering facility.

The Phase I Expansion completed in 1998 set precedence for the future expansion of the facility. While that particular expansion-phasing plan only contemplated one additional phase increasing the capacity from 2.0 to 3.0 MGD, it was concluded that future phases should be designed to parallel the processes as much as practical.

The 2004 Facility Plan Update included four future phased expansion up to an ultimate treatment capacity of 10 MGD. This 2014 Facility Plan Update includes upgrades only up to an ultimate capacity of 6.0 MGD as shown in Table 6-8.

Table 6-8: Phased NMWRD Treatment Facility Expansion Plan - Design Flows

Construction Phase	2004 Facility Plan Design Average Flow (MGD)	2014 Facility Plan Design Average Flow (MGD)
Phase I (Existing)	2.0	2.0
2017 P Removal	n/a	2.0
Phase II	3.0	3.0
Phase III	4.5	4.5
Phase IV	6.0	6.0
Phase V	10.0	n/a

Prior to the Phase II Expansion, the District will be required to rehabilitate infrastructure and modify the treatment process to meet new regulatory requirements. The regulatory driven improvements do not include an expansion of treatment capacity, but rather the capability to remove phosphorus at the current design flows along with other miscellaneous treatment facility rehabilitation upgrades discussed in Section 5 and above in Section 6.3.

As demonstrated in Section 5, the treatment facility is currently operating at 53% of its hydraulic design, 64% of its BOD loading design, 47% of its TSS loading design, and 54% of its ammonia loading design. It is recommended that once the facility plan is approved that the District proceed with the 2017 P Removal (phosphorus removal) design to ensure adequate treatment capabilities are available for continued compliance with the updated NPDES permit. Phases II, III and IV will need to be considered once the facility reaches 80% of its design capacity of the preceding phase (currently 1.6 MGD). It is currently projected that the Phase II Expansion will not be required during the 20-year planning period using CMAP (low rate) growth projections as discussed in Section 2. Only under the most optimistic projections would the treatment facility need to be expanded during the 20-year planning period.

The design flow and loading parameters, and anticipated discharge requirements, for each phase are identified in Table 6-9.

Table 6-9: Phased Treatment Facility Expansion Design Parameters

Parameter	Phase I (Existing)	2017 P Removal	Phase II	Phase III	Phase IV
Population Equivalent, PE	20,000	20,000	30,000	45,000	53,000
Year	1998	2016	2035	2045	2055
Wastewater Flows					
Design Average Flow, MGD	2.0	2.0	3.0	4.5	6.0
Peaking Factor	2.5	2.5	2.5	2.3	2.2
Peak Hourly Flow, MGD	5.0	5.0	7.5	10.35	13.2
Design Loadings					
BOD_5 @ 205 mg/L, lbs/d (1)	2,800	2,800	5,130	7,695	10,260
TSS @ 240 mg/L, lbs/d (1)	3,370	3,370	6,005	9,007	12,010
NH_3-N @ 35 mg/L, lbs/d	417	417	876	938	1,250
Phosphorus @ 6 mg/L, lbs/d	n/a	100	150	225	300
Design Effluent Quality					
BOD_5, monthly avg, mg/L	20	20	20	20	10
TSS, monthly avg, m/L	25	25	25	25	12
NH_3-N, monthly avg, mg/L	1.5	1.5	1.5	1.5	1.5
NH_3-N, monthly avg, mg/L	3.0	3.0	3.0	3.0	3.0
TKN, monthly avg, mg/L	10.0	10.0	10.0	10.0	10.0
Phosphorus, monthly avg, mg/L	n/a	1.0	0.5	0.1	0.1
pH, continuous range	6 - 9	6 - 9	6 - 9	6 - 9	6 - 9
(1) BOD_5 and TSS concentrations for Phase II thru IV.					

Table 6-10 summarizes probable capital costs associated with the future Phase II (3.0 MGD) Expansion project. Detailed costs previously prepared as part of the 2004 Facility Plan Update were escalated at an annual rate of 3.5% to approximate 2015 dollars.

Table 6-10: Probable Capital Costs – Phase II Expansion

Description	2004 Probable Cost	2015 Probable Cost
Construction Sub-Total	$ 9,076,500	$ 13,252,300
General Conditions	1,134,700	1,600,000
Contingency (25%)	2,269,200	3,313,100
Probable Construction Cost	$ 12,480,400	$ 18,165,400
Design Engineering @ 7.5%	936,100	1,362,500
Construction Engineering @ 7.5%	936,100	1,362,500
TOTAL CAPITAL COST	**$ 14,352,600**	**$ 20,890,400**
General Conditions		
Bonds & Insurance @ 2.5%	$ 227,000	$ 320,100
Overhead & Profit @ 10%	907,700	1,279,900
TOTAL GENERAL CONDITIONS	**$ 1,134,700**	**$ 1,600,000**
Phase II Expansion (3.0 MGD)		
Site Work	$ 1,374,400	$ 2,006,700
Raw Sewage Pump Station	65,000	94,900
Headworks	722,200	1,054,500
Oxidation Ditch	1,138,200	1,661,800
RAS/WAS PS & Diversion Structure	903,700	1,319,500
Clarifiers	3,000	4,400
Aerobic Digestion	2,380,500	3,475,600
Sludge Storage Barn	702,400	1,025,600
Fleet Maintenance Garage	836,300	1,221,000
Operations Building	346,400	505,800
Administration Building	604,400	882,500
CONSTRUCTION SUBTOTAL	**$ 9,076,500**	**$ 13,252,300**

6.5 HEADWORKS

The District's Headworks has been expanded twice to date. The facility has a small screening channel that is fed from the 30-inch interceptor sewer. The screening channel is directly connected to the wet well, which is located underneath the channel.

The screening structure, with capacity to treat peak flows up to 6.8 MGD, has adequate capacity for the current peak design flow of 5.0 MGD. Screening capabilities will need to be increased when the Phase II Expansion is constructed, at which time the peak screening capacity will need to be increased to 7.5 MGD (ultimately 13.2 MGD at build-out of the Northern Moraine FPA). Similarly, the wet well is not sized to receive the future peak flow.

The future expansion of the Headworks building (wet well and screening facility) to the east has been discussed with operations staff at times over the past several years. The near-term needs of the District only require the replacement of the older of the two existing screens. The staff is very pleased with the performance of the existing Lakeside mechanical fine screens and would prefer to maintain similar units if practical. The screened material is conveyed to ground level by shaftless conveyors. The screens are now enclosed in a shielding structure that provides a controlled environment for maintenance and reduces winter operational issues.

The existing control building includes a spacious dry well for the raw sewage pumps and allows for easy maintenance of the equipment. The dry well design is very functional and consideration for continued use should be incorporated into the expansion plans. However, it is important to note that the finish floor of the Control Building is only 739. When this structure was originally constructed the Record Flood Elevation was 736.5 according to the 1977 Plans. The 1998 plans indicate the 100-Year flood elevation to be 738.4. In short, this structure could potentially flood. Replacement of the raw sewage pumps should consider either dry-pit immersible motors to prevent damage to the pumps, and to allow them to continue operating, should the dry well flood or flood-proofing of the building.

The usable height of the wet well is limited to five feet from high to low water level, which equates to roughly 4,200 gallons. If the screening facility is expanded further east, the wet well could be expanded similarly.

Some of the design challenges associated with expansion of the Screen Building would include:

- Completing the renovation without removing the existing facility from service.
- Improving downstream hydraulics at the screens.
- Removal of the concrete fillet in wet well.
- Balancing the hydraulic split between future screens.
- Space limitations within the existing property.

Construction of a new facility would provide the District with greater flexibility to incorporate additional treatment alternatives. Options discussed included septic receiving, screening washing, and grit removal.

A large portion of the NMWRD Facility Planning Area is currently served by septic tanks and private mechanical systems. The District has been approached on several occasions to treat septic waste. The District has elected not to accept the waste because it could upset the process without proper pre-treatment. If the septic receiving facility were constructed properly and the loads incorporated into the biological process design, the District could better serve the residents within the Facility Planning Area not currently served by sewers.

A septic receiving station would require convenient truck access. The preferred design would incorporate the septic receiving station into the headworks facility to minimize screenings handling and odor control. However, it is recognized that the receiving station could be constructed in a separate location and housed in its own building. This would require separate odor control, screening and grit facilities. The existing screens manufactured by Lakeside have served the District well. Lakeside also manufactures a package septage receiving station treatment unit that serves to remove grit and ground screenings as pretreatment. The ability to meter and sample hauled-in waste could also be incorporated into the facility.

The expansion of the existing headworks was the preferred alternative. It was determined that the structure should be expanded only once more to provide the most economical approach. The improvements will include construction of a new structure adjacent to the existing influent channel and wet well. The design of the structure allows it to be completed without any major shutdowns or extended by-pass pumping.

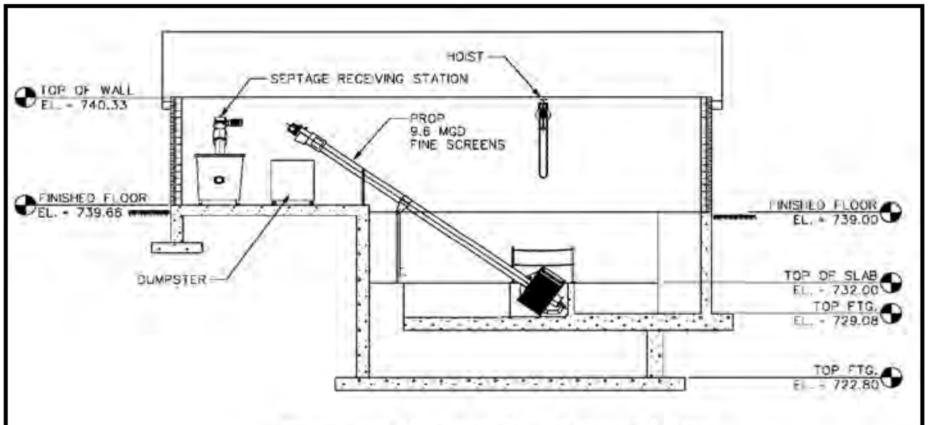
The proposed addition would be constructed at the same footing elevation and match the wall dimensions of the existing Control Building's foundation. Two new channels would be constructed sufficient to install the future Phase IV (9.6 MGD) screens. Each screen would be capable of treating in excess of 50% of the ultimate design flow (13.2 MGD). The existing channel would be maintained to allow for by-pass during times of maintenance.

The screened material would be conveyed from the channel to the upper level. The selected equipment is capable of compacting, dewatering and bagging the material in dumpsters to reduce the odors traditionally associated with the screening. Also the upper level would be equipped with a 400 gallon per minute septage receiving station. The selected unit is also capable of washing, compacting and bagging the waste material.

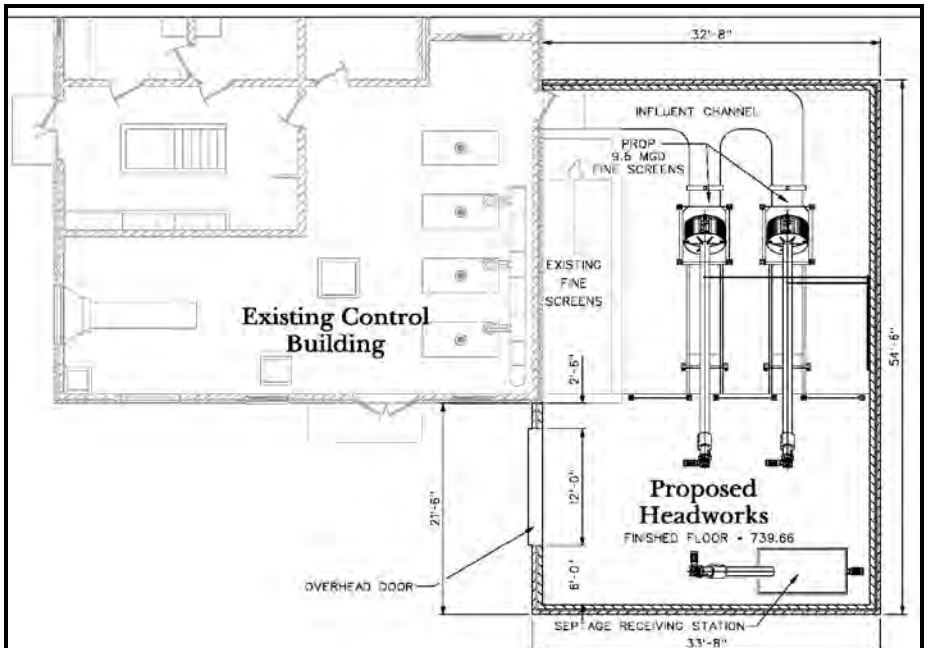
Extension of headworks and wet well will likely require the purchase of additional property to the east of the existing treatment facility site. Some of this property may be within the flood plain. Prior to preliminary design, a wetland and topographic survey should be completed to better define the feasibility of this plan along with other construction issues.

