Comprehensive Utilities Plan

Village of Sister Bay, Wisconsin

April 2008

Volume I of II
REPORT

SEH

Multidisciplined. Single Source.
Trusted Solutions.

REPORT TABLE OF CONTENTS

LIST OF TABLES

LIST OF APPENDICES

Appendix

CHAPTER 1

INTRODUCTION

The Village of Sister Bay is a community of approximately 900 persons located in northern Door County. Sister Bay is in northeastern Wisconsin, approximately 25 miles north of Sturgeon Bay, and 60 miles north of Green Bay. The Sister Bay water and wastewater utilities provide water and sewer service to residences and businesses within the Village limits and the Liberty Grove Sanitary District No. 1. The Village and Sanitary District also operate and maintain a very limited storm sewer system.

The Village is surrounded almost entirely by the Town of Liberty Grove. The Liberty Grove Sanitary District No. 1 (LGSD No. 1) is located immediately north of the Village corporate limit, and is provided with sanitary sewer and water service from the Village of Sister Bay. The Village of Ephraim is located less than one half mile to the south of the Village's corporate limit.

The Village of Sister Bay's location as a major seasonal tourist center offers significant potential for future growth and development. Therefore, proper planning is essential to coordinate the improvement and expansion of municipal utility facilities with short-term as well as long-term needs of the community.

1.1 PURPOSE

This report summarizes the results of a comprehensive utility planning study for the Village of Sister Bay, LGSD No. 1, and adjacent areas within the Town of Liberty Grove. This comprehensive utility planning study included a review and evaluation of the following water-related utility systems:

- Drinking water
- Sanitary sewer collection
- Storm water

The primary purposes of the study were to evaluate the existing and future water and sewer needs of the existing service area, and the utility infrastructure improvements and expansion required to serve current and future planning area residents. In addition, a review and evaluation of the area's existing storm water infrastructure was performed, and a planning area storm water management plan was developed.

Present and future needs of the Sister Bay comprehensive utility planning area have been evaluated, and recommendations made concerning improvements necessary to maintain an adequate level of water, sewer and storm water service. This report will serve as a plan to guide future expansion of the three utility systems.

1.2 SCOPE

The planning approach used for the study began with the identification of existing planning area conditions, and an evaluation of service area needs and characteristics. Current and future water-related needs were evaluated over a 20-year planning period extending to the year 2025.

In planning for the long-term growth anticipated for the study planning area, identifying the physical characteristics affecting the water-related infrastructure systems was performed. Chapter 2 summarizes the physical characteristics of the planning area for this study. Population, community growth, and water consumption projections serve as the foundation for evaluating and identifying recommended improvements to the water and sanitary sewer systems. Chapter 3 discusses existing and expected future land uses and community growth. The assumptions and conclusions presented in Chapter 3 were used to develop projections of water requirements that are presented in Chapter 4.

A review of existing water system facilities is presented in Chapter 5. Chapter 6 summarizes the evaluation of the water system. A summary of recommended water system improvements is presented in Chapter 7.

A review of the existing sanitary sewer collection system is presented in Chapter 8. Chapter 9 summarizes the evaluation of the sanitary sewer system. A summary of recommended sanitary sewer system improvements is presented in Chapter 10.

A review of the existing storm water system is presented in Chapter 11. Chapter 12 summarizes the evaluation of the storm water facilities and storm sewer system. A recommended storm water management plan is presented in Chapter 13.

Chapter 14 presents the overall recommended Comprehensive Utility capital improvements plan (CIP) for the Sister Bay planning area.

1.3 SCHEDULING OF FUTURE IMPROVEMENTS

Based on input from Village staff, a recommended CIP for the infrastructure systems has been developed. The CIP for the Sister Bay Comprehensive Utilities Plan is broken down into short-term and long-term improvements. Short term improvements generally include improvements that are needed to address existing deficiencies. Short term improvements can also include improvements to accommodate future development in areas where development is relatively cost effective, such as areas that do not need to be served by a new water system pressure zones or sanitary sewer lift stations and force mains. Long term improvements typically include providing service to future expansion areas that are located farther from the existing infrastructure systems and are more expensive to construct.

The timing of recommended future infrastructure improvements will be influenced by a number of parameters. Items such as the location of development pressure in specific areas, aging facilities and/or facilities which are undersized, availability of funds, etc., all play a role in the timing of future improvements. Because of the factors involved, it is difficult to accurately predict the timing of future improvements, especially those which may occur far into the future. However, some locations within the Comprehensive Utilities Planning area are more likely to experience rapid development than others.

Because infrastructure needs can change with time, municipal utility system planning is a continuous process. Therefore, the longer term projections and improvements discussed in this Comprehensive Utilities Plan report should be reviewed, re-evaluated, and modified, as necessary, to assure the adequacy of future planning efforts. Proper future planning will help assure that utility infrastructure system expansion is coordinated and constructed in the most effective manner.

1.4 COMPREHENSIVE PLAN USE AND FUTURE REFERENCE

This Comprehensive Utilities Plan has been prepared as a tool to guide the Village of Sister Bay in the siting and sizing of future water, sanitary sewer and storm water infrastructure improvements. While the plan represents the current recommended expansion of the Sister Bay infrastructure systems to serve the identified planning area, future changes in land use, water demands, or customer characteristics could substantially alter the implementation of the plan. For this reason, it is recommended that the Comprehensive Utilities Plan be periodically reviewed and updated using Village planning information to reflect the most current projections of Sister Bay area growth and development.

This Comprehensive Utilities Plan is a guidance document that details existing conditions and recommendations for the future. The recommendations are based on future conditions as perceived in 2006. Estimating future conditions requires making educated assumptions about unknown parameters. As the future unfolds, these assumptions may or may not prove to be correct. Accordingly, the recommendations are intended to be used for guidance purposes only, and are generally written in an “if, then” format. In other words, if the assumptions about the future are correct, and if the Village wishes to accomplish a certain goal by some future time, then a certain course of action is necessary in order to accomplish this goal. The course of action which is anticipated to be necessary is typically presented in the form of a recommendation.

The recommendations will be implemented over time. The schedule for implementation is driven by the pace of development. As development progresses and at the time of a development proposal, the Village should look to the current form of the Comprehensive Utilities Plan to initiate the recommendations that will best serve the future development that will fill in around the proposal at hand.

As time progresses, additional information will become available and events will shape the development of the Sister Bay area. The Village’s Utilities Plan must be dynamic in response; it should be studied and used for Village infrastructure project planning and budgeting, but also adjusted to conform to the changes and knowledge that will come with time. Updates should be made on a regular basis. Due to the rapid rate of growth and development expected within the planning area, it is recommended that the Sister Bay Comprehensive Utilities Plan should be reviewed and updated (as necessary) every five years.

CHAPTER 2

EXISTING PLANNING AREA CHARACTERISTICS

This chapter summarizes the pertinent characteristics of the identified Comprehensive Utility Planning Area for this study. To maintain consistency between this and other area planning efforts, the results of previous planning efforts were reviewed. In particular, the information presented in this chapter has been summarized from the Village of Sister Bay's 20-Year Comprehensive Plan, completed in October 2003, as prepared by the Bay-Lake Regional Planning Commission.

2.1 INTRODUCTION

The Village of Sister Bay is located near the northern end of the Door County Peninsula at the intersections of State Highways 42 and 57. These highways provide access to the Village of Sister Bay from the city of Sturgeon Bay which is located 27 miles south. Historically, the village has derived much of its revenue from fishing and tourism. Presently, the village serves as an important recreational and residential center for northern Door County.

The Sister Bay Comprehensive Utility Planning Area, illustrated on Figure 2-1, contains a variety of natural resources. The principal elements of the area's natural resources that impact the utility planning project include:

- Climate
- Topography
- Geology
- Soils
- Natural areas, including woodlands, wetlands, and water resources

Knowledge and recognition of these elements and their interrelationships are essential for the ability to adequately identify and evaluate their impact on overall water resource planning for the Sister Bay area. The remainder of this chapter summarizes the planning areas natural resources.

2.2 CLIMATE

The climate of the Village of Sister Bay and the surrounding town of Liberty Grove is significantly affected by Green Bay and Lake Michigan. The cool waters of the lake and bay delay the onset of spring weather, while the relatively warm surface water in autumn delays the occurrences of early frosts. Summers, on average, are mild due to the planning area's proximity to the water which moderates daily temperature extremes.

The annual average temperature for the Sister Bay area approximately is 43 degrees Fahrenheit. January has the lowest average monthly temperature of 17 degrees, while July has the highest average temperature of 66 degrees. Frost generally leaves the ground by mid-May, and usually returns during the first week of October. The average growing season typically lasts about 135 days. Ice forms on Green Bay in late December, and generally covers the entire bay by mid-January. During mild winters, the bay does not freeze completely. Ice breakup usually occurs in early April.

The normal annual total precipitation for the area is 28.9 inches. The lowest monthly average of 0.97 inches occurs in February, while the highest of 3.60 inches occurs in June. More than one-half the average annual precipitation falls between May and September. The first half of June and middle of August are the likely time that the heaviest rainfall events occur. The end of August is normally the driest period of the year.

2.3 GEOLOGY

Geology is divided into two categories: glacial (or Pleistocene geology) and bedrock geology. Glacial geology is the material that the most recent glaciers deposited in the area. Bedrock geology is the material beneath the glacial geology.

2.3.1 Bedrock Geology

The bedrock units which underlie the Sister Bay planning area range in age from Precambrian to Silurian. The oldest are impermeable crystalline rock of Precambrian age at depths that average more than 1,500 feet below the ground surface. These are overlain by consolidated sedimentary rocks of Cambrian, Ordovician, and Silurian ages. The sedimentary rocks are solidified marine sediments that dip to the southeast at approximately 45 feet per mile. The rock formations deepen toward the southeast.

Silurian dolomite, which is the uppermost bedrock in the area, is exposed in outcroppings throughout the planning area but primarily along the bluffs near the Green Bay shoreline. This dolomite reaches in thickness up to almost 600 feet. Below the dolomite (commonly referred to as Niagara), is a shale formation known as Maquoketa. It reaches a maximum thickness of 450 feet. The Maquoketa Shale overlies a dolomite formation, termed Platteville-Galena, which is approximately 500 feet in thick. This rock formation overlies Cambrian sandstones which are 450 feet thick. All of these formations overlie Precambrian igneous rocks that form the bottom bedrock unit.

The Silurian or "Niagara" dolomite is perhaps the most notable and influential bedrock unit within the planning area. It makes up the landform known as the "Niagara Escarpment". The Niagara Escarpment is a cuesta which is a gently sloping plain that is terminated on one side by a steep slope (refer to Map 2.4 in the *20-Year Comprehensive Plan Report*). The gentle slope of the Niagara Escarpment dips to the southeast throughout much of the planning area but is somewhat difficult to observe due to glacial deposits. It does emerge, however, along the shoreline of Green Bay as a prominent feature. Because of the dolomite's proximity to the ground surface, especially in the western portions of the planning area, little agriculture, with the exception of orchard cultivation, is practiced. The Silurian dolomite is also the primary source of groundwater for the planning area.

2.3.2 Glacial Geology

The last glacial ice, which left the planning area approximately 10,000 years ago, modified the bedrock surface by scouring highlands and depositing this material in lowlands created by preglacial erosion. Three major types of glacial features are identifiable within the planning area: these include glaciolacustrine deposits, ground moraines, and end moraines (refer to Map 2.4 in the *20-Year Comprehensive Plan Report*). Glaciolacustrine deposits are composed primarily of sand, silt, and clay. These sediments were deposited by glacial predecessors of Green Bay and Lake Michigan. Shorelines of these early lakes fluctuated 20 to 60 feet above the present Lake Michigan level.

The eastern portion of the planning area is located upon glaciolacustrine deposits, indicating these areas were inundated by water several thousand years ago. Most of these deposits are located adjacent to Lake Michigan extending from the northeast to southeast area of the planning area. End moraines are glacial landforms composed of unsorted sand, gravel, cobbles, and boulders that were deposited at the terminus of the glacial ice. Acting as an enormous bulldozer, the ice pushed and mounded this material into substantial hills. The area of end moraine deposits is located primarily within the north central portion of the planning area.

Ground moraines, like end moraines, are composed of unsorted material; however, ground moraines are considerably thinner deposits and have an irregular, gently rolling surface as compared to the more pronounced topography of end moraines. Ground moraines are scattered throughout the planning area. Sand dunes are also present along the Lake Michigan side of the area and drumlins have been identified in the east central portion of the area. The dominant role glacial ice played in shaping the physical setting of the Liberty Grove area, both in terms of deposition, is evidenced by the various topographic features of the area.

2.4 SOIL LIMITATIONS

2.4.1 General Soils Description

Soils are grouped into general soil associations which have similar patterns of relief and drainage. These associations typically consist of one or more major soils and some minor soils. The general character of the soils of the planning area is largely the result of various types of glacial deposits overlying the Silurian dolomite. Within the study planning area, there are two general soils associations (refer to Map 2.5 in the *20-Year Comprehensive Plan Report*):

Carbondale-Seelyeville-Markey. Soils in this association consist of very deep, very poorly drained soils in outwash plains, lakes plains and glacial moraines. The Carbondale series consists of very deep, very poorly drained soils formed in organic deposits more than 51 inches thick on ground moraines, outwash plains and lake plains. These soils have moderately slow to moderately rapid permeability. Slopes range from zero to two percent.

The Seelyeville series consists of very deep, very poorly drained soils that formed in organic materials more than 51 inches thick. These soils are on glacial outwash plains, valley trains, flood plains, glacial lake plains and glacial moraines. They have moderately rapid to moderately slow permeability. Slopes are zero to 15 percent.

The Markey series consists of very deep, very poorly drained organic soils. They formed in herbaceous organic material 16 to 51 inches thick overlying sandy deposits in depressions on outwash plains, lake plains, flood plains, river terraces valley trains and moraines. Permeability is moderately slow to moderately rapid in the organic layers and rapid or very rapid in the sandy material. Slopes range from zero to two percent.

Longrie-Summerville-Kolberg. These soils are shallow to deep, level to moderately steep, well drained, and have a sandy loam or loam subsoil over sandy loam or fine sandy loam till or dolomite bedrock. The Longrie series consists of moderately deep, well drained soils formed in loamy glacial deposits underlain by limestone bedrock at a depth of 20 to 40 inches on ground moraines, glacial lake benches and terraces. Permeability is moderate. Slopes range from zero to 25 percent.

The Summerville series consists of shallow, well drained soils formed in loamy materials overlying limestone on ground moraines, end moraines, and glacial lake benches. Permeability is moderate. Slopes range from zero to 45 percent.

The Kolberg series consists of well drained soils moderately deep to limestone. These upland soils formed in thin, loamy deposits and the underlying moderately fine or fine textured glacial till. Permeability is moderately slow or slow. Slopes range from zero to 12 percent.

2.4.2 On-Site Sewage Disposal Systems

20-Year Comprehensive Plan Report Map 2.6 depicts soil limitations for septic tank absorption fields. These are subsurface systems of tile or perforated pipe that disperse effluent from a septic tank into the natural soil. If the degree of soil limitation is slight, soils are favorable for absorption fields, and limitations are minor and easily overcome. Soils with a moderate rating indicate that soil properties or site features are generally unfavorable for absorption fields, but limitations can be overcome by special planning and design. A severe rating indicates that soil properties or site features are so unfavorable or difficult to overcome that major soil reclamation, special designs, or intensive maintenance is required.

Soils that have slight limitations for absorption fields generally are well-drained and have sufficient depth before encountering bedrock or groundwater. They are located primarily in the western portion of the planning area, east of Old Stage Road. Soils with moderate and severe limitations generally have insufficient depths to bedrock or groundwater, percolate slowly, and are subject to flooding. Soils with moderate limitations are generally located throughout the planning area, while soils with severe limitations are encountered to the north and south of the Village in the planning area. Without consideration of the properties of these soils, on-site wastewater treatment systems may fail and collection systems may require expensive and frequent maintenance. Factors which are considered when evaluating soils for on-site waste systems include:

High or Fluctuating Water Table	When groundwater is near the soil surface, proper filtering cannot take place and often results in on-site systems either backing up into the home or contamination of groundwater. In addition, construction techniques used to de-water systems are costly.
Bedrock	Large stones or bedrock near the soil surface may hinder excavation and considerably increase the cost of construction. In addition, conventional on-site septic systems cannot function properly, which may result in wastewater passing through the cracked bedrock and contaminating the groundwater.
Soil Permeability	Permeability refers to the rate at which water flows through the soil. When passage in too rapid, groundwater can become polluted. If it is too slow, the soils can become saturated and effluent ponding may result.
Flooding	On-site waste disposal systems that are located within a floodplain can result in problems. As water levels rise during periods of flooding, the system become saturated and results in untreated solid and liquid waste being discharged into the ground or surface waters.

New technologies for private wastewater treatment systems are allowed under the revised COMM 83 health and safety code. The code will allow the use of soil absorption systems on sites with at least six inches of suitable native soil. The revised code also gives property owners the opportunity and flexibility to meet environmental performance standards with several treatment technologies. It allows for better planning and land use because it assures that every residentially-zoned lot can be used for the purpose intended by the local zoning board. The code will allow for infill development where it was not permitted previously due to lack of access to an improved septic system.

Housing and population density will likely increase due to the revised COMM 83 code. This in turn may increase the need for land use planning and integration of environmental corridors to address the adverse impacts related to development. Planning along with land use controls such as zoning, will help achieve more efficient development patterns.

2.4.3 Prime Agricultural Lands

Most of the land within the planning area is classified as prime agriculture land with minimal modifications. These lands are located all around the Village of Sister Bay, primarily away from the shoreline. Two classes of prime farmland are identified; those areas where all land is prime farmland and those areas that are considered prime farmland only where drained. The rest of the planning area is classified as not prime farmland. Map 2.7 in the *20-Year Comprehensive Plan Report* shows these areas of prime farmland.