The plan and section views shown on Exhibit 6-1 provide a conceptual layout of the proposed headworks building described above. Expansion of the wet well would increase its volume four-fold from 4,200 gallons to roughly 18,000 gallons.

Exhibit 6-1: Future Headworks Building



Proposed Headworks - Section View



Proposed Headworks - Plan View

The structural improvements shown on Exhibit 6-1 are presented herein as part of the Phase II Expansion. The existing screens will be maintained and one new Lakeside Model FS63 screen would be installed. The septage receiving unit would also be installed during Phase II. The third channel will be utilized for an optional by-pass until Phase III when the second 9.6 MGD fine screen would be required. The existing screens will be replaced in Phase IV with an identical unit to Phases II & III.

Table 6-11: Phased Treatment Facility Expansion Design – Fine Screens

Parameter	Phase I (Existing)	2017 P Removal	Phase II	Phase III	Phase IV
Number of channels	2	2	3	3	3
Design Average Flow, MGD	2.0	2.0	3.0	4.5	6.0
Peak Hourly Flow, MGD	5.0	5.0	7.5	10.35	13.2
Capacity existing screens, MGD	6.8	6.8	6.8	9.6	9.6
Capacity new screens, MGD	n/a	n/a	9.6	9.6	9.6
Capacity bypass channel, MGD	n/a	n/a	9.6	9.6	9.6
Screen Openings, inch	¼	¼	¼	¼	¼

Each phase complies with the minimum design requirements for providing continuous operation with the largest unit out of service.

6.6 RAW SEWAGE PUMP STATION

Expansion of the wet well will provide greater flexibility in operation for the pump station. The raw sewage pumps will continue to be controlled by variable frequency drives. As such the pumping system will be able to flow pace and maintain a near-constant level in the wet well, reducing pump cycling and life shortening electric motor starts. The dry well has space for four pumps. The pumping system for each phase should possess the maximum range to limit start and stop operation. In addition, the pump station is rated with the largest pump out of service. Therefore, the range of flows must be covered by a three-pump system. Additional wet well volume would significantly reduce motor starts.

The pumping system installed during the Phase I Expansion only provided firm capacity for a peak flow of 5.0 MGD, which will not meet the future Phase II requirements. The pumps would need to be replaced as part of the Phase II Expansion.

All four pumps would be replaced with dry-pit immersible pumps all similarly sized. This will allow for automatic alternation of all four pumps, thereby equalizing pump wear. This arrangement will also allow for load sharing between the pumps when operating at variable speed thereby ensuring optimal pumping efficiencies and avoiding potential cavitation. If the building were to be flood-proofed, immersible pump motors would not be needed.

Table 6-12: Phased Treatment Facility Expansion Design – Raw Sewage Pumps

Parameter	Phase I (Existing)	2017 P Removal	Phase II	Phase III	Phase IV
Design Average Flow, MGD	2.0	2.0	3.0	4.5	6.0
Peak Hourly Flow, MGD	5.0	5.0	7.5	10.35	13.2
Peak Hourly Flow, gpm	3,470	3,470	5,210	7,185	9,165
Number of pumping units	4	4	3	3	3
Number of standby pumping units	1	1	1	1	1
Pump Type (1)	SHP	SHP	SCP	SCP	SCP
Pumping Range, each, gpm	1,160	1,160	870-1740	1200-2400	1530-3055
Firm Pumping Capacity, gpm	3,500	3,500	5,220	7,200	9,165
Force Main Reach 1 diameter, in	12	12	18	18	24
Force Main Reach 1 min velocity, fps	3.29	3.29	1.10	1.51	1.08
Force Main Reach 1 max velocity, fps	9.93	9.93	6.58	9.08	6.50
Force Main Reach 2 diameter, inches	18	18	18	18	24
Force Main Reach 2 min velocity, fps	1.46	1.46	1.10	1.51	1.08
Force Main Reach 2 max velocity, fps	4.41	4.41	6.58	9.08	6.50

Each phase complies with the minimum design requirements for providing continuous operation with the largest unit out of service.

6.6.1 Raw Sewage Force Main Improvements

During the Phase I Expansion, the original 12-inch main which conveyed pumped raw sewage to the package treatment facilities was not replaced and still remains in service. The force main was extended to the new oxidation ditch as an 18-inch force main. Typically, force mains are sized to flow at velocities ranging from 2 feet per second to 9 feet per second. This range prevents the settling of solids, yet does not cause unnecessary headloss through the piping.

The current firm pumping rate of 3,500 gpm results in velocities within the 12-inch reach of force main of nearly 10 fps. The original design of the raw sewage pump station provided for dual 12-inch force main connections. It is recommended that both connections be used and that the original reach of 12-inch force main be upsized to 18-inch diameter. However, as discussed in Section 5, it is recommended that the existing 18-inch magnetic flow meter be replaced with a 12-inch mag meter to increase minimum velocities through the meter.

The future Phase II raw sewage pumping rates would range between a minimum of 870 gpm and a maximum of 5,210 gpm. Once the force main is increased to 18-inch diameter throughout, it will be more than adequate to convey the Phase II pumped flows.

The Phase III pumping rates would range between a minimum of 1,200 gpm to a maximum of 7,200 gpm. Velocities in the 18-inch force main would have reached 9 fps but only during maximum flow wet weather events and is considered adequate. Means to divert flow between the existing oxidation ditch and the Phase III adjacent ditch would be incorporated into the raw sewage force main piping.

For the ultimate Phase IV Expansion the 18-inch force main would need to be replaced with a 24-inch force main in order to maintain velocities below 9 fps. The Phase IV pumped flows would range between 1,530 gpm and a maximum of 9,165 gpm.

6.7 BIOLOGICAL PROCESS

As explained in Section 5, the District employs a suspended growth biological process defined as single stage nitrification. The single stage nitrification process is a version of activated sludge that creates an environment to promote BOD_5 reduction and nitrification of ammonia (conversion to nitrate and nitrite) within the same process. The environment promotes the growth of aerobic microorganisms, which metabolize the influent BOD_5 and nutrients to energy and biomass.

Two-Ring Oxidation Ditch

The current process is completed within a two-ring oxidation ditch. The oxidation ditch design allows for the completely mixed basins to operate in series or parallel. Mechanical aerators are attached to the walls of the channels, which include a series of discs that rotate in the wastewater. The discs provide both mixing energy and the oxygen required by the microorganisms for aerobic respiration. Variable speed drives were installed in 2014 for the existing aerators.

The multi-ring oxidation ditch design possesses significant flexibility and can be adapted to incorporate biological phosphorus removal and the denitrification (conversion of nitrite and nitrate to nitrogen gas) by implementing slight operational modifications.

Table 6-13: Phase Treatment Facility Expansion Design – Biological Process

Parameter	2017 P Removal	Phase II
Design	Oxidation Ditch	Oxidation Ditch
Design Average Flow	2.0 MGD	3.0 MGD
Peak Hourly Flow	5.0 MGD	7.5 MGD
BOD_5	168 mg/L	205 mg/L
BOD_5	2800 lbs/day	5,130 lbs/day
TSS	202 mg/L	240 mg/L
TSS	3370 lbs/day	6,005 lbs/day
NH_3-N	35 mg/L	35 mg/L
NH_3-N	417 lbs/day	876 lbs/day
Phosphorus	6 mg/L	6 mg/L
Phosphorus	100 lbs/day	150 lbs/day
Number of channels	2	3
Sidewater depth	14 feet	14 feet
Inner channel width	20 feet	20 feet
Middle channel width	20 feet	20 feet
Outer channel width	n/a	20 feet
Detention time	16.1 hrs	19 hrs
Total Volume	1,346,000 gallons	2,357,509 gallons
Solids Production	2,000 lbs/day	5,505 lbs/day
Solids Inventory @ MLSS = 4000 mg/L	45,000 lbs	78,730 lbs
Sludge Age	22 days	14.3 days
Oxygen Required	342 lbs/day	482 lbs/hr
Oxygen Supplied	450 lbs/day	900 lbs/hr

Denitrification is a process that occurs when the microorganisms that have converted the ammonia to nitrate (NO^{-3}) and nitrite (NO^{-2}) are placed in an environment containing low dissolved oxygen levels. The aerobic microorganisms then recover the oxygen from the nitrate and nitrite for respiration. As a result, the nitrogen gas is released to the atmosphere.

Biological phosphorus removal is slightly more complex. The microorganisms that are selective for phosphorus survive in both aerobic and anaerobic conditions. Under aerobic condition, the organisms convert phosphorus to phosphate PO_4^{-3}, but under anaerobic the phosphate is broken down to provide the required oxygen for respiration. Biological phosphorus removal requires development of a process where the microorganisms are repeatedly exposed to an anaerobic zone as part of the process. The three-channel design accommodates this requirement very well by operating the outer channel very close to an oxygen deficit.

The future Phase II Expansion to 3.0 MGD will include construction of the third channel as originally proposed in the 1998 Facility Plan Amendment. By providing additional process control, the single stage nitrification process can be upgraded to provide biological phosphorus removal and denitrification.

Should process upgrades be included for biological nutrient removal, increased operator attention will be required. Raw sewage and RAS flow rates, along with dissolved oxygen levels in each channel, will need to be monitored and controlled. The staff will also closely monitor the key operational parameters such as sludge age and feed to mass ratio. The variable speed aerator drives provide the capability to vary the speed of the rotating aerators as required to meet the oxygen demands of the biological process.

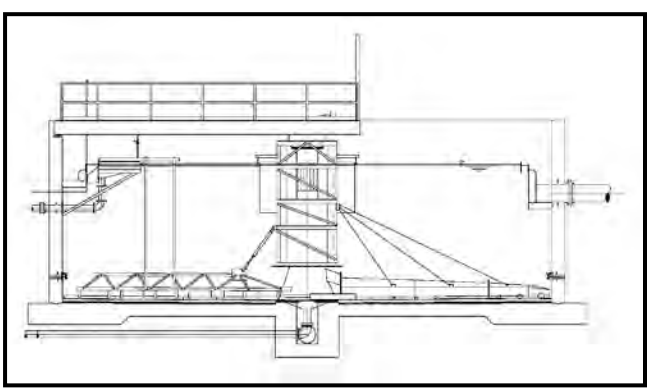
Phase III (4.5 MGD) would include construction of a second oxidation ditch providing sufficient volume to qualify as extended aeration, and as such the aerobic digestion process will not need to be increased during that phase. Phase IV (6.0 MGD) will increase organic and nutrient loading to duplicate the Phase III requirements. The oxidation ditch will not need to be expanded but the aerobic digesters would be upgraded to address the increased sludge production.

6.8 FINAL CLARIFIERS

The performance of the existing final clarifiers has been outstanding. The solids concentration in the final effluent is similar to that of a tertiary treatment facility because the clarifiers are oversized. The Illinois EPA standard for final clarifier surface overflow rates is 1,000 gpd/ft^2. The existing 85-foot diameter final clarifiers are currently designed for a surface overflow rate of 440 gpd/ft^2 which will only increase to 660 gpd/ft^2 during the future Phase II.

The existing mechanisms were manufactured by US Filter/ Envirex. The Tow-Bro® clarifier is shown on Exhibit 6-2 and works on a hydraulic differential principal for sludge removal. The current design is large enough to accommodate the future Phase II flow with only minor piping changes. A third final clarifier would be constructed in Phase III and a fourth in Phase IV.

Exhibit 6-2: To-Bro Suction Arm Clarifier Mechanism



The anticipated design loadings for each phase is summarized in Table 6-14.

Table 6-14: Phased Treatment Facility Expansion Design – Final Clarifiers

Parameter	Phase I (Existing)	2017 P Removal	Phase II	Phase III	Phase IV
Design Average Flow, MGD	2.0	2.0	3.0	4.5	6.0
Peak Hourly Flow, MGD	5.0	5.0	7.5	10.35	13.2
Peak Hourly Flow, gpm	3,470	3,470	5,210	7,185	9,165
Number of Clarifiers	2	2	2	3	4
Clarifier Diameter, feet	85	85	85	85	85
Total Surface Area, ft^2	11,344	11,344	11,344	17,016	22,688
Surface Loading @ PHF, gal/day/ft^2	440	440	661	608	582
Solids Loading @ PHF, lbs/day/ft^2	15	15	22	20	19
Weir Length, feet	486	486	486	729	972
Weir Loading Rate, gal/day/ft	10,288	10,288	15,432	14,197	13,580

6.9 DISINFECTION FACILITY

The chlorination facility was constructed in 1998 as part of the Phase I Expansion. The original design provided adequate capacity for a Peak Hourly Flow of 7.5 MGD, which under current flow projections represents the future Phase II peak flow.

Chlorine is a very strong oxidizing agent used for disinfection. The efficiency of the chlorine disinfection process is a function of chlorine concentration and detention time.

A Parshall flume upstream of the chlorine contact tank measures the final clarifier effluent flow. The chlorine dosage is flow paced to provide adequate disinfection without overdosing. The chlorine contact tank includes two parallel serpentine channels, which ensure that short-circuiting cannot occur which may result in inadequate contact time.

Depending on the detention time in the chlorine contact tank and the strength of the original chlorine concentration, chlorine residual is likely to be present prior to discharge. If the concentration of the chlorine in the effluent is significant, it may have a negative effect on native species within the receiving waters. The NPDES permit limits the concentration of chlorine in the effluent to 0.05 mg/L to avoid these issues. The District neutralizes the chlorine residual by feeding bisulfate prior to discharge.

The existing facility meets the Illinois EPA requirements for the 2017 P Removal and the future Phase II Expansion. The design calculations for the 2017 P Removal and the Phase II improvements related to the Chlorine Contact Tank are summarized in Table 6-15.

Table 6-15: Phased Treatment Facility Expansion Design – Disinfection Process

Parameter	2017 P Removal	Phase II
Design Average Flow, MGD	2.0	3.0
Peak Hourly Flow, MGD	5.0	7.5
Number of Tanks	2	2
Length, feet	56	56
Width, feet	12	12
Sidewater Depth, feet	8.5	8.5
Total Volume, cu ft	11,425	11,425
Total Volume, gallons	85,460	85,460
Detention Time at Peak Flow, minutes	24.6	16.4

Historically chlorination/dechlorination has been a preferred disinfection technology. However, concerns over disinfection by-products, post treatment requirements, public safety and chemical handling are some of the reasons most treatment facilities are converting from chlorination to ultraviolet disinfection.

Ultraviolet disinfection is an environmentally friendly method of disinfecting wastewater. Microorganisms, including viruses, are inactivated when exposed to UV-C light in a controlled environment and dosage. The UV-C light with a frequency of 254 nanometers causes a physical reaction with the organisms' DNA. This reaction prevents cell division and reproduction of potentially dangerous organisms and viruses.

While the chlorination system meets the design standards for the 2017 P Removal and the Phase II improvements, additional capacity in some form will be required for Phases III and IV. The technology for ultra-violet disinfection equipment is rapidly improving. Therefore, selection of a particular design in this report is not practical. It is proposed that the District evaluate the current technology and install an ultraviolet disinfection system during Phase III or possibly during Phase II if it is decided that the conversion to ultraviolet disinfection is desirable.

6.10 RAS/WAS PUMP STATION

The existing RAS/WAS pump station consists of two constant speed 1,400 gpm pumps in a small pre-cast wet well. The pumps are controlled by floats and operate in an on/off cycle. These pumps currently have sufficient capacity for the Phase I and 2017 P Removal designs, but will not be capable of meeting the requirements for future Phase II and beyond. The RAS/WAS pumps were replaced in 2014. It is recommended that consideration be given to providing variable speed drives for the RAS/WAS pumps to reduce the cycling of pumps at the currently lower RAS rates associated with the lower influent flows and loads. RAS pumping requirements for the 2017 P Removal and the Phase II (3.0 MGD) improvements are summarized in Table 6-16. A new RAS/WAS pump station would be needed for Phase II.

Table 6-16: Phased Treatment Facility Expansion Design – RAS Pumping

Parameter	2017 P Removal	Phase II
Design Average Flow, MGD	2.0	3.0
Influent BOD_5, mg/L	220	220
MLSS Concentration, mg/L	4,000	4,000
RAS Concentration, mg/L	7,500	7,500
RAS Rate, MGD	2.16	3.24
RAS Pumping Rate, gpm	1,500	2,250

The waste activated sludge from the biological process is expected to be equal to the incoming BOD_5 due to the potential inclusion of biological phosphorus removal. WAS computations for the 2017 P Removal and the Phase II (3 MGD) improvements are summarized in Table 6-17.

Table 6-17: Phased Treatment Facility Expansion Design – WAS Rates

Parameter	2017 P Removal	Phase II
Design Average Flow, MGD	2.0	3.0
Influent BOD_5, mg/L	220	220
WAS, lbs/day	3,670	5,505
WAS Concentration, mg/L	7,500	7,500
WAS Rate, gpd	58,673	88,010
Number of WAS Pumps	2	2
Pump Rated Capacity, gpm	250	250
WAS Pump Operation, hrs/wk	27.4	41.1

The waste sludge volume in Phase III should not significantly increase from Phase II because the treatment facility will operate in the extended aeration mode providing extended detention time in the biological process. The volume in Phase IV will increase to 11,010 lbs/day or 172,000 gpd.

6.11 BIOSOLIDS STABILIZATION PROCESS

In 2013 the aerobic digesters were upgraded. The interior steel walls were removed, the diffused aeration system was replaced, and the digesters were covered. The digesters in their current configuration are adequate for the 2.0 MGD treatment facility.

The aerobic digesters will need to be expanded in future phases. Alternatives for expansion include mechanical thickening, gravity thickening, or membrane thickening.

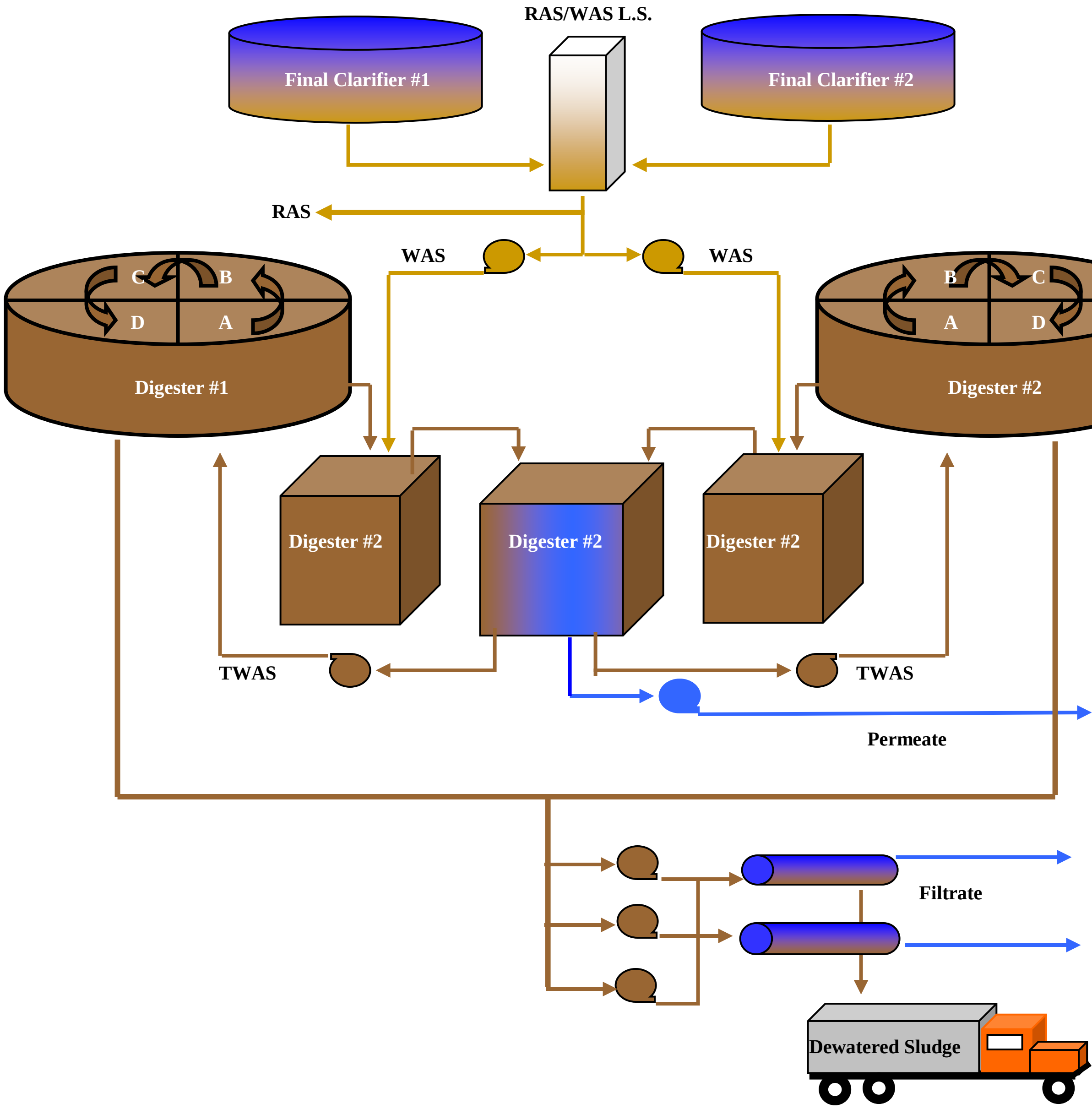
The District is currently capable of providing mechanical thickening using the centrifuge. If this tactic is to be implemented in future phases, it is recommended that the District install centrifuges specifically designed for thickening applications that do not require the use of polymer.

Gravity thickening results can vary and will typically thicken waste activated sludge to 2 to 2.5 percent solids. Therefore, implementation of this alternative will likely require additional aerobic digester volume to be constructed.

Membrane technology for the thickening application was investigated. The membrane thickening system is capable of thickening the digester solids to 3.0 to 3.5 percent solids.

The following conceptual design includes construction of a membrane thickener complex and rehabilitation of the existing digesters. At Design Average Flow it is estimated that the biological process will produce 5,505 lb/day at 0.75% solids or 88,000 gpd of Waste Activated Sludge. The WAS from the RAS/WAS pump station will be tributary to the two anoxic tanks of the proposed membrane thickening complex. The anoxic tanks have capacity for 25,000 gallons each.

The aerobic digestion process will consist of eleven basins including; two anoxic/ alkalinity recovery basin, a membrane thickening basin, and each existing digester (1 and 2) will be divided into four cells (A-D). Digesters 1 and 2 will operate in parallel while the cells within the digesters will operate in series. The process will incorporate continuous recycle from Digesters 1A and 2A to the anoxic and thickening basin. Cells B, C and D of each digester will isolate the digesting sludge to prevent recontamination.

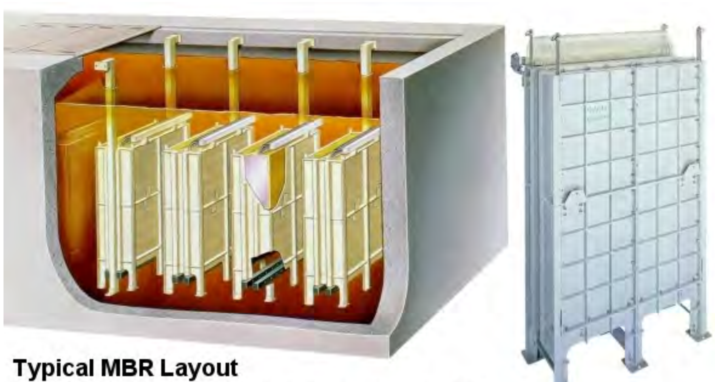


Increasing the solids concentration above 3.0% will allow heat from the exothermic reaction to be retained by the solids within the digesters. As the temperature within the digesters rises, the rate of volatile solids reduction changes. The rate of VSS reduction is a function of time and temperature. In anticipation of the variable conditions, the air design will incorporate capabilities to vary the air distribution to match the volatile solids destruction, taking place within each of the four cells. In addition, it is important to allow for alkalinity recovery. Therefore the design will incorporate an anoxic mixing zone for denitrification.

The proposed design includes placing the membrane system in a continuous loop with the first stage digester. This approach will provide in excess of 80 days detention time in the digesters by removing water as destruction occurs.

The temperature range within the aerobic digesters is expected to range from 20 ºC to 35 ºC. Based on the time–temperature curve provided in Manual of Practice 11, we can anticipate approximately 1,740 days-ºC at 20 ºC that equates to 51% reduction.

A proposed WAS pump controlled by a VFD will continuously feed the 0.75% WAS to the proposed anoxic mixing tanks. Simultaneously, approximately 400 gpm of 3.0% sludge from Digesters 1A and 2A will be transferred to anoxic mixing tanks. The fresh sludge will provide the carbon source for denitrification and pH control within the digestion system. The anoxic mixing tank is sized to provide approximately two hours detention time.



Typical MBR Layout

A MBR basin can be constructed of concrete or steel. To facilitate installation and maintenance, optional gu ide rails are available. MBR basins should be covered to avoid the introduction of fouling debris and mitigate the effects of ambient temperature on plant operation.