2.4.4 Basements

Many of the soils in the planning area have severe limitations for dwellings with basements. According to the *Soil Survey of Door County*, severe limitations indicate one or more soil properties or site features are so unfavorable or difficult to overcome that a major increase in construction effort, special design, or intensive maintenance is required. For some soils rated severe, such costly measures may not be feasible.

In the planning area, the main limitation for dwellings with basements is depth to bedrock or wetness. The soils in the planning area that have severe limitations are located along the in the southwestern portion of the planning area and in a band from the northwest to the southeast in the planning area north of the Village of Sister Bay. The rest of the planning area is rated either moderate or slight. These areas are mostly located in the central part of the planning area. Map 2.8 in the *20-Year Comprehensive Plan Report* shows these limitations.

2.5 TOPOGRAPHY

Topography of the planning area is controlled primarily by the underlying bedrock with two distinct types of relief. The first of these includes an area with relief in excess of 700 feet USGS. It is located within the extreme southwest portion of the planning area. This area is characterized by relatively level tops, similar to plateaus, with steep slopes to the west or north. Many of the steep slopes are near vertical bluffs, especially in the areas immediately adjacent to Green Bay shoreline (refer to Map 2.9 in the *20-Year Comprehensive Plan Report*). These areas are undoubtedly the most obvious in terms of topographic expression within the Village and planning area.

The second group of topographic features includes the eastern portions of the planning area. This large area is characterized by a flat to gently rolling land surface occasionally marked by small depressions. The area slopes gently to the southeast.

2.6 WATER RESOURCES

There are no lakes or named streams within the planning area; however, the western edge of the planning area is adjacent to the bay of Green Bay. The direction of precipitation runoff is primarily southeasterly towards Lake Michigan for the majority of planning area. Runoff into Green Bay is limited to the Village area and a zone along the coast.

2.6.1 Watersheds and Sub-Watersheds

The Village of Sister Bay lies within the Upper Door County watershed. Within this watershed there are four sub-watersheds: Lake Michigan watershed; Green Bay watershed; Three Springs Creek watershed; and the Ephraim Creek watershed.

The Lake Michigan watershed covers the eastern half of the planning area. The Green Bay watershed covers the Village and the western part of the planning area. Three Springs Creek covers the northeastern part of planning area and the Ephraim Creek watershed covers a small portion in the southwest part of the town of Liberty Grove. Map 2.10 in the *20-Year Comprehensive Plan Report* shows these sub-watersheds in the planning area.

2.6.2 Groundwater

In Wisconsin the primary sources of groundwater contamination are agricultural activities, municipal landfills, leaky underground storage tanks, abandoned hazardous waste sites, and spills. Septic tanks and land application of wastewater are also sources for possible contamination. The most common groundwater contaminant is nitrate-nitrogen, which comes from fertilizers, animal waste storage sites and feedlots, municipal and industrial wastewater and sludge disposal, refuse disposal areas, and leaking septic systems.

Groundwater within the study area is derived primarily from the Silurian dolomite aquifer. Well depths range from 60 to 700 feet with yields as high as 1,200 gallons per minute. Water from the Silurian dolomite is a very hard calcium magnesium bicarbonate type with varying concentrations of iron and nitrate. The dolomite has numerous joints and crevices which allow water to move relatively easily through the rock. Pollutants may also enter the groundwater supply via these fractures. The dolomite aquifer is recharged by surface seepage of direct precipitation and snowmelt.

2.6.3 Shoreland Corridors

Coastal areas within the study boundaries include the steep dolomite bluffs adjacent to the shoreline of Green Bay. There are approximately seven miles of Great Lakes shoreline within the planning area. This large amount of shoreline makes residential development very desirable.

2.6.4 Floodplains

Floodplains are often viewed as valuable recreational and environmental resources. These areas provide for storm water retention, groundwater recharge, and habitat for various kinds of wildlife unique to the water. The planning area contains one small area of floodplain on the eastern edge of the planning area adjacent to Three Springs Creek (refer to Map 2.11 in the *20-Year Comprehensive Plan Report*). Development permitted to take place in these areas is susceptible to storm damage and can have an adverse effect on water quality and wildlife habitat. In addition, it can also result in increased development and maintenance costs such as: providing floodproofing, repairing damage associated with flooding and high water, increased flood insurance premiums, extensive site preparation, and repairing water related damage to roads, sewers, and water mains.

Some communities have special ordinances for buildings within the floodplain for remodeling and expanding. New expansions may have to be compliant to the rules of floodplain construction. As a result, the state of Wisconsin requires that counties, cities and villages adopt shoreland/floodplain zoning ordinances to address the problems associated with development in floodplain areas. Development in shoreland areas is generally permitted, but specific design techniques must be considered. Development in floodplain areas is strictly regulated and in some instances is not permitted.

For planning and regulatory purposes, the floodplain is normally defined as those areas, excluding the stream channel, that are subject to inundation by the 100-year recurrence interval flood event. This event has a one percent chance of occurring in any given year. Because of this chance of flooding, development in the floodplain should be discouraged and the development of park and open space in these areas encouraged. The authority to enact and enforce these types of zoning provisions in counties is set forth in Chapter 59.97 of the Wisconsin Statutes and Wisconsin Administrative Code NR 116. This same authority is also vested to cities and villages in Chapter 62.23 of the Wisconsin Statutes.

2.6.5 Wetlands

According to the Wisconsin DNR, wetlands are areas where water is at, near, or above the land surface long enough to be capable of supporting aquatic or hydrophilic vegetation. Other common names for wetlands are swamps, bogs, or marshes. Wetlands serve as a valuable natural resource. They provide scenic open spaces in both urban and rural areas. Wetlands act as natural pollution filters, makings many lakes and streams cleaner and drinking water safer. They act as groundwater discharge areas, and retain floodwaters. Finally they provide valuable and irreplaceable habitat for many plants and animals.

Because of the importance of wetlands, there are strict state and federal regulations regarding wetlands. Wisconsin Administrative Codes NR 115 and NR 117 fall under the jurisdiction of the Wisconsin Department of Natural Resources and mandate that shoreland wetlands be protected in both the rural and urban areas of the state. In the unincorporated areas, NR 115 provides the legislation to protect wetlands of five acres or more that are within the jurisdiction of county shoreland zoning ordinances. This wetland provision would be applicable in the Village of Sister Bay planning area.

NR 117 provides for the protection of wetlands within an incorporated community as provided by the Village's zoning ordinance and state and federal regulations. Wetlands within the planning boundaries include an extensive area along the eastern and southern boundaries and three small areas within the Village. Map 2.12 in the *20-Year Comprehensive Plan Report* shows the WDNR inventoried wetlands greater than two acres. It should be noted that all wetlands, no matter how small, are subject to WDNR and possible federal regulations if they meet the state definition of wetlands.

CHAPTER 3

POPULATION AND COMMUNITY GROWTH

This chapter summarizes the planning assumptions made regarding future service area characteristics for the Village of Sister Bay and the immediate surrounding area. To maintain consistency between individual planning efforts, the results of previous planning efforts were reviewed. The input received from local officials and Utility staff members was also considered and incorporated.

3.1 POPULATION

There is generally a close relationship between a community's population, total water consumption volumes and wastewater discharge flows. Future water usage can be expected to generally reflect future changes in service area population. Similarly, commercial, public, and industrial water consumption will also tend to vary proportionately with the growth of the community.

The Village of Sister Bay has experienced a steady increase in population since 1920. The Village's population according to 2000 Census Bureau data was 886. Since 1990, Sister Bay's permanent population grew an average of almost 3 percent per year. Table 3-1 summarizes past trends and projected future population of the Village of Sister Bay. Future population estimates were based on projections from the *20-Year Comprehensive Plan Report*. Table 3-2 summarizes population changes in Door County communities since the previous census was conducted.

Current projections indicate the Village's total permanent population is expected to increase to approximately 1,160 by the year 2015, and 1,400 by the year 2025. Population projections for the existing service area of the Liberty Grove Sanitary District and surrounding planning area were also developed. For this study, it was assumed the total permanent population served by the Sister Bay Water Utility by the year 2025 will be approximately 2,000.

3.2 EXISTING LAND USE

For this study, an existing Village zoning map and Town land use map were reviewed. From this existing data, the Comprehensive Utility Planning Action Committee prepared a Comprehensive Utility Planning map, previously illustrated in Figure 2-1. This map represents the nature and extent of existing development within the Sister Bay area, as well as planned land uses outside the Village and Sanitary District. Older sections of the Village consist largely of single family residential uses and the Central Business District. New single family and multi-family residential development in recent years has occurred throughout the Village and Sanitary District.

The Village of Sister Bay currently covers approximately 1,650 acres; LGSD No. 1 encompasses approximately 275 acres. Within this area, 13 land use categories have been identified and locations illustrated on Figure 2-1. A summary of the current land use/zoning area is presented in Table 3-3. A detailed study of the planning area land uses was performed by the Bay Lake Regional Planning Commission, and the findings were presented in the *20-Year Comprehensive Plan Report.*

TABLE 3-1

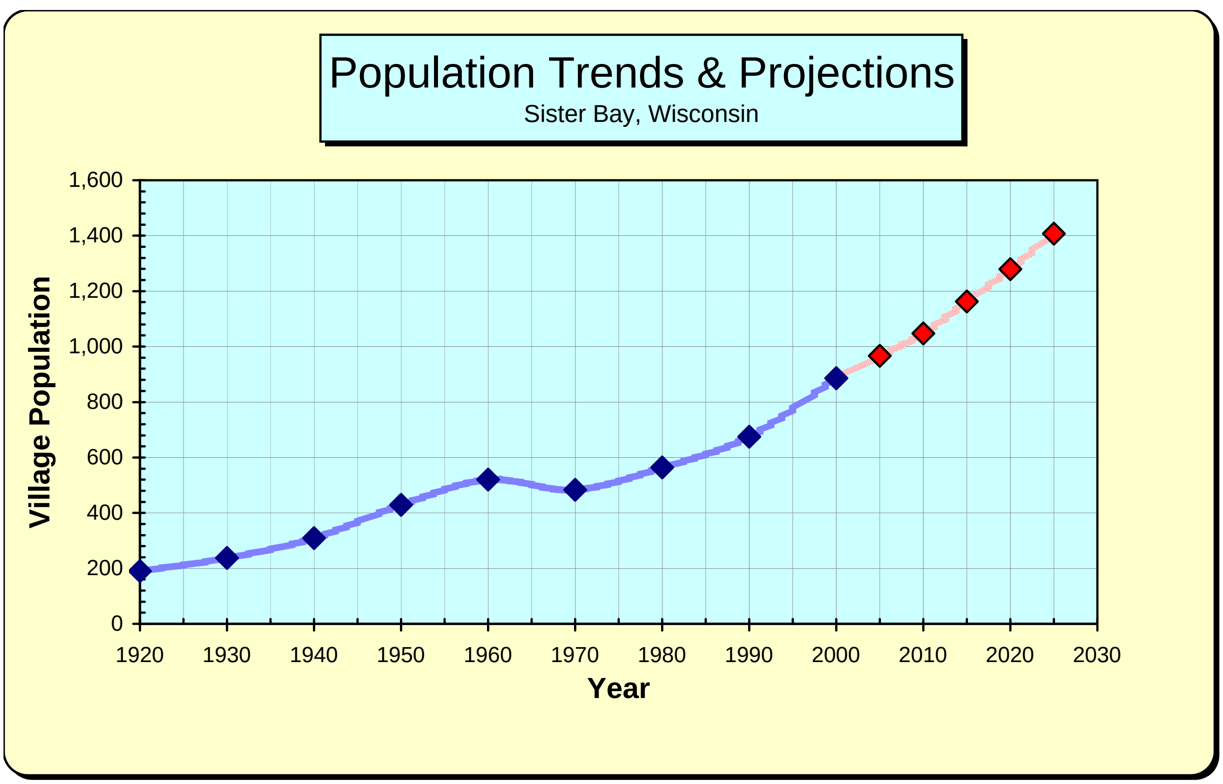
POPULATION TRENDS & PROJECTIONS

VILLAGE OF SISTER BAY, WISCONSIN

Year	Total	Percent Change
1920	190	---
1930	238	25.3%
1940	309	29.8%
1950	429	38.8%
1960	520	21.2%
1970	483	-7.1%
1980	564	16.8%
1990	675	19.7%
2000	886	31.3%
2005	**967**	**9.1%**
2010	**1,047**	**8.3%**
2015	**1,163**	**11.1%**
2020	**1,279**	**10.0%**
2025	**1,407**	**10.0%**

Notes

1. Historical Village population figures taken from Census Data.
2. Year 2005 population estimate based on data from Wisconsin Department of Administration.
3. Year 2010 - 2020 Village population projections based on forecasts from Village Comprehensive Plan.
4. Year 2025 Village population projection based on extrapolation of Comprehensive Plan forecasts.



P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay study\March-April 2008 Report Revisions\Chapters 1-14\Chapter 3\Figures and Tables\[Table3-1 & 3-2.xls]Table 3-1

TABLE 3-2

POPLUATION TRENDS
DOOR COUNTY COMMUNITIES

VILLAGE OF SISTER BAY, WISCONSIN

Community	2000	2004	Increase	Percent Change
Village of Sister Bay	***886***	***914***	***28***	***3.2%***
Door County				
City of Sturgeon Bay	9,437	9,696	259	2.7%
Village of Ephraim	353	356	3	0.8%
Village of Egg Harbor	250	261	11	4.4%
Town of Baileys Harbor	1,003	1,080	77	7.7%
Town of Gibraltar	1,063	1,156	93	8.7%
Town of Liberty Grove	1,858	1,958	100	5.4%
Town of Sevastopol	2,667	2,790	123	4.6%
Door County Total	**25,690**	**27,961**	**2,271**	**8.8%**
Wisconsin Total	5,363,715	5,532,000	168,285	3.1%

Source: U.S. Census Bureau & Wisconsin Department of Administration.

P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay study\Report Final Draft\Chapter 3\Figures and Tables\[Table3-2.xls]Table 3-2

TABLE 3-3

EXISTING LAND USE
SISTER BAY UTILITY SERVICE AREA
SISTER BAY, WISCONSIN

General Land Use Categories	*Existing Land Use (acres)*		
	Village of Sister Bay	Liberty Grove S.D. No. 1	Total
General Business	265.4	84.7	**350.1**
Downtown Business Transition	17.7	0.0	**17.7**
Downtown Business	25.9	0.0	**25.9**
Countryside	47.8	0.0	**47.8**
Institutional	81.9	0.0	**81.9**
Natural	0.0	12.0	**12.0**
Park/Recreational	51.7	0.0	**51.7**
Single Family/ Medium Density Residential	601.1	116.4	**717.5**
Multi-family/ High Density Residential	290.3	47.3	**337.6**
Large Lot/ Low Density Residential	202.3	14.3	**216.6**
Small Lot/ High Density Residential	5.8	0.0	**5.84**
ROW	62.0	0.0	**62.0**
TOTAL	**1,651.9**	**274.7**	**1,926.6**

Source: Acreages computed from Figure 2-1.

C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[table3-3.xls]Table 3-3

3.3 FUTURE COMMUNITY GROWTH

In general, projected growth patterns for Sister Bay and the surrounding area are consistent with recent development trends, and reflect the Village's long-term land use planning goals and objectives as stated in the *20-Year Comprehensive Plan Report*.

The expected increase in residential development is directly related to previous projections of population growth. Commercial land use is also expected to increase with increases in population. There is minimal industrial activity that exists in the area, and no significant changes are anticipated in the future for the planning area. The growth of the utility planning area will be a function of changes in population and commercial activity and employment opportunities in Sister Bay as well neighboring Door County communities.

The Comprehensive Utility Planning Area identified in Figure 2-1 encompasses approximately 4,000 acres. Over one-half of this acreage is currently located outside the Village and LGSD No. 1 boundaries. The vast majority of this outlying area (90 percent) is currently planned to be developed in the future as low to medium density residential or countryside land uses. A breakdown of the planned future land uses of the outlying planning area is summarized in Table 3-4.

3.4 FUTURE UTILITY SERVICE AREA

Figure 2-1 identified the boundaries of the year 2025 Sister Bay utility service planning area. The utility service area is defined as the area in which the Village of Sister Bay is anticipated to provide water and sewer service during the planning period. Future service area expansion is projected to occur primarily in the northern and southern areas of the Village.

The outer boundary of the future utility service area illustrated in Figure 2-1 was used for this study to identify the area which is expected to develop over the next 20 years and require Sister Bay water and sanitary sewer utility service, as well as be incorporated into a comprehensive storm water management area.

3.5 SUMMARY

This chapter summarizes the primary assumptions regarding future growth of the Village of Sister Bay utility service planning area. The present and future needs and characteristics of the identified service area will have a direct impact on the need for expansion of water, sanitary and storm sewer facilities. Therefore, the conclusions discussed in this chapter were used as a primary basis for projecting future water needs, evaluating the adequacy of existing utility system facilities, and identifying needs for future municipal utility system expansion.

TABLE 3-4

CURRENT PLANNED LAND USE
COMPREHENSIVE UTILITY PLANNING AREA
SISTER BAY, WISCONSIN

General Land Use Categories	*Existing and Proposed Planning Area Land Use (acres)*			
	Village of Sister Bay	Liberty Grove S.D. No. 1	Outlying Area	Total
General Business	265.4	84.7	32.5	**382.6**
Downtown Business Transition	17.7	0.0	0.1	**17.8**
Downtown Business	25.9	0.0	0.0	**25.9**
Countryside	47.8	0.0	412.0	**459.9**
Institutional	81.9	0.0	5.5	**87.4**
Industrial	0.0	0.0	27.7	**27.7**
Natural	0.0	12.0	26.3	**38.3**
Park/Recreational	51.7	0.0	1.0	**52.7**
Single Family/ Medium Density Residential	601.1	116.4	442.1	**1,159.6**
Multi-family/ High Density Residential	290.3	47.3	97.1	**434.6**
Large Lot/ Low Density Residential	202.3	14.3	1,000.4	**1,217.0**
Small Lot/ High Density Residential	5.8	0.0	0.0	**5.84**
ROW	62.0	0.0	0.0	**62.0**
TOTAL	**1,651.9**	**274.7**	**2,044.5**	**3,971.1**

C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[table3-3.xls]Table 3-4

CHAPTER 4

WATER REQUIREMENTS

Projections of customer demands serve as the basis for water and sewer system capital improvements planning. Several standard methods were used in this study to project water supply needs based on estimates of population and community growth. This chapter summarizes the methodology used and the results of these projections.

4.1 WATER CONSUMPTION HISTORY

An analysis was made of past water consumption characteristics by reviewing annual pumpage and water sales records for the period from 1988 to 2005. Average and maximum day water consumption during this period, together with the amount of water sold in each customer category, have been analyzed. Projections of future water requirements are based on the results of this analysis coupled with estimates of population and community growth discussed in Chapter 3.

A summary of historical customer water usage and Sister Bay Water Utility pumpage is provided in Table 4-1. Over the 18-year period of data summarized in the table, water usage varied from a low of 44 million gallons per year (MGY) in 1988 to a high of 74 MGY in 2005. Water usage over the 2001-2004 period was relatively stable, averaging 65 MGY with little variation. Water use increased 17 percent in 2005 over 2004 levels.

Average day water utility pumpage over the past 5 years has varied between 192,000 gallons per day (gpd) and 243,000 gpd, averaging 217,000 gpd. Sister Bay Water Utility sales and pumpage trends are graphically illustrated in Figure 4-1.

A recent historical summary of Utility customers served is provided in Table 4-2. Residential customers presently account for 79 percent to the Utility's customers, and 38 percent of total water sales. Commercial water use in 2005 accounted for approximately 45 percent of total sales. Sister Bay presently has no industrial water customers. Public water uses, which also include water sales to LGSD No. 1, account for approximately 17 percent of total demand.

4.1.1 LGSD No. 1 Water Usage

A summary of historical LGSD No. 1 customer water usage is provided in Table 4-3. Over the 9-year period of data summarized in the table, water usage varied from a low of 7 MGY in 2000 to a high of 9.9 MGY in 2005. Water usage by Sanitary District customers has averaged 8.03 MGY over the past five years. Average daily water consumption in 2005 was approximately 22,000 gpd.

A recent historical summary of Utility customers served is also provided in Table 4-3. Residential customers presently account for 81 percent to the District's customers, and 56 percent of the total demand. Commercial water use in 2005 accounted for approximately 44 percent of total sales. LGSD No. 1 does not have any industrial or public authority water customers.

TABLE 4-1

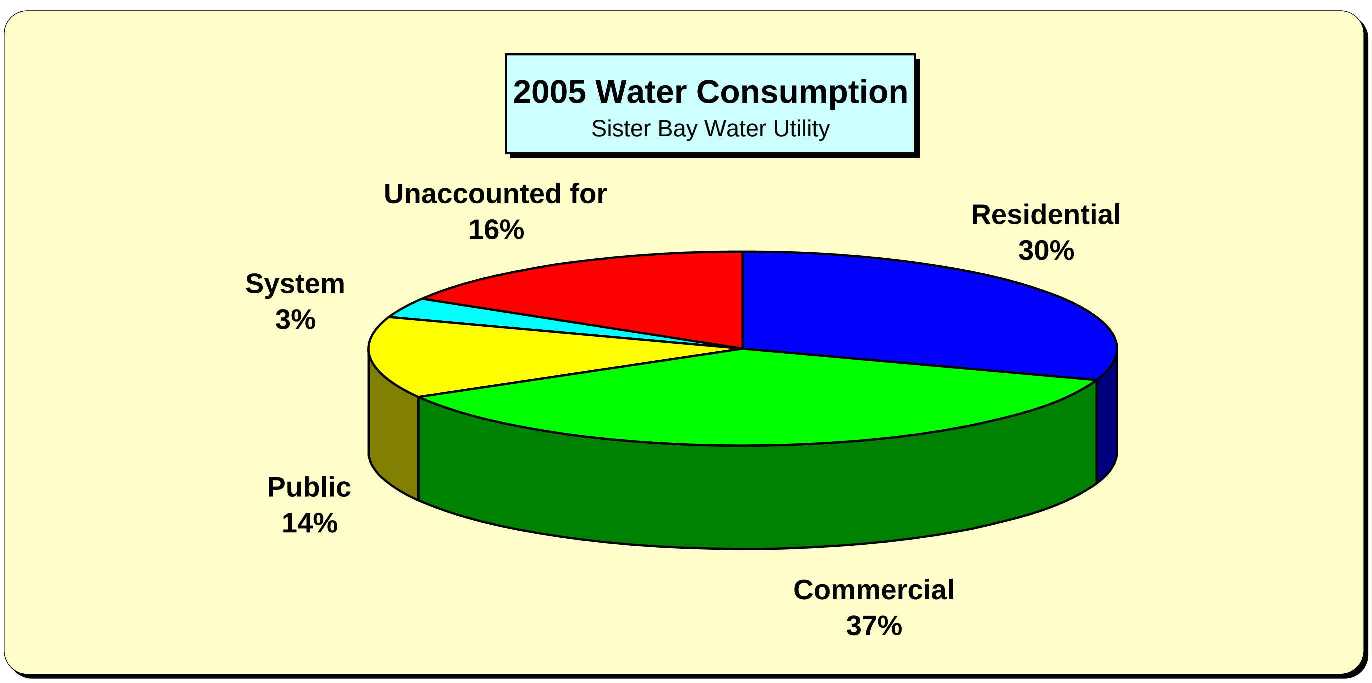
WATER CONSUMPTION HISTORY

SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

Year	Annual Water Sales (MGY)				System Uses	Total Usage (MGY)	Total Pumpage (MGY)	% Pumpage Metered
	Resi-dential	Com-mercial	Indust-rial	Public*				
1988	13.471	25.044	0.000	5.438	---	43.953	73.070	60.2%
1989	15.290	25.236	0.000	6.090	---	46.615	52.277	89.2%
1990	15.517	23.588	0.000	5.876	---	44.981	51.538	87.3%
1991	17.929	25.520	0.000	6.431	---	49.879	54.872	90.9%
1992	17.330	24.778	0.000	6.391	---	48.500	62.343	77.8%
1993	17.649	26.527	0.000	7.605	---	51.781	66.984	77.3%
1994	19.888	29.570	0.000	8.537	---	57.996	71.106	81.6%
1995	19.514	32.460	0.000	9.170	---	61.144	78.726	77.7%
1996	19.586	30.065	0.000	8.630	---	58.281	75.491	77.2%
1997	21.109	32.461	0.000	9.122	0.904	63.596	68.630	92.7%
1998	24.392	**36.721**	0.000	9.994	1.264	72.371	87.293	82.9%
1999	22.778	34.257	0.000	8.228	1.982	67.245	83.516	80.5%
2000	21.017	28.877	0.000	7.870	1.042	58.806	85.448	68.8%
2001	23.745	31.862	0.000	8.182	1.792	65.581	73.906	88.7%
2002	22.529	32.258	0.000	8.502	1.802	65.091	70.207	**92.7%**
2003	24.454	32.109	0.000	8.767	0.587	65.917	79.459	83.0%
2004	21.910	30.849	0.000	9.239	1.217	63.215	83.764	75.5%
2005	**26.856**	32.259	0.000	**12.163**	**2.761**	**74.039**	**88.653**	83.5%

Maximum Value =

* Public sales include water sales to LGSD No. 1



C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\Report\Chapter 1-5 8 11\[table4-x.xls]Tab4-1

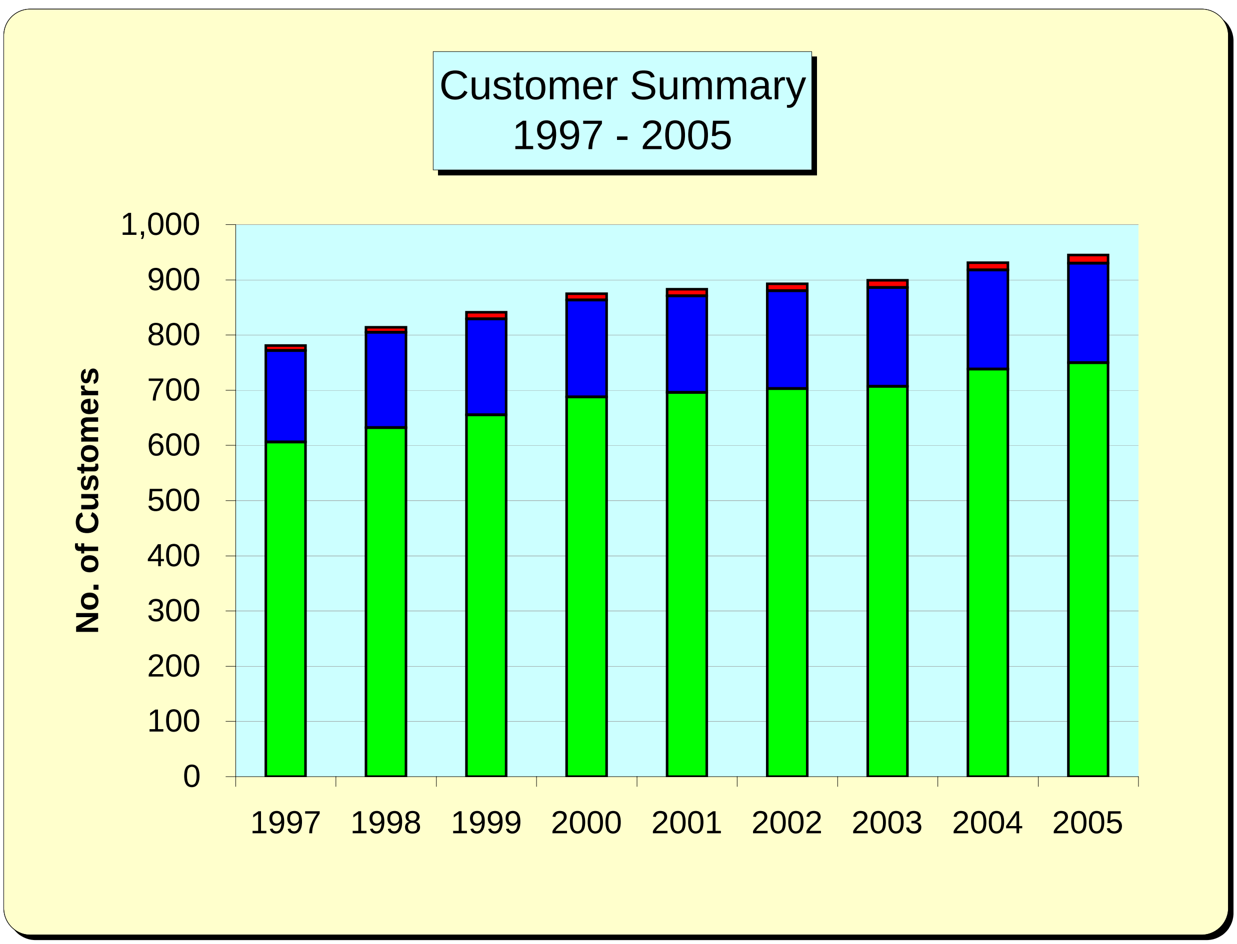
TABLE 4-2

HISTORICAL CUSTOMER SUMMARY

SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

Year	Number of Customers			
	Residential	Commercial	Public Authority	**Total**
1997	606	166	9	**781**
1998	632	173	9	**814**
1999	655	174	12	**841**
2000	688	175	12	**875**
2001	696	175	12	**883**
2002	703	177	13	**893**
2003	707	179	13	**899**
2004	738	**180**	13	**931**
2005	**750**	**180**	**15**	**945**

Maximum Value =



P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay study\March-April 2008 Report Revisions\Chapters 1-14\Chapter 4\Tables and Figures\[Chpt 4 (tables 1-8 & 10-11, fig 1-3).xl

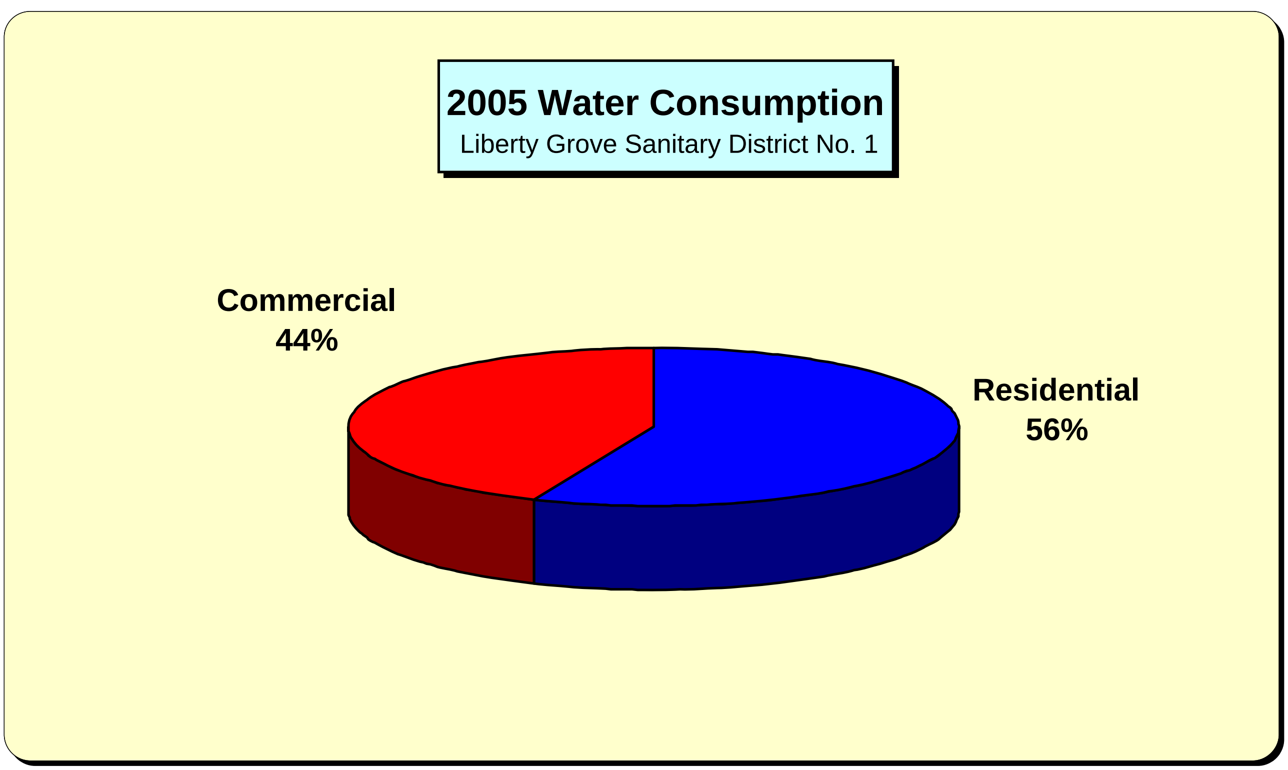
TABLE 4-3

WATER CONSUMPTION HISTORY

LIBERTY GROVE SANITARY DISTRICT NO. 1
TOWN OF LIBERTY GROVE, WISCONSIN

Year	Customers		Water Sales (MGY)			Total Usage (MGY)
	Resi-dential	Com-mercial	Resi-dential	Com-mercial	System Uses	
1997	88	23	3.377	4.246	0.066	7.689
1998	89	23	3.560	**4.968**	0.086	8.614
1999	91	23	3.444	3.807	0.064	7.315
2000	94	23	3.461	3.450	0.123	7.034
2001	96	25	3.987	3.323	0.056	7.366
2002	95	25	3.656	3.686	0.036	7.378
2003	100	25	4.111	3.425	0.049	7.585
2004	109	**26**	4.091	3.794	0.043	7.928
2005	**113**	**26**	**5.496**	4.250	**0.139**	**9.885**

Maximum Value =



C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\Report\Chapter 1-5 8 11\[table4-x.xls]Tab4-3

4.2 PER CAPITA WATER USAGE

Residential, commercial, and public water usage can be correlated to a community's population. An analysis of per capita water consumption for the Village of Sister Bay for each of these customer classifications was made from the available sales records and is summarized in Table 4-4. Figure 4-2 illustrates the results of this analysis. As indicated in this figure, per capita sales to residential, commercial, and public customers have followed certain trends over the previous 18 years.

The apparent trend in per capita residential water usage illustrated in Figure 4-2 is consistent with observed results for other Wisconsin municipal water utilities. Although per capita residential water usage in the U.S. had consistently increased until the early 1970s, water usage statistics indicate that the increasing rate of per capita consumption has leveled off. This may be due in part to residential customers becoming more aware of water costs, and water conservation measures becoming more common.

The Utility's residential per capita consumption has remained relatively constant over the previous 5 years, averaging 70 gallons per capita per day (gpcd). To project future water needs, average daily water usage for residential customers in the Sister Bay Water Utility planning area was projected to be 70 gpcd throughout the 20-year planning period.

Over the previous 5 years, per capita commercial sales have been relatively constant, varying between 89 and 97 gpcd. For this study, it was projected that future per capita commercial consumption will average approximately 93 gpcd. Since 2001, per capita public sales have averaged 4.0 gpcd. For this study, it was projected that future per capita public consumption will continue to average approximately 4 gpcd.

4.3 LARGE WATER USERS

Water consumption can vary widely on an annual basis depending on the types of large customers served, and the annual level of commercial activity. Fluctuations in water consumption for a particular large customer can be attributed to several factors including:

1. Changes in operating schedules or capacity
2. Changes in large water using processes
3. Changes in the number of persons employed
4. Seasonal variation in irrigation requirements
5. Seasonal changes in business activity
6. Implementation of conservation measures

Table 4-5 summarizes annual water sales to the major Utility water customers over the 2001-2005 period. A review of recent water sales records indicates that the top nine Sister Bay high volume water users consumed 52 percent of the total 2005 commercial water sales. Consequently, any significant changes in water consumption characteristics by these high volume users could have a very large impact on total Utility water requirements.