To maintain a continuous flow from the anoxic tank to the membrane thickener and back to the digester, an air-lift pump will be installed within the system to provide recirculation. It was determined that the recirculation rate should be five times influent flow rate and therefore the pump should be capable of moving 500 gpm.

During investigation of the membrane thickening technology, two membrane manufacturers were reviewed. It became evident that the membrane plate technology was much better suited for this application than hollow fiber strands. The membrane system will be designed for a flux rate of 5.1 gallons per day per square foot of membrane area based on results of a pilot installation and operational data from the manufacturer. Based on the influent flow and burn down anticipated within the digester, 13,760 square feet of membrane will be required which equates to 1,600 membrane plates.

Maintenance of the membrane system is limited to biological cleaning with 0.5% sodium hypochlorite solution once every four months. This cleaning process will only take roughly an hour and will require the addition of roughly 1,200 gallons of chemical.

As stated previously the system is expected to operate in a range between 20 ºC and 35 ºC. The lower value was based on operating data accumulated over the past three years at lower solids concentrations. It is expected that the volatile solids reduction in the first phase will be a minimum of 38% reduction. As a result, the aeration requirements for the first phase digester range are 2,400 – 3,000 cfm.

The digested sludge from the first phase digester overflows to the second (B) and third (C) stage before entering the digested sludge holding tank (D). The aeration requirements for the subsequent stages are defined by the required mixing energy rather than oxygen demand, due to the efficiency of the first stage reactor.

Expansion is not projected to be required within the planning period, and it is recommended that this technology be reviewed once again before expansion and implementation.

6.12 SLUDGE DEWATERING AND STORAGE

It is anticipated that the District will continue its current sludge dewatering and storage practices. The existing centrifuge and belt filter press will continue to be used for sludge dewatering and should be rehabilitated or replaced when they reach the end of their useful lives.

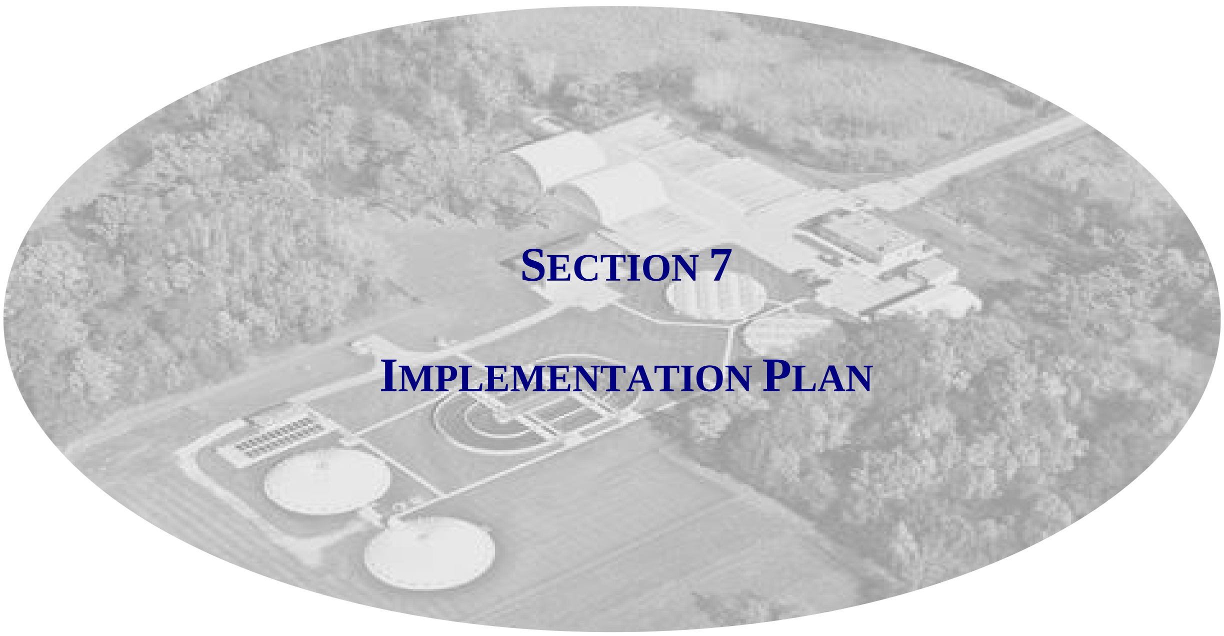
The Phase II improvements will include the covering of additional sludge drying beds to increase the dewatered sludge storage capacity and meet storage requirements. Construction of a new and separate sludge storage building could be considered during design. The District currently contracts for land application of biosolids on agricultural property. It is recommended that the District provide 150 days winter storage if another means of disposal is not available.

6.13 PERMITTING

The approval and permitting requirements for the 2017 P Removal and the Phase II Expansion projects, (when flows reach 1.6 MGD), will include:

- Submittal of the Phosphorus Removal Feasibility Report and other phosphorus-related documentation required by the NPDES permit
- Submittal of the 2014 Facility Plan Update to CMAP
- Submittal of the 2014 Facility Plan Update to the IEPA
- Application for an updated NPDES Permit (Phase II only)
- Request for an Illinois Historical Protection Agency Consultation
- Army Corps of Engineers sign-off for work in Flood Plain
- IEPA Construction Permit

Each of the permits will be acquired during the appropriate phase of the project.



SECTION 7

IMPLEMENTATION PLAN

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7. IMPLEMENTATION PLAN

The previous sections of this report assessed current and future conditions within the Northern Moraine FPA. The capabilities of the existing collection system to convey current and projected future flows was assessed and improvements recommended to provide that capacity required to support continued growth.

The NMWRD treatment facility was also assessed, including required improvements to rehabilitate existing equipment, the incorporation of phosphorus removal capabilities to meet new regulatory requirements, and the necessary expansions to support future growth.

The section integrates all of these recommended improvements into an overall Implementation Plan considering expected service life, regulatory requirements, and projected development.

7.1 COLLECTION SYSTEM IMPROVEMENTS

The need to set aside replacement funds to reinvest in the aging collection system infrastructure was discussed and was approximated at about $617,300 per year. Necessary replacement funds to reinvest in the aging lift station infrastructure was approximated at about $324,800 per year.

Future capital improvements include construction of either the Darrell Road Collection System or the recommended alternative Interim Solution. Probable capital costs for the Interim Solution are summarized in Table 7-1.

Table 7-1: Probable Capital Costs – Interim Solution Collection System

Phase	Description	Probable Cost
1	**Darrell Road Force Main** (16-inch Force Main on Darrell Road from NE Basin to Route 176)	$ 8,464,000
2	**Lift Station 7 Upgrades - Future** (Located at existing Lift Station 7 site)	$ 3,112,000
3A	**Treatment Plant Interceptor** (42-inch Sewer from WWTF to Water's Edge Lift Station)	$ 5,056,000
TOTAL PROBABLE CAPITAL COSTS		**$ 16,632,000**

As discussed in Section 3, the wet well at Lift Station 7 is designed to accommodate the installation of a third submersible pump. The initial improvements at Lift Station 7 would only involve the installation of the third pump, which would be sufficient to meet the projected 20-year peak flow from the Northeastern Basin.

7.2 TREATMENT FACILITY IMPROVEMENTS

Recommended improvements at the NMWRD treatment facility include the rehabilitation or replacement of existing equipment that is reaching the end of its expected service life, improvements to provide for phosphorus removal and additional facilities to expand the treatment capacity in response to growth throughout the Northern Moraine FPA.

Probable costs for the various improvements at the NMWRD treatment facility under each of the three categories of improvements are summarized in Table 7-2.

Table 7-2: Probable Capital Costs – Phased Treatment Facility Improvements

Description	Rehabilitation	Regulatory Compliance	Phase II Expansion
CONSTRUCTION SUBTOTAL	$ 600,000	$ 358,015	$ 13,252,300
GENERAL CONDITIONS	93,750	44,750	1,600,000
CONTINGENCY (25%)	173,440	60,415	3,313,100
PROBABLE CONSTRUCTION COST	$ 867,190	$ 463,180	$ 18,165,400
DESIGN ENGINEERING @ 7.5%	65,050	34,740	1,362,500
CONSTRUCTION ENGINEERING @ 7.5%	65,050	34,750	1,362,500
TOTAL CAPITAL COST	**$ 997,290**	**$ 532,660**	**$ 20,890,400**

7.3 IMPLEMENTATION PLAN

The Northern Moraine NPDES permit was issued on October 23, 2014 and has an effective date of November 1, 2014. The permit includes a compliance schedule which includes milestones related to phosphorus removal planning, design, construction and start-up.

The schedule requires that designs for phosphorus removal be submitted to the IEPA by May 1, 2017 and that construction be completed by May 1, 2018. Full compliance with the 1.0 mg/L phosphorus limit must be achieved by May1, 2019. It is anticipated that the recommended chemical phosphorus removal system will be effective immediately upon start-up; it will not take a year for the system to achieve desired results, although it will be optimized during this period and into the future.

Interim progress reports must also be submitted every 6 months, including an interim report of the Phosphorus Removal Feasibility Study which is due on April 1, 2015. The final Phosphorus Removal Feasibility Study must be submitted prior to November 1, 2015.

The various improvements recommended for rehabilitation of the existing NMWRD treatment facility could be implemented as needed and as funding becomes available.

The Phase II Expansion is not anticipated to be required within the 20-year planning period except under the most optimistic high rate growth assumptions discussed in Section 2.

The construction of the Interim Solution Collection System improvements would not be required until either the current capacity of Lift Station 7 is reached, or until the surplus capacity currently available in the 24-inch Route 176 West Interceptor is exhausted (4,425 PE currently available). The growth projections presented for the Central, Waterford, Northwest and Northeastern Basins indicate the interceptor's capacity being reached in the year 2025 under the most optimistic high rate growth projections. Using the CMAP lower growth rate projections, only an additional 3,622 PE (82 percent of available capacity) will be connected in the four basins.

In consideration of the remaining service life of the existing facilities, regulatory requirements, and projected growth through the Northern Moraine FPA, the phased Implementation Plan for this 2014 Facility Plan Update are summarized in Table 7-3.

Table 7-3: Phased Implementation Plan

Description	Probable Capital Costs ($ millions)						
	2015	2016	2017	2018	2019	2020 to 2029	2030 to 2039
TREATMENT FACILITY							
Rehabilitation / Replacement		$ 0.08	$ 1.13				
Phosphorus Removal		$ 0.04	$ 0.50				
Phase II Expansion						$ 20.90	
COLLECTION SYSTEM							
Interim Solution Eastside System						$ 13.52	$ 3.11
Annual rehabilitation / replacement	$ 0.62	$ 0.62	$ 0.62	$ 0.62	$ 0.62	$ 6.20	$ 6.20
LIFT STATIONS							
Annual rehabilitation / replacement	$ 0.33	$ 0.33	$ 0.33	$ 0.33	$ 0.33	$ 3.30	$ 3.30
TOTAL PROBABLE CAPITAL COSTS	$ 0.95	$ 1.07	$ 2.58	$ 0.95	$ 0.95	$ 43.92	$ 12.61

NOTE: PROBABLE COSTS ARE PRESENTED ABOVE IN 2014 DOLLARS AND DO NOT ACCOUNT FOR FUTURE INFLATION. OVER THE PAST 10 YEARS, ENR COST INDEXES HAVE INFLATED AT AN EQUIVALENT ANNUAL COMPOUND RATE OF 3.16%.

Replacement expenditures for sanitary sewers was estimated at approximately $617,300 annually. Attention should first be given older areas in the collection system and to those areas where I/I has been detected. Replacement costs for lift stations and force mains was estimated to be approximately $324,800 per year.

The scheduling of these improvements are initial estimates employed for planning purposes. The phosphorus removal improvements at the NMWRD treatment facility must be completed by May 1, 2018 as required in the District's NPDES permit. Scheduling for the Phase II Expansion and the Interim Solution Collection System will become known with greater certainty over time as actual development throughout the Northern Moraine FPA occurs.