4.4 UNACCOUNTED-FOR WATER

There is generally a close relationship between the total gallons of water pumped, and the gallons of water metered and sold to water utility customers. Total metered water sales are always less than the amount of pumpage due to several factors, including:

TABLE 4-4

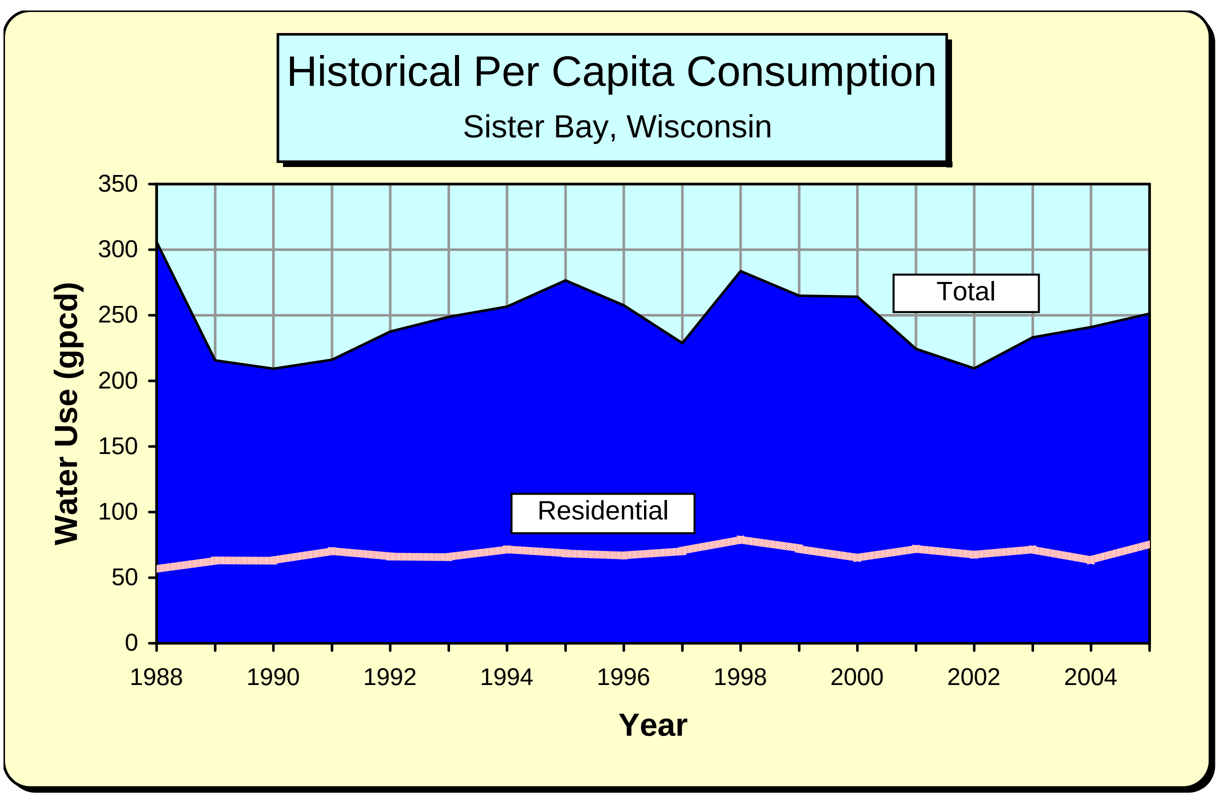
HISTORICAL PER CAPITA USAGE

SISTER BAY WATER UTILITY

VILLAGE OF SISTER BAY, WISCONSIN

Year	Estimated Population	GALLONS PER CAPITA PER DAY				
		Residential	Commercial	Public*	Total Metered	Total Pumpage
1988	653	56.4	104.8	22.8	183.9	305.7
1989	664	63.1	104.1	25.1	192.3	215.7
1990	675	63.0	95.7	23.9	182.6	209.2
1991	696	70.6	100.5	25.3	196.3	216.0
1992	717	66.0	94.4	24.4	184.8	237.6
1993	738	65.5	98.5	28.2	192.2	248.7
1994	759	71.8	106.7	30.8	209.3	256.7
1995	780	68.5	114.0	32.2	214.8	276.5
1996	801	66.8	102.6	29.5	198.9	257.5
1997	822	70.4	108.2	4.8	183.3	228.7
1998	843	79.3	119.3	4.5	203.1	283.7
1999	864	72.2	108.6	2.9	183.8	264.8
2000	886	65.0	89.3	2.6	156.9	264.2
2001	902	72.1	96.8	2.5	171.4	224.5
2002	918	67.2	96.3	3.4	166.9	209.5
2003	934	71.7	94.2	3.5	169.4	233.1
2004	950	63.0	88.7	3.8	155.5	240.9
2005	967	76.1	91.4	6.8	174.3	251.2

* Includes water sales to LGSD No. 1 (1988-1996)



C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\Report\Chapter 1-5 8 11\[table4-x.xls]Tab4-4

TABLE 4-5

SUMMARY OF LARGEST UTILITY CUSTOMERS

SISTER BAY WATER UTILITY

VILLAGE OF SISTER BAY, WISCONSIN

		LARGE CUSTOMER CONSUMPTION (MGY)				
2005 Rank	**LARGEST CUSTOMERS**	**2005**	**2004**	**2003**	**2002**	**2001**
1	Scandia Village	4.64	4.59	3.37	4.92	4.73
2	Pheasant Park Owners Association	2.84	2.24	2.40	2.46	1.86
3	Scandanavian Lodge	1.70	1.85	1.82	1.97	1.73
4	Birchwood Lodge	1.47	1.49	2.13	2.19	0.37
5	Al Johnson's Restaurant	1.41	1.29	1.31	1.26	1.56
6	DuNord Properties (LGSD)	1.05	1.07	1.20	1.22	1.29
7	Church-Hill Inn	1.02	1.04	1.06	1.03	1.07
8	Helms Four Seasons Resort	0.87	1.02	1.00	0.97	0.99
9	Final Rinse Laundromat	0.87	0.83	0.87	0.97	0.96
10	Sister Bay Bowl	0.80	0.95	0.70	1.18	0.65
	Total	**16.67**	**16.36**	**15.87**	**18.16**	**15.21**

C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[table4-x.xls]Tab4-5

1. Unmetered water usage for maintenance purposes such as hydrant flushing and water main repairs
2. Unmetered water usage for fire fighting
3. Inaccuracies in water metering devices
4. Unaccounted-for public water usage
5. Leakage within the distribution system

The difference between total pumpage and total water sales is termed "unaccounted-for" water. The amount of unaccounted-for water is an indication of the condition of the water system and is usually expressed as a percentage. When a distribution system is very old or poorly maintained, the percentage of unaccounted-for water often increases dramatically.

Table 4-1 provided a historical summary of the percentage of total pumpage metered over the past 18 years. The percentage of total Sister Bay pumpage metered has been reported to be as low as 60 percent and as high as 93 percent since 1988. This high degree of fluctuation is often common for small public water utilities, and can be influenced by the factors summarized above. For example, the percentage of total pumpage metered would be expected to decrease in years when unusual problems with leakage or meter stoppage occurred, or when unusually high water demands for fire protection occurred. As a general rule, for small water systems the percentage of total pumpage metered should be maintained above 85 percent, which would correspond to unaccounted-for water amounting to less than 15 percent.

Over the previous 10 years, the Utility has averaged approximately 17 percent unaccounted-for water. For this study, it was assumed that the percentage of total pumpage metered in future years will be maintained at a minimum value of 15 percent.

4.5 VARIATIONS IN CUSTOMER DEMANDS AND PUMPAGE

Seasonal fluctuations in water usage are important factors in the design and sizing of water supply and storage facilities. The seasonal nature of water consumption in the Village of Sister Bay can be demonstrated by an analysis of monthly pumpage variations. The Utility's monthly pumpage variations in 2005 are presented in Table 4-6. In 2005, the maximum monthly pumpage occurred in July, while the minimum monthly pumpage occurred in March.

Maximum daily water demands usually occur during the summer months on hot days when additional water is used for watering lawns, gardening, bathing, and other recreation. The maximum day demand is defined as the amount of water pumped during a single day of the year with the highest water usage, and is often expressed as a ratio of the annual average day pumpage. The maximum day pumpage is of particular importance to water system planning, because water supply facilities are sized to meet this demand.

Table 4-7 presents the average and maximum day pumpage for each year from 1988 to 2005. The maximum day pumpage usually occurs during June, July, or August. The minimum day pumpage typically occurs during winter or early spring. Over the last 18 years, the maximum day pumpage ratio (ratio of maximum to average day pumpage) has varied from a low of approximately 1.98 in 1993 to a high of 3.21 in 2005.

To gain a better understanding of expected fluctuations in customer demands for the Village of Sister Bay; a statistical analysis was performed of historical maximum day pumpage ratios. Table 4-8

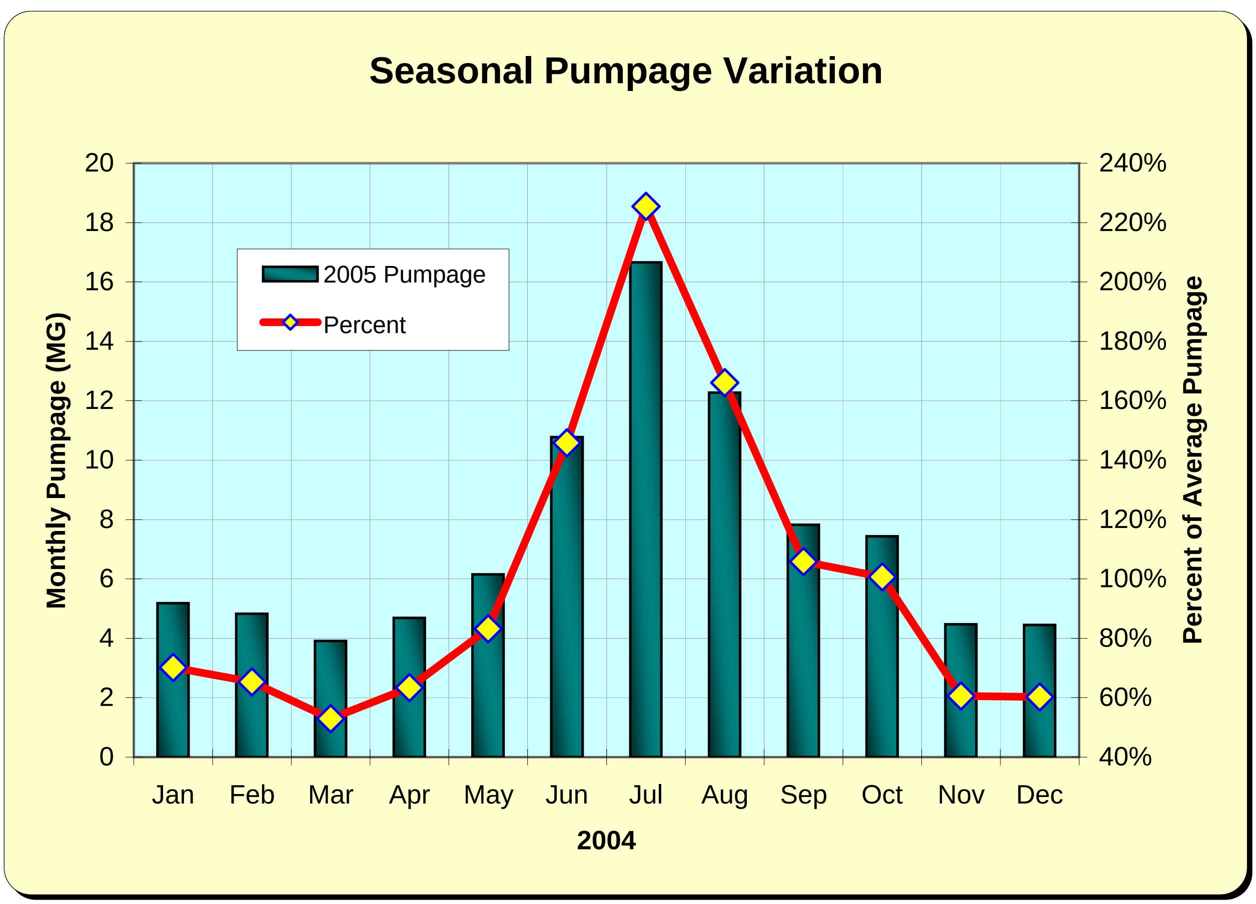
TABLE 4-6

SEASONAL PUMPAGE VARIATIONS

SISTER BAY WATER UTILITY

VILLAGE OF SISTER BAY, WISCONSIN

Month	2005 Monthly Pumpage (MG)	Percentage of Total Pumpage	Percentage of Average Pumpage
January	5.185	5.8%	70.2%
February	4.826	5.4%	65.3%
March	3.914	4.4%	53.0%
April	4.686	5.3%	63.4%
May	6.149	6.9%	83.2%
June	10.775	12.2%	145.8%
July	16.659	18.8%	225.5%
August	12.270	13.8%	166.1%
September	7.823	8.8%	105.9%
October	7.440	8.4%	100.7%
November	4.475	5.0%	60.6%
December	4.451	5.0%	60.2%
Total	**88.653**	**100.0%**	



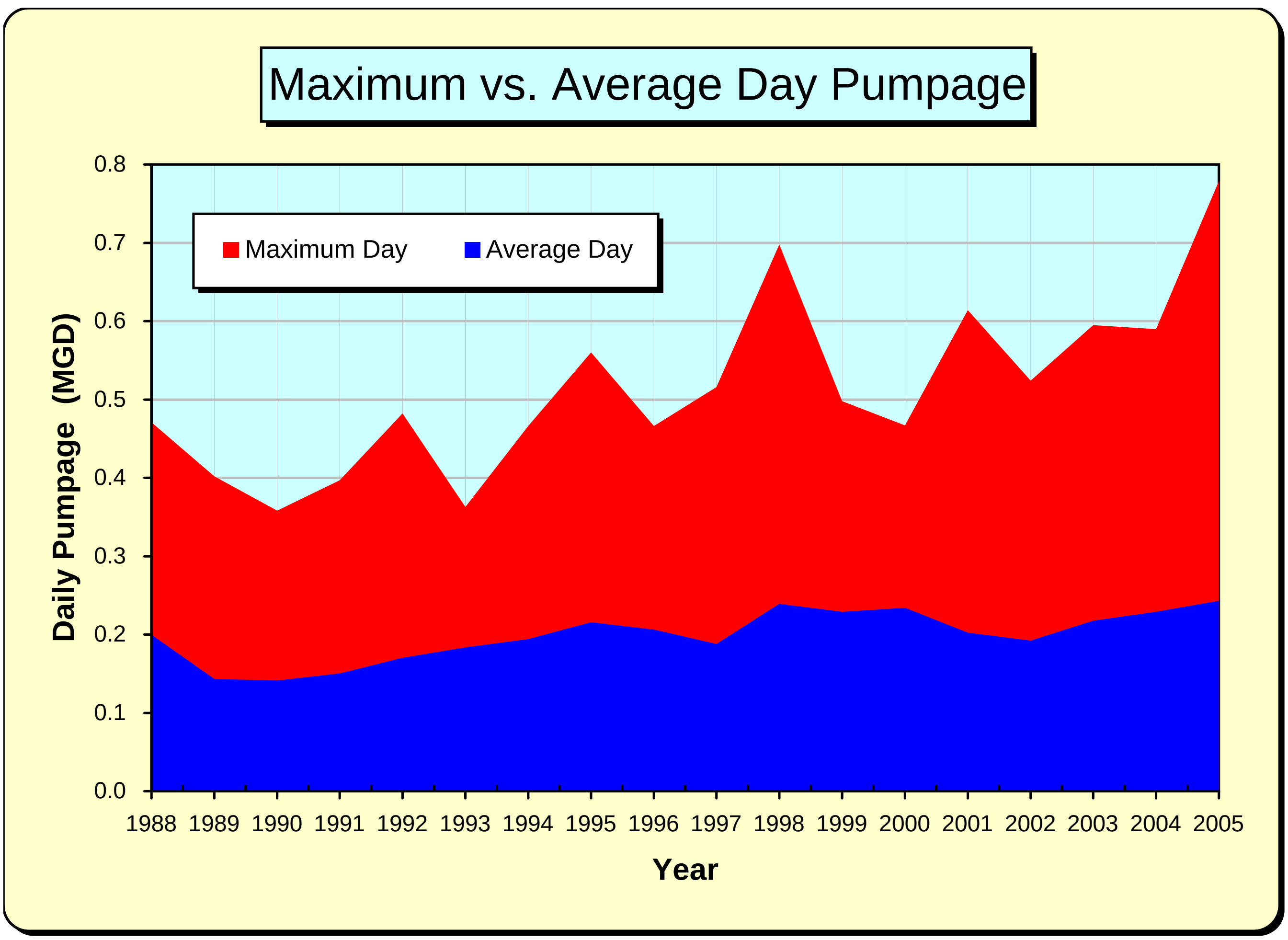
C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\Report\Chapter 1-5 8 11\[table4-x.xls]Tab4-6

TABLE 4-7

DAILY PUMPAGE VARIATIONS
SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

Year	Avg. Day Pumpage (MGD)	Max. Day Pumpage (MGD)	Ratio of Max. to Avg. Day	Year	Avg. Day Pumpage (MGD)	Max. Day Pumpage (MGD)	Ratio of Max. to Avg. Day
1988	0.200	0.471	2.36	1997	0.188	0.516	2.74
1989	0.143	0.402	2.81	1998	0.239	0.698	2.92
1990	0.141	0.358	2.54	1999	0.229	0.498	2.18
1991	0.150	0.397	2.64	2000	0.234	0.467	1.99
1992	0.170	0.482	2.83	2001	0.202	0.614	3.03
1993	0.184	0.363	1.98	2002	0.192	0.524	2.72
1994	0.194	0.466	2.40	2003	0.218	0.595	2.73
1995	0.216	0.560	2.60	2004	0.229	0.590	2.58
1996	0.206	0.466	2.26	2005	**0.243**	**0.779**	**3.21**

Maximum Value =



C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\Report\Chapter 1-5 8 11\[table4-x.xls]Tab4-7

TABLE 4-8

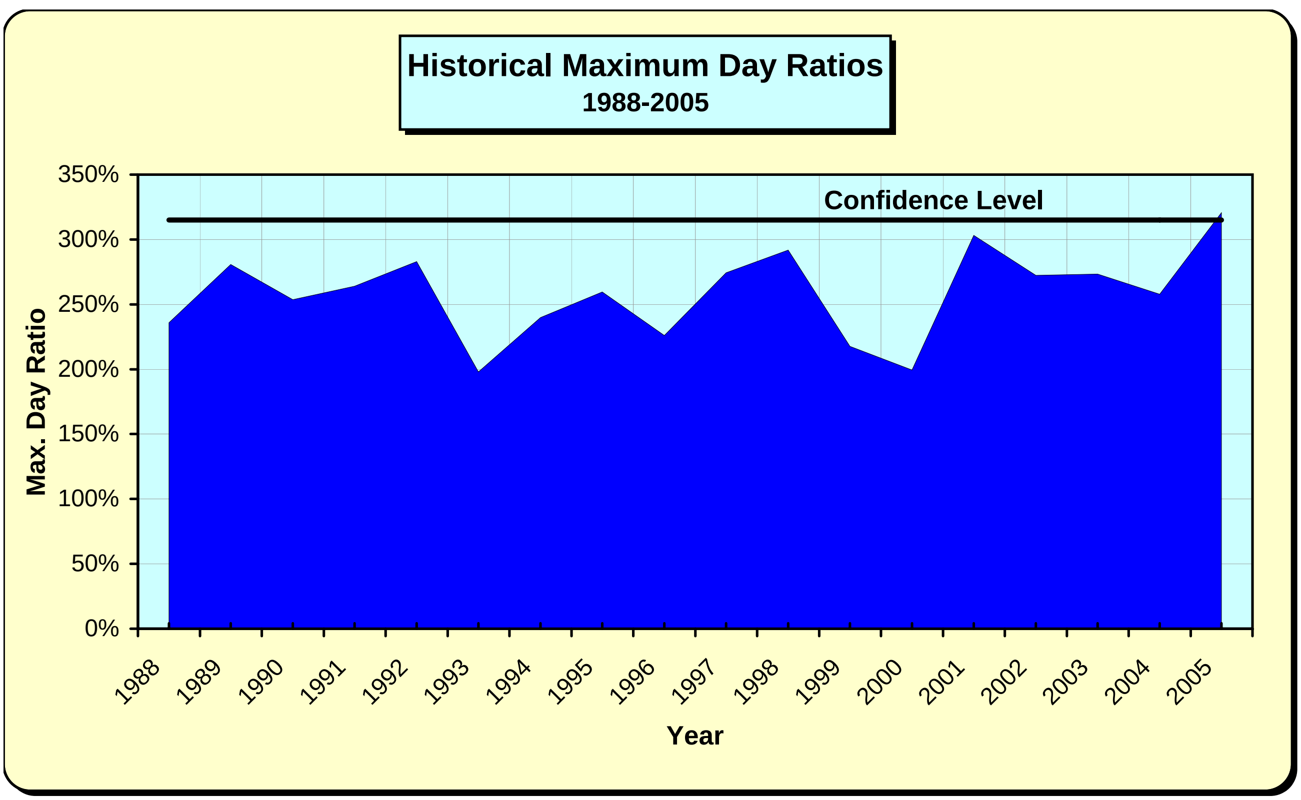
STATISTICAL ANALYSIS:
RATIO OF AVERAGE TO MAXIMUM DAY DEMAND

SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

	2001 to 2005	1988 to 2005
Number of years of Data	5	18
Maximum Ratio - Max. to Avg. Day Pumpage	3.21	3.21
Minimum Ratio - Max. to Avg. Day Pumpage	2.58	1.98
Average Ratio Max. to Avg. Day Pumpage	2.86	2.58
Standard Deviation	23%	33%

Confidence Level (%)	Ratio of Max. to Avg. Day Pumpage	Ratio of Max. to Avg. Day Pumpage
80%	3.05	2.86
85%	3.09	2.93
90%	3.15	3.01
95%	3.23	3.12
98%	3.33	3.26
99%	3.39	3.35

Note The "Confidence Level" represents the probability (%) that in any given year, the actual ratio of maximum to average day pumpage will be less than or equal to the ratio indicated in the table. The ratios in the table were determined based on a statistical analysis of historical ratios over each period of analysis, assuming a normal distribution.



C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\Report\Chapter 1-5 8 11\[table4-x.xls]Tab4-8

summarizes the results of this analysis. Two periods of analysis were examined; the entire period of 1988 to 2005, and the latest 5-year period from 2001 to 2005.

For the years 1988 to 2005, the average maximum day demand ratio was 2.58, with a standard deviation of 33 percent. In comparison, over the period of 2001 to 2005, the average ratio was 2.86, with a standard deviation of 23 percent. For this study, it was projected that future demand variations will resemble the variations observed over the most recent 5-year period.

Table 4-8 also includes a statistical analysis of expected maximum day pumpage ratios for various normal distribution confidence levels. For example, based on the analysis of the data from 2001 to 2005, there is an 80 percent chance in any given year that the actual maximum day pumpage ratio will be less than or equal to 3.05. Conversely, there is a 20 percent chance the actual ratio will exceed 3.05.

To evaluate future water supply and storage needs, a maximum day pumpage ratio of 3.15 was used for this study. This ratio provides a confidence level of 90 percent based on maximum day pumpage ratios over the past 5 years.

4.6 HOURLY DEMAND FLUCTUATIONS

The hour-to-hour variation of customer demands is also an important characteristic used to evaluate water supply and storage requirements. As with maximum day demands, peak hour demand is often expressed as a ratio of average day demand for the year. The peak hour demand is simply the hour of maximum demand that occurs on the maximum day.

The peak hourly rate for Sister Bay was estimated to be approximately 200 percent of the maximum day rate. This estimate is based on hourly demand fluctuations measured in the Sister Bay water system during field testing. The estimated diurnal water demand curve for the Sister Bay water system as determined for the September 27, 2005, field testing date is summarized in Table 4-9. As indicated in the graph in Table 4-9, Sister Bay water consumption typically peaks at three different times during the day: mid-morning, noon hour and early evening.

This diurnal demand curve is common for small, largely residential communities with little or no large industrial water consumption. This analysis would indicate a peak hour demand to average day pumpage ratio of approximately 6.3.

4.7 WATER CONSUMPTION AND PUMPAGE PROJECTIONS

Future sales and pumpage projections are based on assumptions of water demand, coupled with estimates of future population and community growth. A detailed summary of the individual components of projected water sales and pumpage requirements is provided in Table 4-10. Figure 4-3 illustrates the historical annual water sales along with the future projections.

4.7.1 Residential Sales

Residential sales were projected based on current trends and assumptions regarding future population served and per capita water consumption. By the year 2025, it is estimated that the residential consumption rate will be approximately 70 gpcd, resulting in total residential sales of approximately 36 MGY. The projected 2025 residential consumption will be about 36 percent of total annual sales.

TABLE 4-9

ESTIMATED TIME-OF-DAY DEMAND CURVE

Sister Bay Water Utility
Village of Sister Bay, Wisconsin

Time Period	September 27 2005 Time of Day	System Demand Estimate (MGD)	Percent of Average Demand
1	Midnight - 1 am	0.108	37%
2	1 am - 2 am	0.133	46%
3	2 am - 3 am	0.153	53%
4	3 am - 4 am	0.119	41%
5	4 am - 5 am	0.080	28%
6	5 am - 6 am	0.134	46%
7	6 am - 7 am	0.351	122%
8	7 am - 8 am	0.479	166%
9	8 am - 9 am	0.480	167%
10	9 am - 10 am	0.439	152%
11	10 am - 11 am	0.310	108%
12	11 am - Noon	0.334	116%
13	Noon - 1 pm	0.494	172%
14	1 pm - 2 pm	0.548	190%
15	2 pm - 3 pm	0.455	158%
16	3 pm - 4 pm	0.332	115%
17	4 pm - 5 pm	0.304	105%
18	5 pm - 6 pm	0.326	113%
19	6 pm - 7 pm	0.398	138%
20	7 pm - 8 pm	0.416	144%
21	8 pm - 9 pm	0.288	100%
22	9 pm - 10 pm	0.174	61%
23	10 pm - 11 pm	0.086	30%
24	11 pm - Midnight	0.074	26%
TOTAL DEMAND		**0.288**	

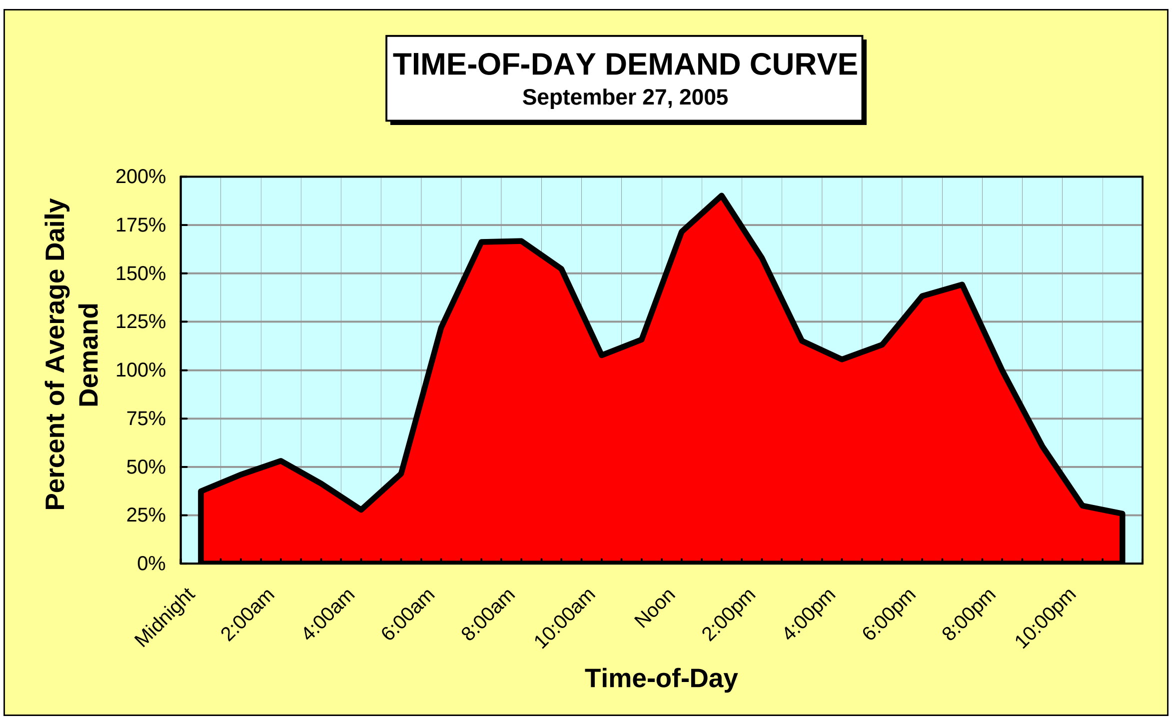


TABLE 4-10

WATER SALES & PUMPAGE PROJECTIONS

SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

Customer Classification	Actual 2005	Projected 2015	Projected 2025
Population Served	967	1,163	1,407
Residential Sales			
Per Capita Sales (gpcd)	76.1	70	70
Annual Sales (MGY)	26.86	29.70	35.90
Public Sales			
Per Capita Sales (gpcd)	6.8	4	4
Annual Sales (MGY)	2.42	1.70	2.10
Commercial Sales			
Per Capita Sales (gpcd)	91.4	93	93
Annual Sales (MGY)	32.26	39.00	48.00
LGSD No. 1 Sales			
Annual Sales (MGY)	9.89	9.50	11.40
System Uses			
Annual Usage (MGY)	2.76	3.30	4.00
TOTAL METERED SALES (MGY)	74.18	**83.20**	**101.40**
Unaccounted-For Water (MGY)	14.48	14.70	17.90
TOTAL PUMPAGE (MGY)	*88.65*	***97.90***	***119.30***

Notes

1. Projections assume no significant changes in consumption patterns of largest Utility customers.
2. Projections assume similar proportional increases in water usage by LGSD No. 1 into the future.
3. Unaccounted-for water was projected at 15% of total pumpage for future years.

C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\Report\Chapter 1-5 8 11\[table4-x.xls]Tab4-10

4.7.2 Public Sales

Future per capita sales to public customers were projected to be approximately 4 gpcd throughout the planning period. By the year 2025, it is estimated that public sales will be approximately 2.1 MGY, or about 2 percent of total annual sales.

4.7.3 Commercial Sales

Future per capita consumption by commercial customers was projected to be approximately 93 gpcd over the planning period. Total annual sales to commercial customers are projected to reach 48 MGY by 2025, or approximately 48 percent of total annual sales.

4.7.4 LGSD No. 1 Sales

Water use by LGSD No. 1 was projected to increase in proportion to the increase in water sales to existing residential and commercial customers of the Sister Bay Water Utility. By the year 2025, it is estimated that water consumption by LGSD No. 1 will be approximately 13.7 MGY, or about 14 percent of total annual sales.

4.8 SUMMARY OF TOTAL DEMANDS AND PUMPAGE REQUIREMENTS

The total annual metered sales projections previously summarized in Table 4-10 were based on a summation of sales projections for each major customer classification. An allowance was also made for unmetered miscellaneous water usage and losses (unaccounted-for water) to arrive at total pumpage projections.

Table 4-11 summarizes projections of future water needs. Future annual sales are projected to increase from 74 MGY to 104 MGY in 2025. Total annual pumpage should increase to approximately 122 MGY by the year 2025.

Estimates of daily demand fluctuations have also been made based on projections of future annual sales. By the year 2025, average day pumpage is projected to increase to 0.334 mgd, and maximum day pumpage is projected to increase to 1.05 mgd. Future projections of maximum day pumpage are based on a ratio of maximum day to average day of 315 percent.

Peak hour demand was projected in a similar fashion. Peak hour demand was projected by assuming a ratio of peak hour demand to maximum day pumpage of 200 percent. Peak hour demand is projected to increase to a rate of approximately 1,460 gpm by the year 2025.

4.9 WATER NEEDS FOR FIRE PROTECTION

In addition to the water supply requirements for residential, public, commercial, and LGSD customers, water system planning for fire protection needs is an important consideration. In most instances, water main sizes are designed specifically to supply needed fire flow requirements.

Guidelines for determining fire flow requirements are developed based on recommendations offered by the Insurance Services Office (ISO), which is responsible for evaluating and classifying municipalities for fire insurance rating purposes. When a community evaluation is conducted by ISO, the water system is evaluated for its capacity to provide needed fire flow at a specific location and will depend on land use

TABLE 4-11

FUTURE PUMPAGE PROJECTIONS

SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

	Actual 2005	Projected 2015	Projected 2025
Total Annual Sales (MGY)	74.2	85.1	103.7
Total Annual Pumpage (MGY)	88.7	100.0	122.0
Average Day Pumpage (mgd)	0.243	0.274	0.334
Design Maximum Day Pumpage (mgd)	0.765	0.860	1.050
Design Peak Hour Demand (gpm)	1,060	1,190	1,460

Notes

1. Year 2005, 2015 and 2025 design maximum day pumpage projections were estimated using a ratio of maximum to average day pumpage of 315 percent.
2. Year 2005, 2015 and 2025 design peak hour demand projections were estimated using a ratio of peak hour demand to maximum day pumpage of 200 percent.

C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\Report\Chapter 1-5 8 11\[table4-x.xls]Tab4-11

characteristics and the types of properties to be protected. In high value districts, fire flow requirements of up to 3,500 gpm can be expected. However, based on consultations with the Sister Bay/Liberty Grove Fire Department, it was agreed that the maximum fire flow requirement of 2,000 gpm for three hours would be used for establishing water supply and storage requirements.

Therefore, for the purposes this study, the basic fire flow requirement for all high density residential and commercial developments was assumed to be 2,000 gpm for 3 hours, and 1,000 gpm for 1 hour for all medium and low density residential developments. Based on current development trends currently expected within the planning area, these basic fire flow requirements are not expected to change over the planning period.

CHAPTER 5

EXISTING WATER SYSTEM FACILITIES

The water system facilities operated and maintained by the Sister Bay Water Utility include:

1. Three groundwater wells and pump stations
2. Two elevated water storage tanks
3. Seven pressure reducing stations
4. Water system controls located in the Wastewater Treatment Plant Administration Building
5. A network of transmission and distribution water mains

The general location and layout of the water system facilities is illustrated in Figure 5-1. A schematic of the water system is illustrated in Figure 5-2. This chapter presents a summary of the design and operating characteristics of the existing water system components.

5.1 EXISTING WELLS

The Sister Bay Water Utility operates three groundwater wells located throughout the Sister Bay area. All of the wells are completed in the deep dolomite aquifer. The rock well yields are reported to range from approximately 450 gpm to 500 gpm. The constructed depths of the deep wells range from 208 to 305 feet. Current specific capacities range from approximately 10 to 16 gpm per foot of drawdown. Table 5-1 summarizes the system supply well data. Table 5-2 presents a summary of the pump and motor data for the Village's supply wells.

5.1.1 Well 1

Well 1 is located on Scandia Road immediately east of STH 42. The well was constructed in 1972 to a total depth of 208 feet. The well contains a 10-inch diameter casing to a depth of 138 feet. The well is grouted to a depth of 138 feet. Well 1's original static water level was reported to be at ground level. Fall 2005 operating conditions included a static water level of 6 feet, with a specific capacity of 10.1 gpm per foot of drawdown.

Well 1 is equipped with a Peerless 6-stage vertical turbine, line shaft pump powered by a 40 horsepower Westinghouse electric motor. The pump is rated for 400 gpm at 250 feet TDH, and is set at 120 feet. Well 1 is pumped directly into the Main Pressure Zone distribution system. The station is served by a standby diesel generator that can supply power to the well pump motor in the event of an emergency.