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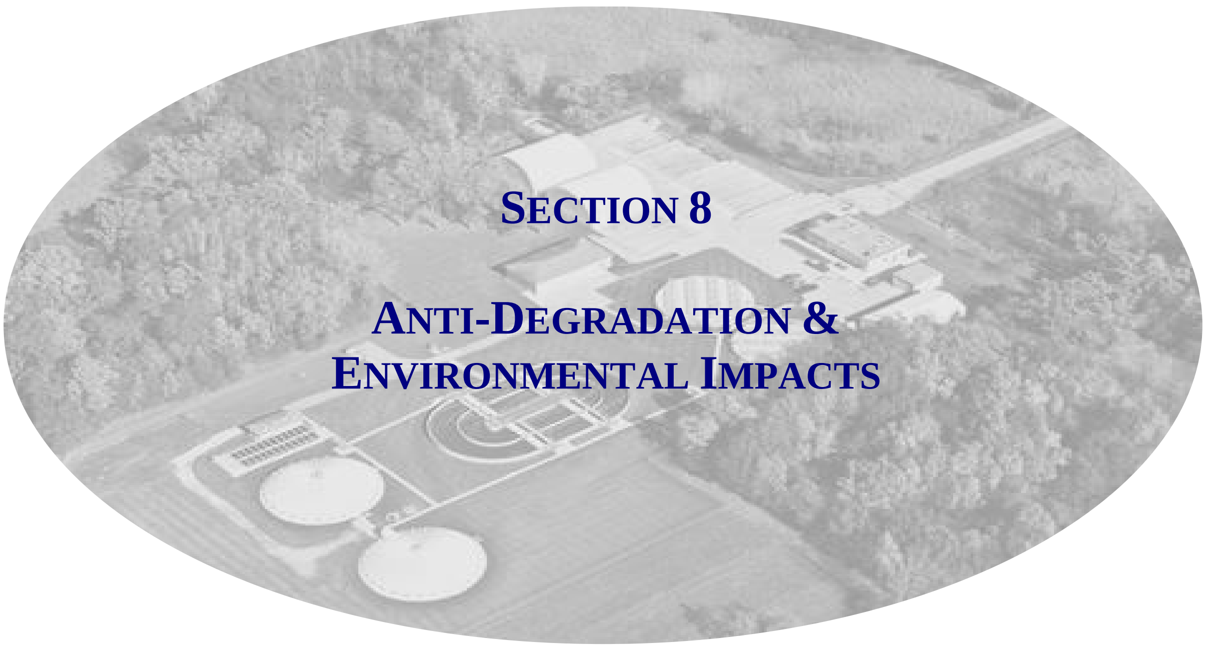

SECTION 8

ANTI-DEGRADATION & ENVIRONMENTAL IMPACTS

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8. ANTI-DEGRADATION AND ENVIRONMENTAL IMPACT ANALYSIS

8.1 GENERAL DISCUSSION

The Northern Moraine Wastewater Reclamation District is responsible for providing sanitary service and treatment for the communities within the Facility Planning Area. Sections 1 through 6 describe the Facility Planning Area, the anticipated development, collection system, and treatment facility expansion needs in detail. As the designated management agency, the District is also responsible for meeting the long-range goals of the Clean Water Act and to minimize the environmental impacts of pollution from the sanitary waste generated within the Facility Planning Area.

The District has and continues to work with each of the affected communities by providing sanitary service, encouraging responsible development practices and working with state and local agencies to protect the Fox River from pollutants.

In addition to actively pursuing solutions to the community's wastewater collection needs, the District has invested in upgrading the treatment facility with newer technologies to meet the needs of the Fox River Watershed. Some of the improvements to protect the environment incorporated since the last major expansion include:

- Rehabilitation of the aerobic digester for improved performance
- Installation of a high-efficiency blower for improved aeration
- Rehabilitation of the headworks including replacement of an influent screen

Based on the comprehensive plans provided by the surrounding communities, the wastewater treatment facility must be ultimately expanded to 6.0 MGD. As described in Section 6, expansion of the treatment facility will occur over four additional phases. While this report is required to address the next phase, a more appropriate and proactive approach is to identify the long range impacts of growth, discuss potential alternative solutions, and develop a plan of action that is appropriate and provides the greatest benefit to the environment and public served.

As shown in Section 5, the wastewater treatment facility's performance has been outstanding. The BOD_5, suspended solids, and ammonia loadings are continuously well below the NPDES permit limits.

The District is committed to upgrading the wastewater treatment facility in a manner that will be a benefit to both the communities served and the ecosystem surrounding the Fox River. The purpose of this environmental analysis is to identify the parameters of concern with an increase in discharge, as well as to minimize the impact of expansion and improve the existing conditions.

8.2 ENVIRONMENTAL AREAS OF CONCERN

Areas of environmental concern include not only the Fox River, but the wetlands and nature preserves within the area. The wildlife habitat and open space represent a significant portion of Facility Planning Area. Each community is responsible for creating and implementing criteria to protect the environment from impacts of development. The comprehensive plans prepared by the communities within the FPA recognize the importance of preserving open space and incorporating responsible development.

The most significant concern for future expansion of the treatment facility includes the quality of the final effluent. The plant's current effluent quality is exceptional. However, growth within the Facility Planning Area will lead to higher pollutant loading from other sources. Concerns over impacts on the surrounding environment including wetlands, wildlife habitat, and endangered species must be considered in anticipation of potential development.

8.2.1 Water Quality Concerns

The Clean Water Act was established to protect and revive the lakes, rivers, and streams throughout the United States. Restoring their quality is crucial in maintaining a healthy environment and ensuring the sustainability of these waters for all to use and enjoy.

Title 35 of the Illinois Administrative Code, Section 302 establishes the method for determining, implementing, and regulating Water Quality Standards. Section 302.105 – Antidegradation has been added to protect existing uses of all water, maintain the quality of waters, and prevent unnecessary deterioration of the waterways.

The Clean Water Act also established the NPDES Permitting program managed by the individual state agencies. The program establishes effluent limits that the Publically Owned Treatment Works (POTWs) must meet. The NMWRD has consistently been in accordance with its NPDES permit limits.

There are two methods of determining effluent limits. The first is Water Quality Based Effluent Limits (WQBEL's). WQBEL's have historically been used throughout Illinois to establish the NPDES Permit Limits for POTW Discharges.

The second method is to study a particular body of water and establish TMDL's (Total Maximum Daily Load) based on the ecosystem's ability to receive pollutants without having an adverse effect on the streams ability to support its designated uses. By taking a watershed approach, a TMDL considers all potential sources of pollutants, both point and non-point sources. It also takes into account a margin of safety, which reflects scientific uncertainty and future growth. The effects of seasonal variation are also included.

In short, a TMDL is calculated using the following equation:

$$TMDL = WLA + LA + MOS + SV$$

Where:

WLA = Waste Load Allocation (point sources)
LA = Load Allocation (non-point sources)
MOS = Margin of Safety
SV = Seasonal Variation

Section 303(d) of the Clean Water Act requires each state to prepare a list of waters of the state that are considered to be impaired for their intended uses. In 2014, the Illinois EPA issued a revised Integrated Water Quality report and Section 303(d) List. Portions of the Fox River have been placed on the list.

The District's wastewater treatment facility discharges to segment DT-22, which includes 7.86 miles of the Fox River. This segment has been identified as impaired but at a low priority. The assessment was based on site-specific data and concluded that segment DT-22 was not supporting aquatic life, fish consumption, or primary contact. A summary of these impairments and their causes is shown in Table 8-1.

Table 8-1: Summary of Fox River Segment DT-22 Impairments

Order	Priority	Hydrologic Unit Code	Water Name	Assessment ID	Water Size (miles)	Designated Use	Cause
1415	Low	0712000611	Fox River	IL_DT-22	7.86	Aquatic Life	Chloride, Copper, Sedimentation/Siltation
1416	Low	0712000611	Fox River	IL_DT-22	7.86	Fish Consumption	PCBs
1417	Low	0712000611	Fox River	IL_DT-22	7.86	Primary Contact Recreation	Fecal Coliform

The Illinois EPA defines the potential causes and sources of impairment for given water bodies. The causes of these impairments for segment DT-22 are summarized in Table 8-2.

Table 8-2: Causes of Impairments

Code	Cause
84	Alteration in stream-side or littoral vegetative covers
138	Chloride
163	Copper
319	Other flow regime alterations
371	Sedimentation/siltation
479	Aquatic Algae
348	Polychlorinated biphenyls
400	Fecal coliform

The sources of the impairments are also listed as codes, summarized below. The sources of impairment are summarized in Table 8-3.

Table 8- 3: Sources of Impairment to Fox River Segment DT-22

Code	Source	Description
58	Impacts from hydrostructure flow regulation/modification	Alteration of normal flow regimes (e.g. dams, channelization, impervious surfaces, water withdrawal) based upon actual observations and/or other existing data
157	Habitat modification- other than hydromodification	General alteration of riparian habitat based upon actual observation and/or other existing data
49	Highway/road/bridge runoff (non-construction related)	Salt and pesticide runoff from highways, roads, and bridges based upon actual observation and/or other existing data
177	Urban runoff/storm sewers	Urban and storm sewer runoff based upon facility-related stream survey, agency effluent and/or other existing data
142	Dam or impoundment	Dam or upstream impoundment based upon actual observation and/or other existing data
140	Source unknown	No identifiable source based upon available information

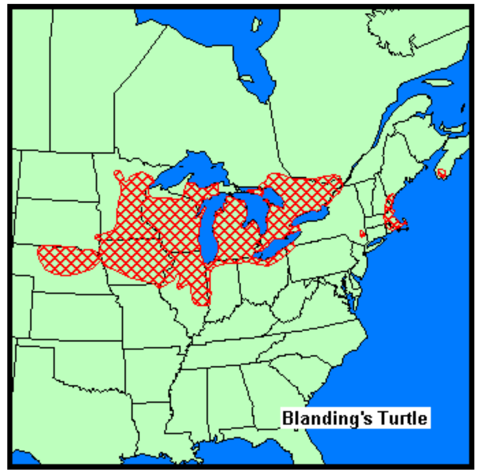
Interestingly, neither "municipal point source discharges" nor "on-site treatment systems" were listed as sources of impairment. As such, it can be concluded that the NMWRD does not cause any noteworthy impairment to segment DT-22 of the Fox River. However, it is still important to address threatened or protected areas/species that exist in the surrounding area.

8.2.2 Threatened and Endangered Species

The Illinois Department of Natural Resources offers an Ecological Compliance Assessment Tool (EcoCAT) that analyzes a given area and provides a list of protected resources in the vicinity of the project location. An EcoCAT was conducted for the areas surrounding the treatment facility and determined that the Blanding's turtle is found to be at risk in the areas affected by the treatment facility discharge. In addition, the Cotton Creek Marsh and Bates Fen Nature Preserves are both protected areas of open space that lie within the vicinity of the treatment facility.

Blanding's Turtle

The Blanding's turtle is named for William Blanding, the Philadelphia naturalist who originally described it. They are indigenous to the upper Midwest and Great Lakes region. The turtles primarily inhabit marshy shorelines, inland streams, and meadows. Although essentially aquatic, the Blanding's turtle also dwells on land. They are medium sized with a shell length of approximately seven to nine inches ranging up to 10 inches long. Males are larger than females and have longer tails



The turtle's bright yellow chin and throat distinguish it from others. The carapace, or upper shell, is domed, but slightly flattened along the midline and speckled with yellow flecks. The plastron, or lower shell, is yellow with symmetrical dark blotches. The dark head and legs are also speckled yellow.

The Blanding's turtle can remain at the bottom of waterbodies for hours when alarmed. It is gentle and not typically aggressive towards humans.

The turtle is omnivorous. While in the water, it feeds on crustaceans, snails, insects, frogs, and fish. It is capable of catching live fish, but crayfish appear to be a preferred food. On land it consumes earthworms, slugs, grasses, berries, and vegetation. The Blanding's turtle is unique because it can swallow food both in and out of the water.

The Blanding's turtle hibernates under or near water, in mud or under vegetation. Mating occurs in April and early May with nesting lasting throughout the month of June. After fertilization, females will bask in the sun as a warming behavior, called thermoregulation, which the development of their eggs enabling them to be laid sooner. This gives the eggs a better chance of hatching before the autumn frost. This, in turn, allows the hatchlings to grow before hibernating, giving the immature turtles a greater chance of surviving the winter.

Blanding's Turtle
Picture provided by Ohio DNR – Department of Wildlife

About half of the female population breeds annually. Like all turtles, they must lay their eggs on land and prefer a patch of sandy ground for nesting. They will travel up to one and a half miles from water to nest, and they usually return to the same nesting site each year. They typically lay their eggs during the late afternoon or after dusk. Once they deposit the eggs in the ground, the mothers return to the water, and the sun's warmth is used to incubate the nested eggs. The clutch may contain from 3 to 17 elliptical eggs. Between 65 and 90 days pass before they hatch.

The young are patterned differently from adults; their shells help them blend in with their surrounding environment. The hinge of the Blanding's turtle's plastron is not functional until the turtle is 3 to 5 years old. Before this age its yellow throat markings are not apparent. Blanding's turtles take 18-22 years to reach maturity and may live to be 70 years old.

A major treat to the Blanding's turtle is the destruction of its habitat during construction projects. Roads that cross routes between the ponds where the turtles hibernate and the areas where they nest are a significant threat.

Nature Preserves

The Cotton Creek Marsh is located along highway 176 in McHenry County and consists of 249 acres of wetlands. The marsh is home to many native plant species and is owned by the McHenry County Conservation District. This preserve is accessible by permission only. It lies directly west of the treatment plant on the east side of the Fox River.