Water pumped from Well 1 is disinfected using gas chlorine. The pump discharge piping includes a check valve to prevent backflow, flow meter for quantifying pumpage and a pressure gauge for monitoring station discharge pressure. The station is in good structural condition, and the building, pumping and electrical equipment have been well maintained and are in good condition.

5.1.2 Well 2

Well 2 is located along Smith Drive east of STH 57. The well was constructed in 1972 to a total depth of 305 feet. The well contains a 10-inch diameter casing to a depth of 171 feet. The well is grouted to a depth of 171 feet. Well 2's original static water level was reported to 92 feet below the ground surface.

TABLE 5-1

EXISTING WELL DATA

SISTER BAY WATER UTILITY

VILLAGE OF SISTER BAY, WISCONSIN

	SUPPLY WELLS		
Well Data	**Well 1**	**Well 2**	**Well 3**
Year Constructed	1972	1972	2000
Depth (feet)	208	305	262
Well Driller	Miller Well & Pump	Miller Well & Pump	Layne Christensen
Casing: Diameter (in.)	10	10	12
Depth (ft.)	138	171	171
Formation	Silurian Dolomite	Silurian Dolomite	Silurian Dolomite
Grouted Depth (ft.)	138	171	171
Original Construction:			
Static Water Level (ft.)	0	92	19
Pumping Water Level(ft.)	83	157	56
Drawdown (ft.)	83	65	37
Pumping Rate (gpm)	400	450	500
Specific Capacity (gpm/ft)	4.8	6.9	13.5
September 2005 Conditions:			
Static Water Level (ft.)	6	98	27
Pumping Water Level(ft.)	54	128	55
Drawdown (ft.)	48	30	28
Pumping Rate (gpm)	485	480	455
Specific Capacity (gpm/ft)	10.1	16.0	16.3

C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[table5-x.xls]Table 5-1

TABLE 5-2

EXISTING WELL PUMP DATA

SISTER BAY WATER UTILITY

VILLAGE OF SISTER BAY, WISCONSIN

	Supply Wells		
Pump Data	**Well 1**	**Well 2**	**Well 3**
Type	Vertical Turbine	Vertical Turbine	Vertical Turbine
Manufacturer	Peerless	Peerless	Goulds
Year Installed	1972	1972	2001
Pump Setting (feet)	120	160	180
No. of Stages	6	5	8
Rated Conditions:			
Flow Rate (gpm)	400	400	450
TDH (feet)	250	215	300
Motor Data			
Manufacturer	Westinghouse	U.S. Motors	U.S. Motors
Horsepower	40	30	50
RPM	1800	1800	1800
Voltage	230/460	230/460	230/460
Phase / Cycles	3 / 60	3 / 60	3 / 60
Standby Power:	Yes	Yes	Yes
Type	Generator	Generator	Generator
Fuel	Diesel	Diesel	Diesel
Pump Discharges to:	*Distribution System*	*Distribution System*	*Distribution System*
Pressure Zone:	*Main*	*Main*	*High Level*

C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[table5-x.xls]Table 5-2

Fall 2005 operating conditions included a static water level of 98 feet, with a specific capacity of 16 gpm per foot of drawdown.

Well 2 is equipped with a Peerless 5-stage vertical turbine, line shaft pump powered by a 30 horsepower U.S. Motors electric motor. The pump is rated for 400 gpm at 215 feet TDH, and is set at 160 feet. Well 2 is pumped directly into the Main Pressure Zone distribution system. The station is served by a standby diesel generator that can supply power to the well pump motor in the event of an emergency.

Water pumped from Well 2 is disinfected using gas chlorine. The pump discharge piping includes a check valve to prevent backflow, flow meter for quantifying pumpage and a pressure gauge for monitoring station discharge pressure. The station is in good structural condition, and the building, pumping and electrical equipment have been well maintained and are in good condition.

5.1.3 Well 3

Well 3 is located at the intersection of Hill Road and North Spring Street. The well was constructed in 2001 to a total depth of 262 feet. The well contains a 12-inch diameter casing to a depth of 171 feet. The well is grouted to a depth of 171 feet. Well 3's original static water level was reported to 19 feet below the ground surface. Fall 2005 operating conditions included a static water level of 27 feet, with a specific capacity of 16.3 gpm per foot of drawdown.

Well 3 is equipped with a Gould 8-stage vertical turbine, line shaft pump powered by a 50 horsepower U.S. Motors electric motor. The pump is rated for 450 gpm at 300 feet TDH, and is set at 180 feet. Well 3 is pumped directly into the High Level Pressure Zone distribution system. The station is served by a standby diesel generator that can supply power to the well pump motor in the event of an emergency.

Water pumped from Well 3 is disinfected using sodium hypochlorite. The pump discharge piping includes a check valve to prevent backflow, flow meter for quantifying pumpage and a pressure gauge for monitoring station discharge pressure. The station is in good structural condition, and the building, pumping and electrical equipment have been well maintained and are in good condition.

5.1.4 Historical Well Performance

The historical performance of each water supply well was analyzed. Available well and pump operating and performance data was collected and reviewed. The performance indicators include static and pumping water levels, pumping rate, and well specific capacity. The performance of each well with respect to each of the performance indicators is graphically summarized in Appendix A.

Seasonal declines in static water levels are apparent in each well due to high pumpage rates in summer. However, no long-term static water level decline trend is noticeable in the graphs of each well. Additionally, no significant well or pump operating concerns were observed during the inspection of the pumping facilities or during a review of the historical well performance information in Appendix A.

5.2 EXISTING BOOSTER PUMP FACILITIES

The Sister Bay Water Utility operates two booster pumping stations that supply water to the High Level Pressure Zone. Table 5-3 presents a summary of the pump and motor data for the Village's booster stations. The following sections briefly summarize the design and operating characteristics of each station.

TABLE 5-3

EXISTING BOOSTER PUMP DATA
SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

Pump Data	BOOSTER STATION			
	Sister Bay		*Liberty Grove*	
	Pump 1	Pump 2	Pump 1	Pump 2
Pump Type	Vertical Centrifugal	Vertical Centrifugal	Vertical Centrifugal	Vertical Centrifugal
Manufacturer	Aurora	Aurora	Weinman	Weinman
Rated Conditions:				
Flow Rate (gpm)	500	500	100	100
TDH (ft.)	101.8	101.8	96	96
Pump Discharges to:	*Distribution System*	*Distribution System*	*Distribution System*	*Distribution System*
Pressure Zone:	*High Level*	*High Level*	*High Level*	*High Level*
Pump Motor Data				
Manufacturer	Marathon	Marathon		
Horsepower	20	20	5	5
Phase / Cycles	3 / 60	3 / 60	3 / 60	3 / 60
RPM	1750 - Variable	1750 - Variable	1750 - Variable	1750 - Variable
Standby Power:	Yes	Yes	Yes	Yes
Type	Generator	Generator	Generator	Generator
Fuel	Diesel	Diesel	Diesel	Diesel

C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[table5-x.xls]Table 5-3

5.2.1 Sister Bay Booster Station

The Sister Bay Booster Station is located inside the Well 2 pump station facility. The station is equipped with two identical Aurora vertical centrifugal pumps, powered by a 20 horsepower Marathon electric motor. The pumps are rated for 500 gpm at 102 feet TDH. Both booster pumps are equipped with variable frequency drives. The standby diesel generator that supplies emergency power to the Well 2 pump motor can also supply power to booster pump motors in the event of an emergency. Water levels in the Jungwirth Tower control the operation of the booster pumps.

5.2.2 Liberty Grove Booster Station

The Liberty Grove Booster Station is located inside the Well 3 pump station facility. The station is equipped with two identical Weinmann vertical centrifugal pumps, powered by 5 horsepower electric motors. The pumps are rated for 100 gpm at 96 feet TDH. Both booster pumps are equipped with variable frequency drives. The standby diesel generator that supplies emergency power to the Well 3 pump motor can also supply power to booster pump motors in the event of an emergency. System pressures in the High Level Zone control the operation of the booster pumps.

5.3 EXISTING STORAGE FACILITIES

The Sister Bay Water Utility operates two elevated storage facilities that provide pressure equalization for each pressure zone, provide stored water for fire protection and other emergencies, and provides a means for controlling the well and booster pumps. Table 5-4 presents a summary of the pump and motor data for the Village's booster stations. The following sections briefly summarize the design and operating characteristics of each storage tank.

5.3.1 Highway 57 Standpipe

The standpipe was constructed in 1972 immediately adjacent to Well 2 by the Brown Tank Company. The tank has a water storage volume of 100,000 gallons. The standpipe is 19 feet in diameter and has an overflow elevation of 730 feet USGS (48 feet above ground). The water level in the standpipe is maintained to provide system pressures in the Main Pressure Zone.

5.3.2 Jungwirth Tower

The Jungwirth Tower was constructed in 1996 on Jungwirth Court by the Caldwell Tank Company. The tank has a water storage volume of 150,000 gallons. The tower is approximately 110 feet high, and has an overflow elevation of 826 feet USGS. The water level in the tower is maintained to provide system pressures in the High Level Pressure Zone.

5.4 EXISTING PRESSURE REDUCING STATIONS

The Sister Bay Water Utility operates seven pressure reducing stations that provide additional water to the low level Pressure Zone in the event of a low pressure or fire fighting emergency. Three of the stations are located along the pressure boundary in northern and central Sister Bay. Two stations are located in the western area of the Village and serve the low lying areas to the east of the Jungwirth Tower.

TABLE 5-4

EXISTING ELEVATED STORAGE TANK DATA
SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

	Standpipe	Jungwirth Tower
Village Location	NW of STH 57 and Smith Drive	Jungwirth Courth west of N. Highland Road
Pressure Zone Served	Main Zone	High Level Zone
Year Constructed	1972	1996
Constructed By	Brown Tank Co.	Caldwell Tank
Type	Standpipe	Single Pedestal Sphere
Storage Reservoir Material	Steel	Steel
Maximum Storage Volume (gal)	100,000	150,000
Height to Overflow (feet)	48	109.5
Overflow Elevation (feet USGS)	730	826
Base Elevation (feet USGS)	682	716.5
Diameter (feet)	19	40
Head Range (feet)	48	30

C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[table5-x.xls]Table 5-4

The exact pressure settings of the stations is uncertain, but they were designed to open upon a low pressure reading on the downstream side of the valve (Main Pressure Zone side), while also maintaining a required minimum upstream pressure in the High Level Pressure Zone. During the inspections performed for this study, it was observed that the older pressure reducing station located on STH 42 near the intersection of Meadow Lane was not functional.

5.5 WATER DISTRIBUTION SYSTEM

The Village's water distribution system provides a means of transporting and distributing water from the supply sources to Utility customers and other points of usage. The distribution system must be capable of supplying adequate quantities of water at reasonable pressures throughout the service area under a range of operating conditions. Furthermore, the distribution system must be able to provide not only uniform distribution of water during normal and peak demand conditions, but must also be capable of delivering adequate water supplies for fire protection purposes.

The Village of Sister Bay's water system is comprised of approximately 17 miles of water mains ranging in size up to 12 inches in diameter. The current water main size inventory is summarized in Table 5-5. Of the 17 miles of water main, 3 percent are 10 inches in diameter or larger. These large diameter water mains represent the system's primary transmission facilities. The LGSD No. 1 distribution system is comprised of over 4 miles of water mains ranging in size up to 8 inches in diameter. The current water main size inventory is summarized in Table 5-6. Of these 4 miles of water main, 79 percent are 8 inches in diameter.

The 2005 water main inventory based on pipe age for the entire water system (including Liberty Grove Sanitary District) is summarized in Table 5-7. The pipe age summary was developed through the development of the Sister Bay water system computer model. Over 70 percent of the existing distribution system was installed prior to 1990. The entire water distribution system is composed of ductile iron pipe.

5.6 WATER SYSTEM CONTROLS

The water system controls are located in the Control Room at the Sister Bay wastewater treatment plant. The existing controls consist of a computer-based telemetry control panel, allowing operators to operate and control pumps, and monitor and trend elevated tank levels. System well and booster pumps are scheduled and automatically sequenced by operators using a pump selection matrix system that uses water levels in the elevated tanks for pump operating control. Additional well and/or booster pumps are operated based on decreasing water levels in the tanks.

The water level in the Standpipe serves as the primary control for Wells 1 and 2 well pump operation. The Jungwirth Tower serves as the primary control for Well 3 and the Sister Bay and Liberty Grove Booster Station pumps.

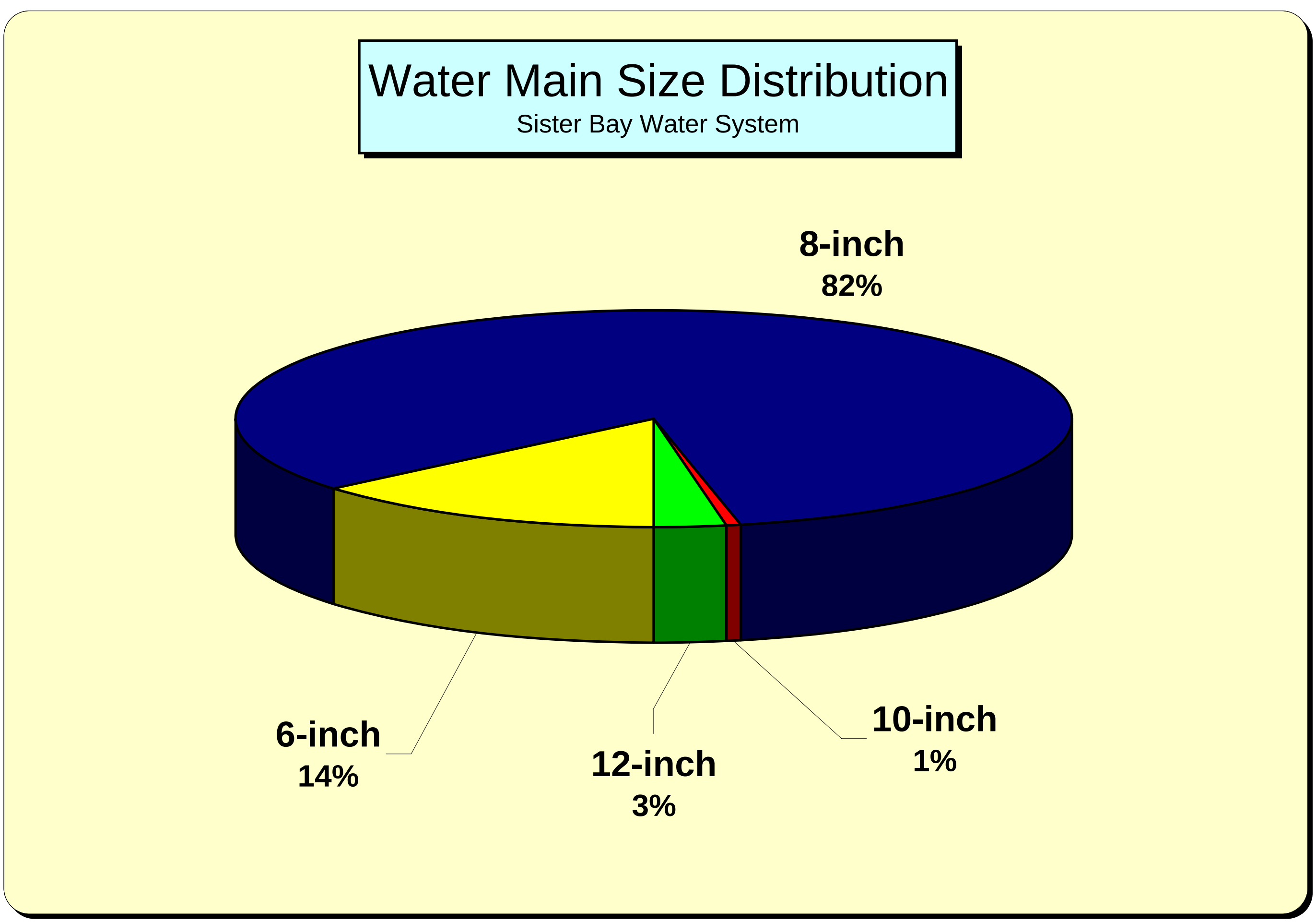
TABLE 5-5

WATER MAIN SIZE DISTRIBUTION

SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

Diameter (inches)	Approximate Total Length[1] (feet)	Percentage of Total
6	11,512	13.9%
8	68,659	82.8%
10	480	0.6%
12	2,269	2.7%
Total	**82,920**	**100.0%**

[1] Source: Sister Bay Water Utility 2004 PSC Annual Report



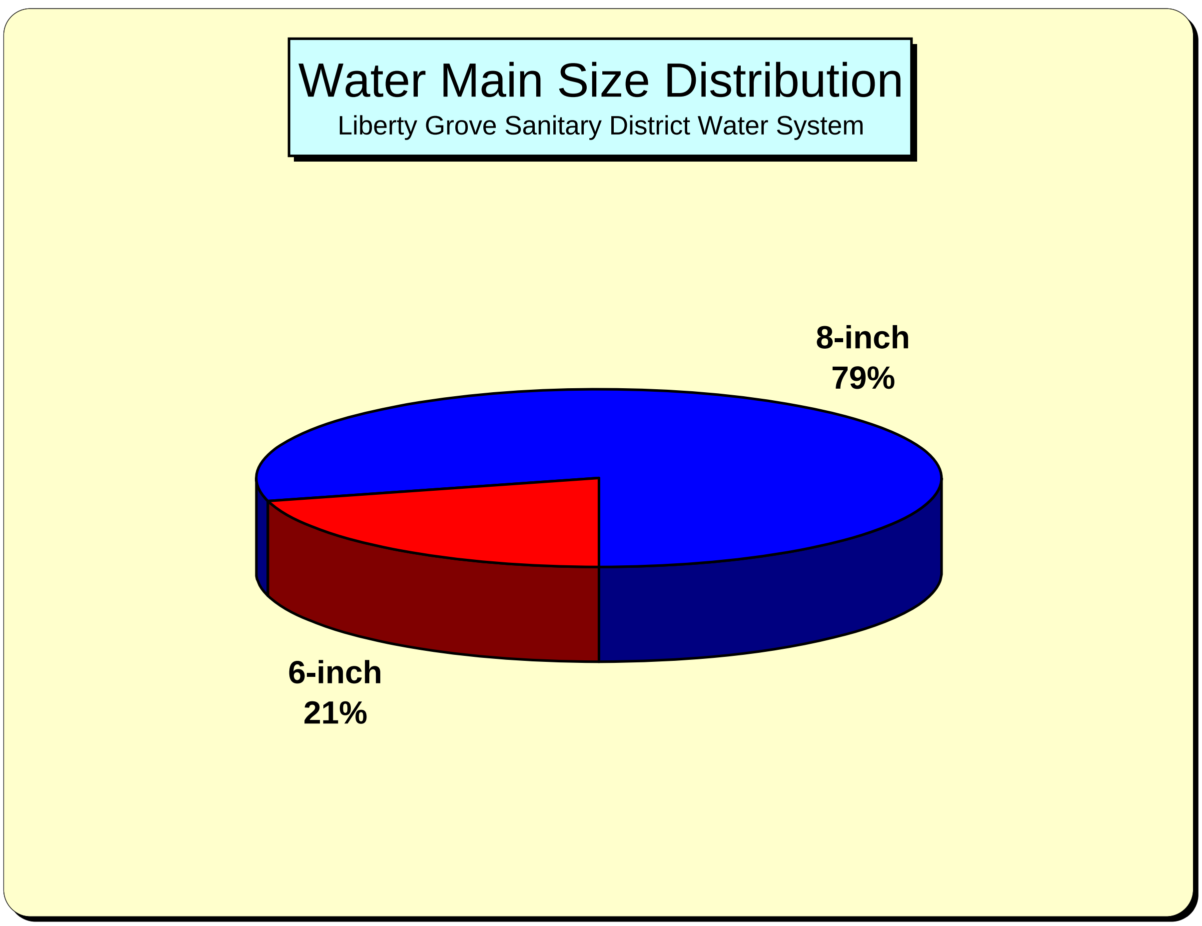
C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[table5-x.xls]Table 5-5

TABLE 5-6

WATER MAIN SIZE DISTRIBUTION
LIBERTY GROVE SANITARY DISTRICT NO. 1
VILLAGE OF SISTER BAY, WISCONSIN

Diameter (inches)	Approximate Total Length[1] (feet)	Percentage of Total
6	4,674	20.8%
8	17,808	79.2%
Total	**22,482**	**100.0%**

[1] Source: Liberty Grove Sanitary District No. 1 2004 PSC Annual Report



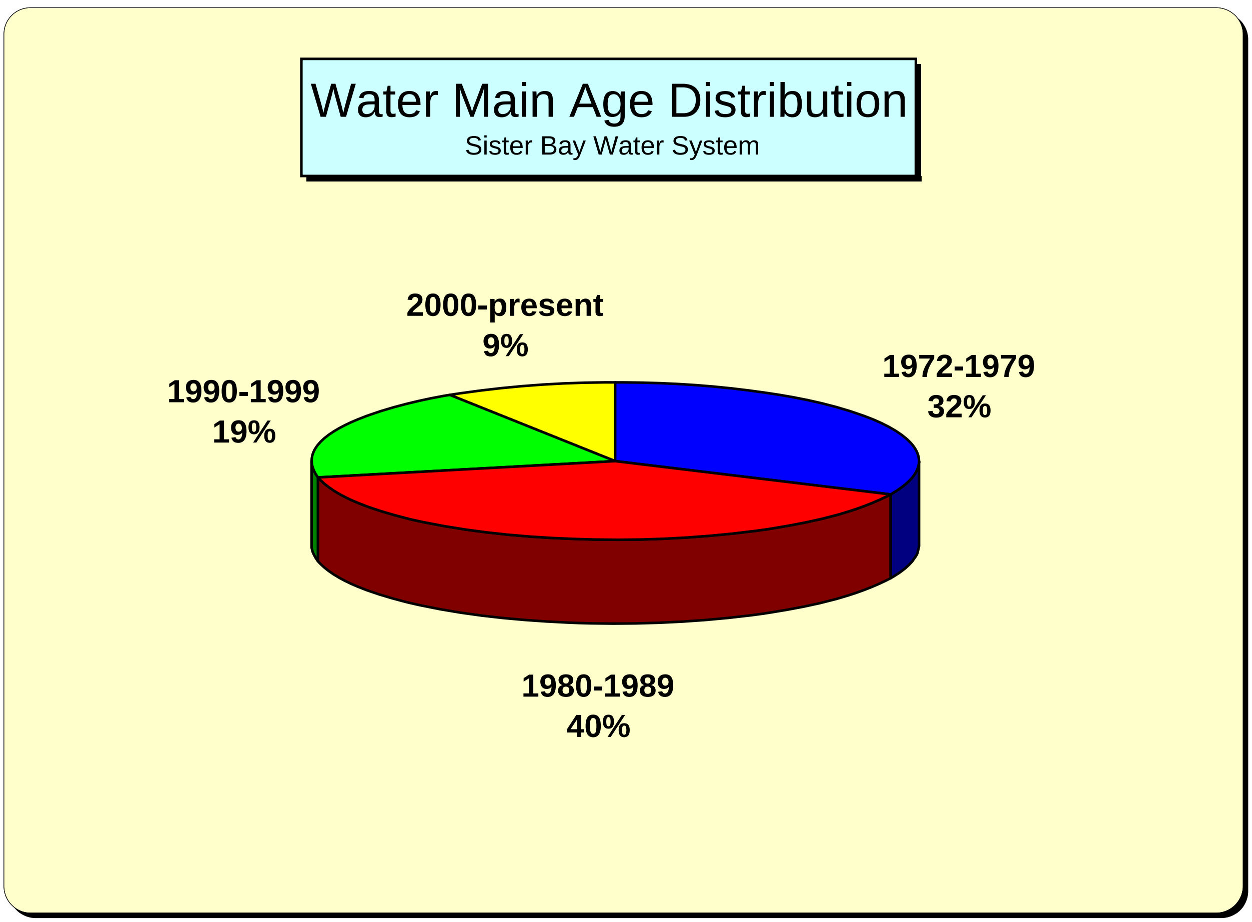
C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[table5-x.xls]Table 5-6

TABLE 5-7

WATER MAIN AGE DISTRIBUTION
SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

Pipe Material	Approximate Total Length[1] (feet)	Percentage of Total
1972-1979	37,127	31.9%
1980-1989	46,153	39.7%
1990-1999	22,515	19.3%
2000-present	10,584	9.1%
Total	**116,379**	**100.0%**

[1] Source: 2005 Sister Bay water system computer model (including Liberty Grove S.D.)



C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[table5-x.xls]Table 5-7

CHAPTER 6

WATER SYSTEM EVALUATION

An important component of this study was the evaluation of the existing water system and performing a deficiency analysis. This chapter summarizes the findings from this evaluation.

6.1 EXISTING SYSTEM DEFICIENCY ANALYSIS

Water systems are analyzed, planned, and designed primarily through the application of basic hydraulic principles. Some important factors that must be considered when performing this analysis include:

1. The location and capacity of supply facilities
2. The location, sizing, and design of storage facilities
3. The location, magnitude, and variability of customer demands
4. Water system geometry and geographic topography
5. Minimum and maximum pressure requirements
6. Land use characteristics with respect to fire protection needs
7. Other operational criteria which define the manner in which the system can most efficiently be operated

For this study, an evaluation of the Sister Bay water system was performed to determine the adequacy of the system to supply existing and future water needs and to supply water for fire protection purposes.

The system was evaluated based on the following criteria:

1. Pressure
2. Flow Capacity
3. Reliability
4. Supply
5. Storage

The water system evaluation was based on compliance with Wisconsin State code requirements and standard water industry engineering practice.

6.2 WATER SYSTEM COMPUTER MODEL

A computer model was developed of the Village's water distribution system. The Sister Bay system was modeled using *H₂OMap*, a pipe network program developed by MWH Soft. Individual system water pipe roughness coefficients were estimated based on the diameter and types of pipe materials, and approximate age of each section of water main using roughness aging curves developed from field testing of the Sister Bay system.

The Sister Bay water system model was calibrated using results of flow testing performed for this study in Fall 2005. Table 6-1 summarizes flow testing results. During the model calibration process, pumping rates, customer demands, and tower water levels were set to the field conditions, and pipe roughness coefficients were adjusted until the calibrated system model adequately simulated field test data.

TABLE 6-1

SYSTEM FLOW TEST RESULTS
SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

Date of Testing: September 26 & 27, 2005

Test No.	Hyd. No.	Flowing Hydrant Street	Flowing Hydrant Street	Flow (gpm)	Hyd. No.	Residual Hydrant Street	Residual Hydrant Street	Static (psi)	Residual (psi)
F-1	254	North end of Hillcrest Drive Cul-de-sac		420	316	First Hydrant south of flowing		55	41
F-2	314	North end of Beach Road		508	34	Beach Road	Bayview Road	84	62
F-3	18	Bay Shore Drive - 3rd Hydrant south of Waters End Road		750	16	Bay Shore Drive - 4th Hydrant south of Waters End Road		59	38
F-4	96	Trillium Lane east of Birchwood Dr.		780	62	Birchwood Drive west of Trillium Ln.		83	59
F-5	157	North end of West Little Sister Road		828	155	1st Hydrant south of flowing		87	57
F-6	101	Bay Shore Drive west of Forest Lane		1,108	97	Bay Shore Drive	Meadow Lane	67	51
F-7	329	West end of Sunnyside Road		922	317	Sunnyside Road	Sunnyside Ct.	45	40
F-8	223	Cherrywood Lane	Koessl Lane	1,632	125	2nd Hydrant north of flowing		45	34
F-9	318	Last Hydrant east of WWTP		922	186	1st Hydrant west of flowing		82	52
F-10	245	East of Smith Drive		953	241	2nd Hydrant west of flowing		60	51
F-11	13	Mill Road	South Spring Dr.	998	37	Mill Road	Park Lane	48	38

C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\[Table 6-1.xls]Results

6.3 WATER SYSTEM PRESSURES

The Sister Bay water system model was used to evaluate existing water distribution system characteristics and identify deficiencies with respect to pressures and flow capacities. Water system pressure will vary around the service area based on differences in topographic elevations, as well as supply rates and customer demands. In general, as customer demands increase, pressures will decrease. Areas higher in topographic elevation will also tend to exhibit lower water system pressures.

A water distribution system must be designed to provide pressures within a range of minimum and maximum allowable conditions. When system pressure is too low, customers may complain of inadequate water supply, customer meters may tend to record inaccurately, and fire protection will be limited. Pressures that are too high can cause problems with system operation and maintenance, and will tend to cause higher consumption rates by customers. High water system pressures can also increase the amount of water loss, as leakage rates will increase with increases in system pressure.

The Wisconsin Administrative Code requires that municipal water systems be designed with a minimum pressure of 35 psi and a maximum pressure of 100 psi at all locations in the service area under normal operating conditions. Furthermore, water systems are required to be operated so that under fire flow conditions, the residual pressure in the system will not fall below 20 psi at any location.

Highest system pressures, between 80 and 90 psi, typically occur in low topographic elevation areas of the HLPZ in the far western portions of the Village along the Green Bay shoreline, and in areas along Woodcrest Road and Scandia Road. The lowest system pressures (30 to 40 psi) can occur in the Main Pressure Zone along the pressure zone boundary near the Highway 42 and 57 intersection. Pressures in the LGSD No. 1 can range between 40 and 80 psi. The lowest pressures in the District are typically in the far eastern and northeaster portion of the service area.

Figure 6-1 illustrates ranges of water system pressures throughout the Village for a current typical peak hour demand condition. As indicated in the figure, peak hour system pressures can vary between 30 and 90 psi.

6.4 FIRE FLOW CAPACITIES

Water system planning for fire protection is an important consideration. In most instances, water main sizes are designed specifically to supply desired fire flows. Guidelines for determining fire flow requirements are provided by the ISO. ISO is the insurance service organization responsible for evaluating and classifying municipalities for fire insurance rating purposes.

Fire protection needs vary with the physical characteristics of each building to be protected. For example, needed fire flows for a specific building can vary from 500 gpm to as high as 12,000 gpm, depending on habitual classifications, separation distances between buildings, height, materials of construction, size of the building, and the presence or absence of building sprinklers. Municipal fire insurance ratings are partially based on the Village's ability to provide needed fire flows up to 3,500 gpm. If a specific building has a needed fire flow greater than this amount, the community's fire insurance rating will only be based on the water system's ability to provide 3,500 gpm.

Table 6-2 shows typical fire flow requirements for various land uses. The requirements shown in the table are only intended as a general guideline. The actual needed fire flow for a specific building can vary considerably as discussed above. The minimum fire flow requirements used as a basis for evaluating the

TABLE 6-2

TYPICAL FIRE FLOW REQUIREMENTS
SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

Land Use	Range of Needed Fire Flows (gpm)
Single & Two-Family:	
Over 100 feet Building Separation	500
31 to 100 feet Building Separation	750
11 to 30 feet Building Separation	1,000
10 feet or Less Building Separation	1,500
Multiple Family Residential Complexes	2,000 to 3,000+
Average Density Commercial	1,500 to 2,500+
High Value Commercial	2,500 to 3,500+
Light Industrial	2,000 to 3,500
Heavy Industrial	2,500 to 3,500+
Other Commercial, Industrial & Public Buildings	Up to 12,000

C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\[table6_x.xls]Table 6-2

Sister Bay water system were 1,000 gpm in medium and low density residential areas, and 2,000 gpm in all high density residential and commercial development areas.

Figure 6-2 illustrates the estimated available fire flow throughout the Village for a typical maximum day water demand while maintaining a residual pressure of 20 psi throughout the system. In general, the majority of the Village is well protected with minimum fire flows of 2,000 gpm or higher in most high density residential and commercial areas. Areas with lower available flows are primarily located on the far northeastern extremity of the system in the HLPZ, where available fire flows are less than 1,000 gpm. There is a small isolated area in the northeast corner of the HLPZ (LGSD No. 1) where available fire flows are less that the code required minimum of 500 gpm at 20 psi. The hydraulic strength of the distribution system is limited in this area, resulting in limited available fire flows.

Figure 6-3 identifies areas where fire flow deficiencies currently exist. Deficiencies were identified where the basic Sister Bay/Liberty Grove Fire Department guidelines were not met for a particular land use as determined from the planning area land use map. The primary areas identified with fire flow deficiencies include the low flow areas in the LGSD and several locations in the HLPZ with high density residential development.

6.4.1 Pipe Velocities, Head Loss, and Flow Carrying Capacity

Pipe flow velocities within the distribution system are typically well below 1 foot per second (fps) under average demand conditions. Even during periods of higher demand, flow velocities typically do not exceed 5 fps anywhere in the system. Water main pipe segments that have high flow velocities or head losses have limited flow or transmission capacity caused by the limited number and/or sizes of water mains.

6.5 SUPPLY RELIABILITY

For any water utility to serve its customers and protect the public welfare, water system facilities, equipment, and distribution systems must be reliable under all operating conditions. Reliability of utility service comprises a large part of the Water Utility’s investment in plant and equipment.

Wisconsin Administrative Code requires all pumping stations to be served by a power supply from at least two independent electrical substations, or from a standby, auxiliary power source dedicated to water supply use. As a general rule, the Utility should be able to reliably supply average day customer demands and maintain adequate fire protection using auxiliary power sources.

From a review of the alternative power and supply sources available, the system can supply approximately 2.1 mgd using standby power sources in the event of an emergency or other power interruption. Therefore, the system has sufficient auxiliary power to meet current needs and projected year 2025 average day pumpage requirements. It will be important for the Utility to continue to maintain a water supply capacity provided with auxiliary sources of power to meet a minimum of an average day water demand throughout the planning period.

6.6 WATER SUPPLY AND STORAGE

A critical step in long-range planning for the Sister Bay water system was identifying the future needs of the service area coupled with an assessment of water supply and storage requirements. Water supply and storage needs are closely related. The primary criteria used in determining required supply rates and storage volumes include maximum and peak demands, operational characteristics, and fire protection needs.

6.6.1 Reliable Supply Capacity

It is frequently necessary to take a well pump out of service for periods of several days to several weeks for maintenance. Therefore, the reliable supply capacity of a water system is the total available delivery rate with the largest pumping unit out of service. For example, under present operating conditions, the existing wells have a combined total capacity of approximately 1,440 gpm as shown in Table 6-3. However, the reliable capacity of the supply wells is approximately 950 gpm with the largest unit (Well 1) out of service.

For evaluating a municipal water system, reliable supply capacity should at least equal maximum day pumpage requirements, assuming adequate storage is available. If this criterion is met, supply facilities will have adequate capacity to replenish storage during off peak hours, while depletion of available storage occurs during peak demand hours. Using this criteria, and projections of future water supply needs, Table 6-4 summarizes minimum future supply needs.

The water utility currently has adequate supply capacity, as the existing reliable capacity (1.37 mgd) is larger than the current design maximum day pumpage of 0.77 mgd. Figure 6-4 compares historical water supply capacities with historical maximum day pumpage requirements. As illustrated in the figure, the utility should have adequate reliable supply capacity to meet current maximum day demands in the system.

Figure 6-5 compares Sister Bay water supply capacities with historical and projected maximum day pumpage requirements. As illustrated in Figure 6-5, the Utility has sufficient reliable supply capacity to meet current and future water needs throughout the planning period.

6.6.2 Water Storage Needs

In addition to providing water for fire protection, system storage is used as a “cushion” to equalize fluctuations in customer demands, establish and maintain water system pressures, provide operational flexibility for water supply facilities, and improve water supply reliability. The primary criteria used in this study for evaluating storage volume needs includes average and peak demands, water supply capacities, and fire protection needs.