The Bates Fen is a preserve also owned by the McHenry County Conservation District that consists of fen, meadow, and marsh located east of the Village of Oakwood Hills. The fen contains approximately 178 acres of land and is southwest of the treatment plant on the west side of the Fox River.

8.2.3 Input from Stakeholders

The USEPA, along with the IEPA, is currently implementing initiatives to limit nutrient concentrations in an effort to reduce or eliminate local water quality impairments as well as hypoxia in the Gulf of Mexico. As discussed in Section 6, the Illinois EPA is focused on statewide nutrient removal criteria for wastewater treatment facilities. The Illinois EPA, along with the Fox River Study Group and other stakeholders, are developing solutions to address the impairments found along the Fox River.

The IEPA is revising the water quality standards in Illinois which requires lower effluent limits for ammonia-nitrogen and phosphorus at Illinois POTWs. The Illinois nutrient strategy was released for public comment in November of 2014. The revised strategy is anticipated to be released in the first half of 2015. Coinciding with this, the Fox River Study Group is creating the Fox River Implementation Plan to improve dissolved oxygen and reduce algal growth by means of nutrient discharge limits at treatment facilities. This is expected to be completed by September 2015.

At this time many NPDES permits have expired; as such the IEPA has opted to release updated NPDES permits with new effluent standards in anticipation of the release of the Illinois nutrient strategy.

The IEPA issued a final permit for the NMWRD facility in October of 2014 with updated effluent standards. As anticipated, the District is now required to observe a 1.0 mg/L annual average phosphorus limit as well as monitor the total nitrogen and TKN in the effluent. Expansion of the plant to 3.0 MGD will change the phosphorus limit of 1.0 mg/L from an annual average to a monthly average maximum.

8.3 IMPACTS OF EXPANSION

The most significant impact of expansion on the environment will be from an increased discharge to the Fox River. As previously mentioned, the impairment to segment DT-22 of the Fox River is not attributed to any point discharge from municipal works, including the NMWRD. Further expansion of the plant will incorporate nutrient removal in accordance with IEPA standards and is not expected to contribute any negative impacts on the river.

8.4 REDUCING IMPACTS OF EXPANSION

8.4.1 Removal of Private Septic Systems

Much of the shoreline and low-lying areas adjacent to the river are populated by residences which possess private wells and septic systems. The soil and topographic conditions within these areas have raised concerns over the effectiveness of the septic systems and seepage fields. The District has committed to identify methods for extending service to these residents. The benefit of this effort will be the removal of the seepage fields that will subsequently relieve pollutant loadings on the river.

The best example of this effort is the District's intergovernmental agreement with the Village of Port Barrington. The Village includes roughly 270 homes that were previously served by private well and septic systems. Many of these homes are adjacent to the Fox River in low-lying areas. As a result several properties had experienced issues with failing septic systems. The District and the Village combined forces in the construction of a low-pressure sewer system to resolve this issue.

8.4.2 Reducing Construction Impacts on Wetlands

While the District has no authority to impact or dictate development practices, the District's responsibility is to improve the environment within its jurisdiction through providing superior collection and treatment solutions. The District is committed to providing any system expansion in a way that minimizes the impact on the existing wetlands and open space.

Section 3 details the District's plan to route the major sewers in the Darrell Road Collection System along the existing right-of-way where practical instead of installing it through the wetlands and low-lying areas. While this plan is slightly more expensive for construction, the alignment will minimize the construction and long-term maintenance impacts within sensitive areas.

8.4.3 Water Reuse

One of the methods for reducing the impact from the treatment facility expansion would be to incorporate a water reuse program into the project. Reviewing the Land Use Plan and the Facility Planning Area boundary, conservation areas and golf courses are the most eligible recipients for reuse water.

It is recognized that the District does not currently control property to incorporate spray irrigation into the proposed expansion from 2.0 to 3.0 MGD. It was concluded that the project could incorporate some beneficial reuse on-site in the NPW system and irrigate the plant site.

It is also suggested that as development within the facility planning area occurs that the District should investigate opportunities to irrigate future golf courses or provide reclaimed water for businesses where appropriate.

8.4.4 Biological Nutrient Removal

One approach to mitigating the impacts of the increased discharge quantity is to reduce the concentration of nutrients discharged from the treatment facility. As described in Section 5, the current biological process is single stage nitrification utilizing an oxidation ditch. The performance of the process has been exceptional and produced effluent results well below the current NPDES Permit Limits, including ammonia. However, this process does not address concerns over total nitrogen and phosphorus, which can be contributing factors to algae blooms.

It is well documented that the dams along the Fox River create the still water environment that promotes algae blooms. Removal of the dams has been discussed in several forums, and some are under consideration for removal. Yet, it is very unlikely that all the dams, particularly the Algonquin Dam, would be removed.

Without removal of the dams, the next best solution is to minimize the nutrients that algae need for reproduction. Phosphorus removal in wastewater treatment can be accomplished either through chemical precipitation or by biological means. Most biological phosphorus removal systems include an anaerobic selector zone to promote the survival of microorganisms that adsorb phosphorus. An analysis of these alternatives is described in Section 6.

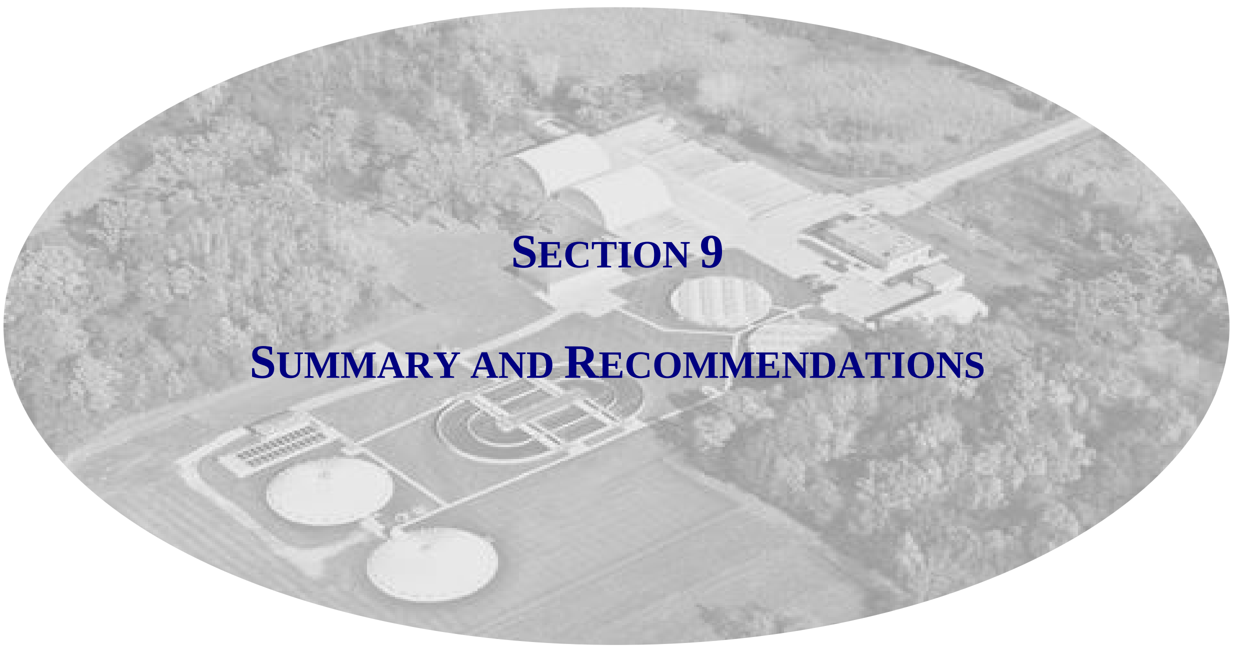
Nitrogen removal is commonly accomplished through the denitrification process. The denitrification process occurs under anoxic conditions, thereby starving the microorganisms for oxygen. The microorganisms are forced to breakdown the nitrate and nitrite molecules produced during nitrification to oxygen and nitrogen gas.

8.4.5 Adherence to NPDES Permitting

The updated NPDES Permit includes limits on BOD_5, TSS, ammonia, and now phosphorus. Based on the historical performance of the facility, it is projected that the existing treatment facility will not exceed the permit limits for BOD_5, TSS, or ammonia. In order to achieve effective phosphorus removal, the District will have to incorporate either biological or chemical treatment into its process as mentioned above.

These improvements in conjunction with the investment into removal of the private septic systems will result in a net benefit to the Fox River. Also described in Section 6, the District will comply with the special conditions described in the new NPDES Permit that dictate a timeline for phosphorus removal including a feasibility study to be submitted 12 months from the issuance of the permit.

It is recommended that the future NPDES permit increase the design average flow to 3.0 MGD and Design Maximum Flow to 7.5 MGD when development necessitates it. The permit could maintain the same weekly and monthly effluent concentration limits, but incorporate annual limits for BOD_5, TSS, and ammonia to represent the allotted discharge pounds of pollutants in the existing NPDES Permit.



SECTION 9

SUMMARY AND RECOMMENDATIONS

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9. SUMMARY AND RECOMMENDATIONS

9.1 GENERAL DISCUSSION

This section includes a summary of the recommendations outlined in this report, probable costs for these improvements, a potential implementation schedule, and the financial impacts and potential funding strategies for constructing the improvements.

Section 2 included an evaluation of current and projected population equivalents, wastewater flows and pollutant loadings. Current land use and local comprehensive planning documents were discussed as related to the potential for future growth.

The existing wastewater collection system was presented in Section 3 including a hydraulic assessment of the existing system's capacity to convey current and projected future flows. Current and future deficiencies were identified and alternatives reviewed to provide the sewage conveyance capacity necessary to support continued growth.

Section 4 summarized the District's lift stations, including preliminary assessments of each station's pumping capacity in relation to current flows. The condition of each lift station was discussed and the results of pump drawdown tests at each site were summarized.

Similarly, the existing wastewater treatment facility was discussed in Section 5, including the evaluation of each unit process to treat the current and projected wastewater flows and pollutant loadings. Recommendations for near-term rehabilitation, repair or replacement of worn or aging equipment were presented.

The existing NMWRD treatment facility was not designed to meet new phosphorus removal regulations now included in the Northern Moraine NPDES permit. The facilities will need to be improved to include phosphorus removal systems, and likely will be required to remove total nitrogen in the future. The special conditions of the NPDES permit require that construction of the phosphorus removal systems must be completed by May 1, 2018, and that full compliance be achieved by May 1, 2019. Projected growth throughout the planning area will also necessitate the treatment facilities to be expanded in response to increasing wastewater flows and pollutant loadings. Section 6 evaluates alternatives to attain near-term regulatory compliance along with the long-term plan for expanding the NMWRD treatment facility.

Section 7 comprised an Implementation Plan for implementing the recommended collection system and treatment improvements at the NMWRD facilities that were presented in Sections 5 and 6, including the rehabilitation of existing equipment and systems, regulatory compliance, and treatment facility expansions. A schedule for the phased implementation of these improvements was presented. The timing of collection system improvements and treatment facility expansions are inseparably related, and as such collection system improvements are also included in the Implementation Plan.

Facility planning reports must include an Anti-Degradation and Environmental Impact assessment of the proposed improvements. The environmental impacts of the recommended plan are summarized in Section 8.

9.2 POPULATION EQUIVALENTS AND WASTEWATER FLOWS

Future growth within the NMWRD facility planning area was assessed based on the Comprehensive Plans prepared by the Villages of Island Lake, Lakemoor, and Port Barrington. Outside of those planning areas, Lake County and McHenry County development codes were used to project development densities throughout the remainder of the Northern Moraine FPA. Overall flow and loading projections are summarized in Table 9-1.

Table 9-1: Current and Projected Influent Wastewater Flows and Loadings

Condition	PE	Average Flow (MGD)	Per Capita Flow (gcd)	BOD_5 (lbs/day)	Per Capita BOD_5 (lbs/PE/d)	TSS (lbs/day)	Per Capita TSS (lbs/PE/d)	NH_3-N (lbs/day)
Design	20,000	2.0	100	2,800	0.14	3,370	0.17	417
Current Conditions (2013– 2014)								
Current	13,695	1.05	77	1,793	0.13	1,595	0.12	227
Percent of Design	68%	53%	77%	64%	93%	47%	71%	54%
Projected Conditions at Build-out of NMWRD Incorporated Area								
Projected	16,338	1.3	80	2,242	0.14	2,124	0.13	282
Percent of Design	82%	66%	80%	80%	100%	63%	76%	68%
Projected 20-year Conditions within NMWRD FPA – HIGH RATE GROWTH								
Projected	27,921	2.47	88	4,211	0.15	4,440	0.16	524
Percent of Design	140%	124%	88%	150%	107%	132%	94%	126%
Projected 2-year Conditions within NMWRD FPA – LOW RATE GROWTH								
Projected	19,641	1.64	84	2804	0.14	2784	0.14	351
Percent of Design	98%	82%	84%	100%	100%	83%	82%	84%
Projected Conditions at Build-out of NMWRD FPA								
Projected	56,460	5.3	94	9,063	0.16	10,148	0.18	1,125
Percent of Design	282%	266%	94%	324%	114%	301%	106%	270%

In total, the population equivalent served by the NMWRD collection and treatment facilities is projected to increase by 412 percent at ultimate build-out of the FPA. For the 20-year facility planning period, the population equivalent served is projected to increase by about 43 percent for the lower CMAP growth projections, and to roughly double under the most optimistic high rate growth projections.

Table 9-2 illustrates how the current population equivalent served and the projected future growth is distributed across the eight wastewater drainage basins within the NMWRD FPA. It can be seen that projected growth is predominantly located within the Northwestern, Northeastern, Eastern, and Southern Basins.

Table 9-2: Current and Projected PE Breakdown by Basin

Drainage Basin	Existing PE	Percent of Total PE	Additional Future PE	Percent of Add'l Future PE	Future PE	Percent of Future PE
Central Drainage Basin	3,155	23%	2,270	5%	5,425	10%
Eastern Drainage Basin	69	0.5%	5,254	12%	5,323	9%
Near East Drainage Basin	2,235	16%	1,252	3%	3,487	6%
Northeastern Drainage Basin	1,912	14%	12,675	30%	14,587	26%
Northwestern Drainage Basin	1,618	12%	5,625	13%	7,243	13%
South Central Drainage Basin	378	2.5%	1,707	4%	2,085	4%
Southern Drainage Basin	1,197	9%	10,292	24%	11,489	20%
Waterford Drainage Basin	3,131	23%	3,690	9%	6,821	12%
Total PE	**13,695**	**100%**	**42,765**	**100%**	**56,460**	**100%**

9.3 WASTEWATER CONVEYANCE PROJECTS SUMMARY

An assessment of the existing NMWRD collection system was presented in Section 3. The analyses revealed that the existing systems are sufficient to convey current wastewater flows. However, as additional connections are made to the system, certain key components of the collection system will become overloaded.

The existing interceptor sewers immediately upstream of the NMWRD treatment facility and further upstream along Route 176 have sufficient capacity to convey current flows and also have surplus capacity to accept additional flow from a limited amount of growth. The existing 24-inch Route 176 West Interceptor has surplus capacity to serve an additional 4,425 PE before reaching full capacity. Additional capacity could be provided by paralleling the existing sewers or by constructing new collection systems to convey wastewater along an alternative route to the treatment facility site.

The Darrell Road Interceptor was previously identified in the 2004 Facility Plan Update as a solution that would relieve the overloading of the existing sewers, lift stations and force main while also providing a means of serving new development throughout the Northeastern and Eastern Basins. That plan has been modified for this 2014 Facility Plan Update to maintain proposed sewers and force mains within the roadway right-of-way, and not on private lands, in light of currently uncertainty as to if or when the Eastern Basin might be developed.

Probable capital costs for the Darrell Road Collection System as modified for this 2014 Facility Plan Update are summarized in Table 9-3.

Table 9-3: Probable Capital Costs - Darrell Road Collection System

Phase	Description	Probable Cost
1	Darrell Road Interceptor (South)	$ 2,255,000
2A	Mutton Creek Force Main	$ 2,149,000
2B	Mutton Creek Lift Station	$ 3,112,000
3A	Treatment Plant Interceptor	$ 5,056,000
3B	Water's Edge Interceptor Replacement	$ 1,902,000
4	Darrell Road Interceptor (Central)	$ 3,279,000
5	Darrell Road Interceptor (North)	$ 2,065,000
6A	Darrell Road Interceptor (Far North)	$ 1,687,000
6B	Lakemoor Lift Station 7 Force Main	$ 1,615,000
7	Fisher Road Interceptor	$ 3,321,000
TOTAL PROBABLE CAPITAL COSTS		**$ 26,441,000**

Interim Solution Collection System

Funding of the Darrell Road Collection System has always been contingent upon development within the Eastern Basin. An interim solution is to upgrade Lift Station 7 and construct a new force main from the Northeastern Basin to the intersection of Darrell Road and Route 176, as well as construction of Phase 3A. Therefore, the District would be able to construct all other phases if and when the Eastern Basin is developed.

Probable capital costs for the Interim Solution Collection System are summarized in Table 9-4. The major upgrades to Lift Station 7 would be deferred to the future as it was shown in Section 3 that the 20-year flow projections in the Northeastern Basin can be met simply by installing a third submersible pump in the existing wet well.

Table 9-4: Probable Capital Costs – Interim Solution Collection System

Phase	Description	Probable Cost
1	Darrell Road Force Main	$ 8,464,000
2	Lift Station 7 Upgrades - Future	$ 3,112,000
3A	Treatment Plant Interceptor	$ 5,056,000
TOTAL PROBABLE CAPITAL COSTS		**$ 16,632,000**

The interim solution provides capacity to serve only the Northeastern Basin, and can be constructed at less cost than implementation of the full Darrell Road Collection System. The interim solution includes major improvements to Lakemoor Lift Station 7 to increase its capacity to serve future growth in the Northeastern Basin and to pump that flow all the way to the intersection of Route 176 and Darrell Road. It is recommended that the District implement the Interim Solution to allow for unimpeded development of the Northeastern Basin. This project would need to be completed prior to exhaustive of the available capacity in the Route 176 West Interceptor (an additional 4,425 PE).

9.4 LIFT STATION ASSESSMENTS

The existing wastewater pumping facilities throughout the NMWRD collection system were summarized in Section 4. The existing features and condition of each lift station were discussed.

Pump run times at each of the twenty-two NMWRD lift stations were reviewed to assess the ability of the existing pumping equipment to meet current flow conditions. This data is summarized in Table 9-5, and indicates that all of the existing pump installations are sufficient for the current flow conditions.

Table 9-5: Lift Station Pump Average Runtimes

Lift Station	Pump Rated Capacity (gpm)	Pump 1 Average Runtime (hrs/day)	Pump 2 Average Runtime (hrs/day)	Total Average Runtime (hrs/day)
Hale 1	363	1.88	2.43	4.31
Hale 2	238	1.57	1.64	3.21
Waterford	850	1.18	2.40	3.58
Clearwater	60	0.26	0.27	0.53
Walnut Glen	284	0.30	0.27	0.57
Prairie Woods	265	0.35	0.32	0.67
Rolling Oaks	200	1.64	1.06	2.70
Deer Grove North	532	0.15	0.17	0.32
Rawson Bridge	740	1.08	1.19	2.27
Water’s Edge	100	0.88	0.80	1.68
Westridge	236	0.87	0.70	1.57
Burr Oak	90	0.52	0.55	1.07
South Shore	200	1.69	1.88	3.57
Fern	576	2.85	1.74	4.59
Treatment Plant	235	2.25	1.74	3.99
Lakemoor Station 1	450	3.00	2.72	5.72
Lakemoor Station 2	270	0.22	0.21	0.43
Lakemoor Station 3	270	2.41	2.55	4.96
Lakemoor Station 4	270	2.70	2.11	4.81
Lakemoor Station 5	200	1.48	1.42	2.90
Lakemoor Station 6	502	1.45	4.04	5.49
Lakemoor Station 7	800	1.82	2.19	4.01

Only at Lakemoor Lift Stations 1 and 6 do average runtimes exceed 5 hours per day. The capacities at these pump stations are becoming marginalized under average conditions and will be exacerbated under wet weather flows. Somewhat less severe conditions exist at the Hale 1 and Fern Lift Station, and at Lakemoor Lift Station 3, 4, and 7 where average pump runtimes exceed 4 hours per day. Caution should be exercised and further evaluation performed prior to connecting additional services to these pumping stations.

Pump drawdown tests were also conducted at each lift station. Pump drawdown tests allow actual installed pumping rates to be verified which can then be compared to each pump's rated capacity to determine whether the pumps are operating as intended by design, or if they are under- or over-performing. Significant deviations could be a result of a worn pump impeller, varying motor speed, partially obstructed pump discharge or force main piping, or an improperly designed installation. The results of the pump drawdown tests are summarized in Table 9-6.

Table 9-6: Lift Station Drawdown Test Results

Lift Station	Pump Rated Capacity	Pump 1 Drawdown Test (gpm)	Tolerance from Rated Capacity	Pump 2 Drawdown Test (gpm)	Tolerance from Rated Capacity
Hale 1	363	384	6%	231	-36%
Hale 2	238	244	3%	362	52%
Waterford	850	834	-2%	762	-10%
Clearwater	60	43	-28%	63	5%
Walnut Glen	284	285	0%	230	-19%
Prairie Woods	265	319	20%	366	38%
Rolling Oaks	200	193	-4%	212	6%
Deer Grove North	532	499	-6%	433	-19%
Rawson Bridge	740	584	-21%	572	-23%
Water's Edge	100	95	-5%	92	-8%
Westridge	236	247	5%	268	14%
Burr Oak	90	155	72%	103	14%
South Shore	200	208	4%	187	-6%
Fern	576	558	-3%	601	4%
Treatment Plant	235	222	-6%	238	1%
Lakemoor Station 1	450	497	10%	461	2%
Lakemoor Station 2	270	212	-21%	212	-21%
Lakemoor Station 3	270	294	9%	283	5%
Lakemoor Station 4	270	278	3%	214	-21%
Lakemoor Station 5	200	167	-17%	163	-19%
Lakemoor Station 6	502	583	16%	305	-39%
Lakemoor Station 7	800	869	9%	704	-12%

9.5 WASTEWATER TREATMENT FACILITY IMPROVEMENTS

The existing wastewater treatment facilities were discussed and assessed in Section 5. The existing features and condition of each treatment process were discussed.

Influent wastewater flow and pollutant loading data at the NMWRD was reviewed for the past three years. Current flows and loads are well within the treatment capabilities of the existing NMWRD wastewater treatment facility. However, some of the equipment at the existing facility is aging and planning should include their rehabilitation or replacement.

Current and projected wastewater flows and pollutant loadings at the treatment facility are summarized in Table 9-1. The treatment facilities also have sufficient capacity to support planned development throughout those areas currently incorporated into the Northern Moraine WRD. It is only as additional areas in the Northern Moraine FPA become annexed to the District that the treatment capacity of the plant will be exceeded.

Current population and flow projections estimate that the NMWRD treatment facility will reach capacity around the year 2025. The logical expansion of capacity at this facility would be to increase the average treatment capacity from 2.0 MGD to 3.0 MGD as has always been planned. In fact, the existing plant was designed in a manner which facilitates this type of expansion through the construction of a third ring to the existing oxidation ditch.

However, the next major improvement at the NMWRD treatment plant is necessitated by the changing regulatory environment, not by growth. The District's new NPDES permit requires that phosphorus removal capabilities be incorporated, that the construction of these upgrades be completed by May 1, 2018, and that full compliance with an effluent limit of 1.0 mg/L total phosphorus be achieved by May 1, 2019. Alternatives to achieve compliance with the new phosphorus limit were evaluated in Section 6, including chemical phosphorus removal and biological phosphorus removal.

The new Northern Moraine NPDES permit was issued on October 23, 2014, has an effective date of November 1, 2014 and will expire on October 31, 2019. The permit includes special conditions related to the Fox River Study Group (FRSG) implementation plan. The special conditions require NMWRD to complete a phosphorus removal feasibility report within 12 months of the effective date of the permit. The study must evaluate options and costs associated with reducing phosphorus concentrations in the plant effluent to 1.0 mg/L as well as 0.5 mg/L. The compliance schedule contained in the District's NPDES permit is summarized in Table 9-7.

Table 9-7: Phosphorus Removal Compliance Schedule

Item	Required date of completion	Completion Date
Interim Report on Phosphorus Removal Feasibility Report	6 months from the effective date of Permit	May 1, 2015
Phosphorus Removal Feasibility Report submitted	12 months from the effective date of Permit	November 1, 2015
Progress Report on FRSG Phosphorus Input Reductions and Implementation Plan	18 months from the effective date of Permit	May 1, 2016
Progress Report on Recommendations of FRSG Implementation Plane	24 months from the effective date of Permit	November 1, 2016
Plans and specifications submitted	30 months from the effective date of Permit	May 1, 2017
Progress Report on Construction	36 months from the effective date of Permit	November 1, 2017
Complete Construction	42 months from the effective date of Permit	May 1, 2018
Progress Report on Optimizing Treatment System	48 months from the effective date of Permit	November 1, 2018
Achieve Annual Concentration and Loading Effluent Limitations for Total Phosphorus	54 months from the effective date of Permit	May 1, 2019

The various improvements at the NMWRD treatment facility were segregated into three categories, including:

- Existing Treatment Facility Rehabilitation
- Regulatory Compliance (Phosphorus Removal)
- Treatment Facility Expansion

Existing Treatment Facility Rehabilitation

It is not anticipated that the existing treatment facility will reach capacity for at least another 10 years. However, there is equipment at the plant that will be reaching the end of its useful service life before the plant needs to be expanded. These improvements are discussed in Section 5 and include:

- Replace the raw sewage pumps
- Upsize a portion of the raw sewage force main
- Replace the influent magnetic flow meter
- Replace the older (40-inch) fine screen

This equipment should be replaced to ensure the reliability of the existing treatment plant to provide uninterrupted and effective treatment. Probable costs associated with these improvements are summarized in Table 9-8.

Table 9-8: Probable Capital Costs – Treatment Facility Rehabilitation

Description	Probable Capital Cost
GENERAL CONDITIONS	$ 93,750
RAW SEWAGE PUMP REPLACEMENT	250,000
INFLUENT FORCE MAIN AND METER REPLACEMENT	100,000
FINE SCREEN REPLACEMENT	250,000
SUBTOTAL CONSTRUCTION	$ 693,750
CONTINGENCY @ 25%	173,440
CONSTRUCTION TOTAL	$ 867,190
ENGINEERING @ 15%	130,100
PROBABLE CAPITAL COST – TREATMENT FACILITY REHABILITATION	$ 997,290

Regulatory Compliance (Phosphorus Removal)

Two approaches were considered for compliance with the 1.0 mg/L phosphorus limit in the new Northern Moraine NPDES permit, chemical phosphorus removal and biological phosphorus removal. In either case, the ability to chemically remove phosphorus would be provided as a safeguard and also to provide polishing as needed to remove that phosphorus not removed biologically to the level necessary for compliance.

The alternatives were evaluated on a life-cycle present value cost basis, as each has differing initial capital expenditures and differing annual operating costs associated with them.

Annual chemical costs associated with sole reliance on chemical phosphorus removal, for the current 2 MGD facility and also for future expansions, are summarized in Table 9-9.

Table 9-9: Phosphorus Removal Chemical Costs

Phase	DAF (MGD)	Phosphorus (lbs/day)	Phosphorus (lbs/year)	$FeCl_3$ (gallons/year)	Estimated Annual Cost
I	2.0	0	-	-	-
2016 Upgrade	2.0	100	36,500	54,750	$ 54,750
II	3.0	150	54,750	82,125	$ 82,125
III	4.5	225	82,125	123,188	$ 123,188
IV	6.0	300	109,500	164,250	$ 164,250

Either alternative (chemical or biological P removal) will result in additional sludge production. The aeration costs for digesting this sludge in the aerobic digesters must also be considered in the life cycle cost effective analysis. Table 9-10 summarizes the estimated sludge production from the secondary process under three conditions – current design, chemical phosphorus removal and biological phosphorus removal.

Table 9-10: WAS Production – Chemical-P Removal versus Biological-P Removal

Phase	Design Average Flow (MGD)	WAS Production Without P Removal (lbs/day)	WAS Production With Chemical P Removal (lbs/day)	WAS Production Using Biological P Removal (lbs/day)
I	2.0	1,600	-	-
2016 Upgrade	2.0	1,600	1,978	1,970
II	3.0	2,400	2,966	2,954
III	4.5	3,601	4,450	4,432
IV	6.0	4,801	5,933	5,909

It can be seen that the amount of sludge produced is essentially equal under either phosphorus removal alternative. However, in either case sludge production will increase by approximately 23 to 24 percent above that produced by the existing processes without phosphorus removal.

Chemical phosphorus removal would require construction of chemical feed facilities which would include a building to house the chemical storage tank and feed pumps, which must include sufficient containment to prevent spills from escaping the building. Probable capital costs associated with construction of the Chemical Feed Building are presented in detail in Section 6, and are summarized in Table 12.

Biological phosphorus removal would require construction of an anaerobic selector tank upstream of the existing oxidation ditch, sized at 150,000 gallons to provide 1.5 hours detention time at the current design flow of 2.0 MGD. Probable capital costs associated with the selector tank are presented in detail in Section 6, and are also summarized in Table 9-11.

Table 9-11: Probable Capital Costs - Phosphorus Removal

Description	Chemical Feed Building	Anaerobic Selector Tank
CONSTRUCTION SUBTOTAL	$ 358,015	$ 589,800
GENERAL CONDITIONS	44,750	73,725
CONTINGENCY @ 25%	100,690	165,880
CONSTRUCTION TOTAL	503,455	829,405
DESIGN ENGINEERING @ 7.5%	37,760	62,205
CONSTRUCTION ENGINEERING @ 7.5%	37,760	62,205
TOTAL CAPITAL COSTS	**$ 578,975**	**$ 953,815**

The life-cycle cost effective analysis comparing chemical phosphorus removal to biological phosphorus removal at the NMWRD treatment facility is summarized in Table 9-12.

Table 9-12: Cost Effective Analysis – Phosphorus Removal

	Chem-P	Bio-P
Capital Cost	$ 578,975	$ 1,532,790
Annual Costs:		
Chemicals	$ 54,750	$ 11,467
Blower Energy	40,887	50,323
Mixing Energy	-	18,291
Subtotal – Annual Costs	$ 95,637	$ 80,080
Discount Rate	4%	4%
Planning Period (years)	20	20
Present Value – Annual Costs	$ 1,299,740	$ 1,088,317
Total Present Value	$ 1,878,715	$ 2,621,107

The analysis shows that biological phosphorus removal is not the cost-effective solution at the NMWRD treatment facility at this time. When the facility is expanded to a treatment capacity of 3 MGD, the addition of the third ring to the oxidation ditch will provide the flexibility to achieve a degree of biological phosphorus removal through modification to the operation and control of oxygen in the multi-channel ditch. Even then, the chemical feed facility would be used for polishing as needed to comply with the NPDES effluent limits.

Treatment Facility Expansion Phasing

The Phase I Expansion completed in 1998 set precedence for the future expansion of the facility. While that particular expansion-phasing plan only contemplated one additional phase increasing the capacity from 2.0 to 3.0 MGD, it was concluded that future phases should be designed to parallel the processes as much as practical.

The 2004 Facility Plan Update included four future phased expansion up to an ultimate treatment capacity of 10.0 MGD. Future population and flow projections have been revised for this 2014 Facility Plan Update, which now includes upgrades only up to an ultimate capacity of 6.0 MGD as shown in Table 9-13.

Table 9-13: Phased NMWRD Treatment Facility Expansion Plan - Design Flows

Construction Phase	2004 Facility Plan Design Average Flow (MGD)	2014 Facility Plan Design Average Flow (MGD)
Phase I (Existing)	2.0	2.0
2016 Upgrades	n/a	2.0
Phase II	3.0	3.0
Phase III	4.5	4.5
Phase IV	6.0	6.0
Phase V	10.0	n/a

The treatment facility is currently operating at 53% of its hydraulic design, 64% of its BOD loading design, 47% of its TSS loading design, and 54% of its ammonia loading design. It is recommended that once the facility plan is approved that the District proceed with the 2017 P Removal (phosphorus removal) design to ensure adequate treatment capabilities are available for continued compliance with the updated NPDES permit. Phases II, III and IV will need to be considered once the facility reaches 80% of its design capacity of the preceding phase (currently 1.6 MGD). It is currently projected that the Phase II expansion will not need to be completed within the 20-year planning period except under the most optimistic high rate growth assumptions. Probable capital costs associated with the Phase II expansion of the NMWRD treatment facility are summarized in Table 9-14.

Table 9-14: Probable Capital Costs – Phase II Expansion

Description	2004 Probable Cost	2015 Probable Cost
CONSTRUCTION SUBTOTAL	$ 9,076,500	$ 13,252,300
GENERAL CONDITIONS	1,134,700	1,600,000
CONTINGENCY (25%)	2,269,200	3,313,100
PROBABLE CONSTRUCTION COST	$ 12,480,400	$ 18,165,400
DESIGN ENGINEERING @ 7.5%	936,100	1,362,500
CONSTRUCTION ENGINEERING @ 7.5%	936,100	1,362,500
TOTAL CAPITAL COST	**$ 14,352,600**	**$ 20,890,400**

Probable costs for the various improvements at the NMWRD treatment facility under each of the three categories of improvements are summarized in Table 9-15.

Table 9-15: Probable Capital Costs – Phased Treatment Facility Improvements

Description	Rehabilitation	Regulatory Compliance	Phase II Expansion
CONSTRUCTION SUBTOTAL	$ 600,000	$ 358,015	$ 13,252,300
GENERAL CONDITIONS	93,750	44,750	1,600,000
CONTINGENCY (25%)	173,440	60,415	3,313,100
PROBABLE CONSTRUCTION COST	$ 867,190	$ 463,180	$ 18,165,400
DESIGN ENGINEERING @ 7.5%	65,050	34,740	1,362,500
CONSTRUCTION ENGINEERING @ 7.5%	65,050	34,750	1,362,500
TOTAL CAPITAL COST	**$ 997,290**	**$ 532,660**	**$ 20,890,400**

9.6 IMPLEMENTATION PLAN

The Northern Moraine NPDES permit includes a compliance schedule which includes milestones related to phosphorus removal planning, design, construction and start-up. The schedule requires that designs for phosphorus removal be submitted to the IEPA by May 1, 2017 and construction be completed by May 1, 2018. Full compliance with the 1.0 mg/L phosphorus limit must be achieved by May 1, 2019. Interim progress reports must also be submitted, including an interim report of the Phosphorus Removal Feasibility Study due on May 1, 2015. The final Phosphorus Removal Feasibility Study must be submitted prior to November 1, 2015.

The various improvements recommended for rehabilitation of the existing NMWRD treatment facility could be implemented as part of the phosphorus removal construction project. This would place the replacement pumps online in 2018, at which time they will be 20 years old.

The Phase II Expansion is not anticipated to be required until at least the year 2025 as projected under the growth assumptions discussed in Section 2.

The construction of the Interim Solution Collection System improvements would not be required until either the current capacity of Lift Station 7 is reached, or until the surplus capacity currently available in the 24-inch Route 176 West Interceptor is exhausted (4,425 PE currently available). The growth projections presented for the Central, Waterford, Northwest and Northeastern Basins indicate the interceptor's capacity being reached in the year 2015.

In consideration of the remaining service life of the existing facilities, regulatory requirements, and projected growth through the Northern Moraine FPA, the phased Implementation Plan for this 2014 Facility Plan Update are summarized in Table 9-16.

Table 9-16: Phased Implementation Plan

Description	Probable Capital Costs ($ millions)						
	2015	2016	2017	2018	2019	2020 to 2029	2030 to 2039
TREATMENT FACILITY							
Rehabilitation / Replacement		$ 0.07	$ 0.93				
Phosphorus Removal		$ 0.04	$ 0.50				
Phase II Expansion						$ 20.90	
COLLECTION SYSTEM							
Interim Solution Eastside System						$ 13.52	$ 3.11
Annual rehabilitation / replacement	$ 0.62	$ 0.62	$ 0.62	$ 0.62	$ 0.62	$ 6.20	$ 6.20
LIFT STATIONS							
Annual rehabilitation / replacement	$ 0.33	$ 0.33	$ 0.33	$ 0.33	$ 0.33	$ 3.30	$ 3.30
TOTAL PROBABLE CAPITAL COSTS	$ 0.95	$ 1.06	$ 2.38	$ 0.95	$ 0.95	$ 43.92	$ 12.61

Replacement expenditures for sanitary sewers was estimated at approximately $617,300 annually. Attention should first be given older areas in the collection system and to those areas where I/I has been detected. Replacement costs for lift stations and force mains was estimated to be approximately $324,800 per year.

The scheduling of these improvements are initial estimates employed for planning purposes. The phosphorus removal improvements at the NMWRD treatment facility must be completed by May 1, 2018 as required in the District's NPDES permit. Scheduling for Phase II expansion and Interim Solution Collection System will become known with greater certainty over time as actual development throughout the Northern Moraine FPA occurs.

9.7 USER RATES AND PROJECT FUNDING

The District recently completed an internal User Rate Study that concluded that the current rate schedule was insufficient to cover current operating expenses, and adopted a 5-year rate plan to cover operation, maintenance, and replacement costs.

The 2017 P Removal Improvements might be funded out of capital project cash reserves.

A more detailed user rate and connection fee study will be required to assess how to cover the debt service that would be incurred for the Phase II Expansion. Planning for that expansion will be required when average flows to the treatment facility approach 1.6 MGD, and the expansion will likely involve borrowing in the form of an IEPA revolving fund low-interest loan. In any case, it is not expected that planning for the Phase II Expansion will be required for at least another 10 to 20 years.

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