In general, storage facilities should be adequately sized to provide sufficient quantities of water for fire protection on days of maximum customer demands. Although storage requirements for fire protection are not anticipated to change over the planning period of this study, peak hour demands and reliable supply capacities will change as the Village grows and improvements are implemented.

Figure 6-6 illustrates general categories of system storage. As customer demands exceed supply capacities during peak hour conditions, these excess demands must be met by depleting available storage. The amount of storage depleted is referred to as equalizing storage for peak hour requirements. Storage should also be available for fire protection purposes. To assure a reliable supply for fire protection, this reserve storage should not be utilized to meet peak hour requirements.

In some instances, it may be desirable to provide additional reserve storage for other purposes. Reserve storage may be needed as a safety factor in emergencies or where customer demands are unpredictable

TABLE 6-3

RELIABLE SUPPLY CAPACITY

SISTER BAY WATER UTILITY

VILLAGE OF SISTER BAY, WISCONSIN

Supply Source	Current Minimum Operating Capacities	
Well Pumps:	**(gpm)**	**(MGD)**
Well 1	490	0.71
Well 2	480	0.69
Well 3	470	0.68
Total Supply Capacity	1,440	2.07
Less: Largest Supply Unit	490	0.71
RELIABLE SUPPLY CAPACITY	**950**	**1.37**

Notes

1. Approximate minimum operating capacities of well pumps based on current available operating data.

C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\[table6_x.xls]TABLE 6-3

TABLE 6-4

RECOMMENDED SUPPLY CAPACITY

SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

	Actual 2005	Projected 2015	Projected 2025
Total Annual Pumpage (MGY)	89	100	122
Average Day Pumpage (MGD)	0.24	0.27	0.33
Design Maximum Day Pumpage (MGD)	0.77	0.86	1.05
Existing Reliable Supply Capacity (MGD)	1.37	1.37	1.37
ADDITIONAL CAPACITY REQUIRED (MGD)	**None**	**None**	**None**
ADDITIONAL CAPACITY REQUIRED (gpm)	***0***	***0***	***0***

Note

Design maximum day pumpage requirements were estimated based on 315% of average day pumpage.

* The above figures are based on the pumps running 24 hours per day.

C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\Report\Chapter 6\[table6_x.xls]TABLE 6-4 and fluctuate widely. Additional storage may also be desired where a Utility wishes to take advantage of off peak electrical rates for pumping. Additional reserve storage of approximately 10 to 15 percent is usually provided to allow an operating range for well and booster pump operation.

Three primary criteria were used to develop a relationship between supply capacities and optimum storage volumes for the Village of Sister Bay:

1. Reliable supply capacity should at least equal projected maximum day pumpage requirements.

2. Total available storage should be capable of meeting fire protection needs, assuming reliable supply capacity is just adequate to meet maximum day requirements. A base fire flow of 2,000 gpm for three hours was used.

3. Reliable supply capacity, plus available storage volume, should equal or exceed fire flow requirements plus maximum day requirements.

6.6.3 Available Storage Capacity

Total available system storage was calculated based on the effective storage volume available from the elevated tank and standpipe. The effective storage volume of the elevated storage tanks is the volume in storage above the lowest water level that could by maintained and provides minimum required pressures in the system. Under normal conditions, system pressures are required to be maintained above 35 psi. Under emergency conditions, pressures may be reduced to 20 psi.

Figure 6-7 illustrates this relationship between standpipe storage volume and required minimum water levels needed for establishing system pressures. Effective peak hour operating storage and emergency storage volumes for the Sister Bay system have been determined based on minimum required system pressures in the distribution system.

The effective storage volume from the standpipe is summarized in Table 6-5. Under normal operating conditions, the standpipe water level can drop approximately 4 feet below the overflow level before system pressures can fall below the required minimum 35 psi at the highest ground elevations served by the Main Pressure Zone. Therefore, the maximum effective peak hour storage volume of the standpipe is approximately 9,000 gallons. During fire flow or emergency situations, the standpipe water level can drop an additional 35 feet, resulting in an additional 73,000 gallons of water available. Therefore, the maximum total effective storage volume of the 0.1 MG standpipe is approximately 0.082 MG.

6.6.4 Supply and Storage Requirements

The amount of water storage required is related to available supply capacity. As supply capacity is increased, the amount of storage required decreases. This relationship is illustrated in Figure 6-8, which is a plot of supply and optimum storage requirements for the Sister Bay water system in the years 2005 and 2025. Optimum storage requirements were estimated assuming future supply capacities would just equal maximum day demands.

A point is plotted on the graph in Figure 6-8 that represents existing conditions where Sister Bay supply facilities have a reliable capacity of 950 gpm, and the total storage available is 0.232 MG. To comply with the design criteria specified earlier, the point that corresponds to actual reliable supply and storage capacities should fall on or above the supply-storage curves indicated in this figure. As illustrated in the

TABLE 6-5

EFFECTIVE STANDPIPE VOLUME
SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

Design Volume (gallons)	100,000
Diameter (feet)	19
Head Range (feet)	48
Storage volume per foot (gallons)	2,100
Overflow elevation (feet USGS)	730
Highest elevation served in Main Zone (feet USGS)	645
Hydraulic Grade Elevation needed to provide minimum 35 psi to all areas	726
Maximum Effective Peak Hour Storage Volume (gallons)	9,000
Hydraulic Grade Elevation needed to provide minimum 20 psi to all Main Zone areas	691
Additional Effective Fire Protection and Emergency Storage Volume (gallons)	73,000
Total Effective Storage Volume (gallons)	**82,000**

Notes

1. Effective peak hour storage is considered the volume available which will continue to maintain pressures in the distribution system at a minimum of 35 psi.
2. Effective fire protection and emergency storage is considered the volume available which will continue to maintain pressures in the distribution system of a minimum of 20 psi.

C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\[Table6-5.xls]Sheet1 figure, while the Utility currently has adequate reliable supply capacity, there is inadequate water storage volume available to meet present and future system needs. Current and projected Sister Bay supply and storage needs are summarized in Table 6-6.

6.7 SUMMARY

This chapter summarized the findings from evaluation of the Sister Bay water system. Major findings from this evaluation include the following:

1. Under all normal operation conditions, the system provides pressures between 30 and 90 psi. There is only a very small isolated area where distribution system pressures can fall below the minimum required 35 psi. There are no locations in the existing water service area where pressures can exceed the maximum 100 psi.

2. There are several large areas within the HLPZ where available fire flows are below recommended minimum flows.

3. The system can adequately supply water to meet average day customer demands in using standby power generating equipment throughout the planning period.

4. The Utility has adequate reliable water supply capacity to meet current and projected future supply needs throughout the planning period.

5. The Utility has inadequate water storage volumes available to meet current and projected future storage needs.

TABLE 6-6

SUPPLY AND STORAGE NEEDS

SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

SUPPLY REQUIREMENTS	**Actual 2005**	**Projected 2015**	**Projected 2025**
Recommended Reliable Supply Capacity (gpm)	530	600	730
Present Maximum Day Reliable Supply Capacity (gpm)	950	950	950
Additional Capacity Required (gpm)	**None**	**None**	**None**

STORAGE REQUIREMENTS	**Actual 2005**	**Projected 2015**	**Projected 2025**
Peak Hour Equalizing Requirements (gallons)	97,000	109,000	133,000
Optimum Fire Protection Needs (gallons)	360,000	360,000	360,000
Reserve Storage (gallons; 10% of Total)	50,000	52,000	54,000
Total Optimum Storage Requirements (gallons)	**507,000**	**521,000**	**547,000**
Total Storage Capacity (gallons):			
Jungwirth Tower	150,000	150,000	150,000
Hwy 57 Standpipe	82,000	82,000	82,000
Total	**232,000**	**232,000**	**232,000**
Additional Capacity Required (gallons)	***275,000***	***289,000***	***315,000***

Notes

1. Peak hour storage is storage required to meet demands which exceed the reliable supply capacity. Future peak hour equalizing storage requirements were calculated assuming the available supply is equal to the maximum day demand rate.
2. Reserve storage is storage required to provide a start/stop range for well pump operation and an emergency reserve storage supply.

C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\Report\Chapter 6\[table6_x.xls]TABLE 6-6

CHAPTER 7

RECOMMENDED WATER SYSTEM IMPROVEMENTS

This chapter summarizes recommended water system improvements. The following categories of improvements are discussed:

- Water storage improvements
- Water service to outlying areas
- Distribution system improvements
- Distribution system expansion

Based on projected growth planned for the Sister Bay Water Utility service area, the water system will require improvements to accommodate future service needs and address existing system deficiencies.

7.1 WATER STORAGE IMPROVEMENTS

To address the system's existing storage deficiency, the Water Utility currently requires an additional 0.15 MG of water storage volume. Based on projected water demand growth over the planning period and to meet the water storage needs of the planning area, the Utility will require an additional 0.25 MG of water storage volume by the year 2025. The additional recommended water storage volume needed is illustrated in Figure 7-1.

In general, a water utility has three types of storage facilities to choose from when additional water storage is required. Storage facility alternatives include:

- Clearwell Storage
- Ground Storage
- Elevated Storage

Storage located adjacent to the water supply or treatment facilities is generally defined as clearwell storage. Clearwell storage is provided to meet peak demands which exceed water supply and/or treatment production rates and to allow production facilities to operate at a constant rate which results in more uniform and efficient operation.

Ground storage is simply storage located on or beneath the ground. It is generally located within the distribution system network to provide equalization of system pressures and to supply peak or fire flow water demands. For the Sister Bay system, water from ground storage facilities would be required to be pumped into the distribution system.

Sister Bay currently utilizes an elevated tank and a standpipe to provide water system storage. Advantages of elevated storage include an increase in system reliability and reducing the need to construct large size mains to the system extremities. In contrast to ground storage, elevated storage provides increased reliability for fire protection and for emergencies during power outages or other pumping interruptions.

For this study, two types of storage, elevated and ground level, were considered suitable alternatives for addressing the planning area's additional storage needs. Table 7-1 summarizes the primary advantages and disadvantages of these two alternative storage types.

7.1.1 Alternative 1 - Ground Storage and Booster Pump Station

Alternative 1 involves the construction of a new 250,000 ground storage tank and booster pumping station. Ground storage is typically less costly to construct and maintain than elevated storage. However, a ground storage reservoir will require the Utility to construct and operate a booster pumping station. Therefore, operating costs would be higher with this option, because the stored water must be re-pumped into the distribution system.

A primary advantage of implementing a ground reservoir and pump station would be the ability of the booster pumps to overcome the limited hydraulic capacity of existing distribution system, especially in the LGSD No. 1 area. In addition, the ground reservoir approach could be constructed in phases (separate reservoirs) that would provide the Utility with the flexibility to add additional ground storage volume in the future to meet growing planning area needs. Water stored in ground reservoirs is less likely to be subjected to freezing problems compared to elevated storage. Finally, a ground storage tank could be constructed in two segments that would allow one-half of the reservoir to be removed from service for maintenance (or for seasonal operational needs), while the other half continues to function for the system.

7.1.2 Alternative 2 - Water Tower (Elevated Storage) in High Level Zone

Alternative 2 involves construction of a new 250,000 gallon water tower in the northern or central part of the High Level Pressure Zone. The primary advantages of this alternative include added reliability of elevated storage versus ground storage, lower operating costs incurred using elevated storage, and simple control methodologies needed to operate the system. This alternative should not significantly impact existing pump operating procedures of the Utility. To provide the greatest benefit to the identified lower pressure and lower fire flow areas, a new elevated tank should ideally be located close to the northeastern portion of the existing distribution system, serving the High Level Pressure Zone. A second elevated tank serving the HLPZ would work in conjunction with the existing Jungwirth Water Tower to establish pressure for the HLPZ, and provide a reliable supply of water held in storage to meet the additional water storage needs of the Utility. The additional water provided by this storage facility would be available in the Main Pressure Zone through the existing interzone PRV stations.

The primary disadvantages of constructing an additional elevated storage facility in Sister Bay would be higher capital and maintenance costs compared to costs for the same storage in a ground level facility, and concerns regarding potential water stagnation and freezing problems. Operation of a second, larger volume water tower will be problematical for the Utility, as the current average daily demand in the HLPZ is estimated to less than 0.15 mgd, while the total volume of elevated storage operated in the zone would be would be 0.40 MG.

Of even greater concern for operation of two HLPZ elevated tanks would be the current minimum-day HLPZ demand of less than 0.10 mgd, or less than one-quarter of the proposed elevated water storage volume including a third elevated tank. The potential for significant water freezing problems in elevated tanks rises sharply when winter season elevated tank turnover exceeds 2-3 days; it would be over 4 days with the addition of a new 250,000 gallon elevated tank.

TABLE 7-1

GROUND AND ELEVATED STORAGE COMPARISONS

SISTER BAY WATER UTILITY

VILLAGE OF SISTER BAY, WISCONSIN

Advantages of Adding Additional Elevated Storage	Advantages of Adding Ground Storage
• Reliability of emergency supplies • Better water system pressure equalization which helps minimize pressure variations and reduce surging • Can be located based on water system hydraulics to minimize or eliminate need for large diameter system mains • May be possible to take advantage of off-peak electric rates to reduce pumping costs	• Lower initial construction costs (may be offset by cost of booster pumping facilities) • Lower maintenance costs • Less significant visual impact on surrounding properties • Usually less susceptible to freezing problems

Disadvantages of Adding Additional Elevated Storage	Disadvantages of Adding Ground Storage
• Available flow capacity limited by capacity of distribution system mains to transport water from tank to area of need • More susceptible to freezing problems during winter months • Significant visual impact • Higher cleaning and painting costs	• Higher operating costs associated with need to pump stored water into system and inefficiencies in dual pumping systems • Available delivery rates limited by capacity of booster pumping equipment • Pressure variations may occur when booster pumps are operated

C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\[Table 7_x.xls]Table 7-1

Finally, the limited hydraulic capacity of the northern portion of the Sister Bay distribution system does not lend itself to the easy siting of a new water tower. A tower located in the northern part of the existing service area would provide minimal benefits to the southern portion of the distribution system without significant transmission main improvements. Similarly, a new tower located in the southern part of the existing service area would provide minimal benefits to the northern portion of the distribution system without significant transmission main improvements.

7.1.3 Alternative 3 - Water Tower (Elevated Storage); Combined Pressure Zones

Alternative 3 is similar to Alternative 2, and involves construction of a new 250,000 gallon water tower in the northern or central part of the High Level Pressure Zone to address the existing and future system water storage deficiency. However, in addition to the added elevated storage, Alternative 3 would also eliminate the two pressure zone system operation, by combining both pressure zones into a single zone.

Combining the pressure zones into a single zone would require the following:

- Abandon the existing seven PRV stations located on the boundary between the pressure zones
- Open all closed water main isolation valves located on the existing pressure zone boundary
- Modify Well 1 pump equipment to allow well pump to operate against the additional 90+ feet of head
- Operate the existing Hwy 57 Standpipe as a ground reservoir

The primary advantages of this storage improvement alternative are the same as Alternative 2, (greater reliability, lower operating costs, simple system control operations), but also address the water storage turnover concerns of Alternative 2. The greater demand of the combined zones would significantly reduce turnover concerns. In addition, the weak system hydraulic concerns of the northern distribution system would be eliminated by combining the pressure zones, and significantly increased available fire flows throughout the central and northern system area. Operation and maintenance of the PRV stations would also be eliminated.

The Alternative 3 water tower is still recommended to be located in the northern or central part of the existing water service area (similar to Alternative 2), and would work in conjunction with the existing Jungwirth Water Tower to establish pressure for the entire water distribution system. The standpipe would no longer establish pressure for the system, but would need to be operated as a ground reservoir.

The primary disadvantage of combining the pressure zones into a single zone would be the increased normal operating pressure throughout the Main Zone distribution system area. Pressures would increase between 35 to 40 psi in the existing Main Zone service area. Pressures in the lowest lying areas along the Green Bay shoreline would be increased to between 95 and 100 psi; however, there may be isolated properties with pressures slightly above 100 psi. The higher available system pressures would generate significantly higher available fire flows throughout the existing Main Zone service area.

7.1.4 Storage Alternative Evaluation

The three storage improvement alternatives were screened for feasibility. Each alternative plan was evaluated with respect to each other on the basis of functional water utility operational standards. The results of this initial screening are summarized in the table below. For the terminology used in the table, a "marginal" rating indicates that, although the alternative may meet minimum criteria, it is clearly inferior to the other alternatives, or is of doubtful long-term suitability. An "adequate" rating describes an alternative which more than meets the minimum criteria, but which exhibits either long-term unsuitability, or is not as desirable as other plans. Those alternatives that provide superior performance with the capability of meeting or exceeding all anticipated criteria, including long-term suitability, were rated as "superior".

Functional Standard	**Alternative No. 1**	**Alternative No. 2**	**Alternative No. 3**
System Pressure	Adequate	Superior	Adequate
Fire Flows	Adequate	Adequate	Superior
System Hydraulics	Adequate	Adequate	Superior
Water Storage Turnover	Adequate	Marginal	Adequate
Reliability	Adequate	Superior	Superior
Operational Flexibility	Adequate	Adequate	Superior
Operating Cost	Marginal	Superior	Superior
Maintenance Cost	Superior	Marginal	Marginal
System Control	Marginal	Adequate	Superior

Based on the preliminary screening of the three storage improvement alternatives, Alternative Nos. 1 and 3 are clearly superior to Alternative No. 2. Therefore, further evaluations of Alternatives No. 1 and 3 were performed, and are summarized below.

7.1.4.1 Storage Alternative No. 1 Evaluation

The criteria used to evaluate the location for a recommended ground storage reservoir/booster pump facility for Sister Bay included the following:

- Land availability
- Proximity to large water mains
- Compatibility with distribution system hydraulics
- Proximity to areas with high fire protection needs
- Proximity to future growth areas
- Compatibility of reservoir aesthetics with surrounding land uses
- Impact of future reservoir maintenance activities on surrounding property

Based on a review of potential planning area site alternatives using the above criteria, the recommended location for a new 0.25 MG ground reservoir and associated booster pump station is adjacent to the Sister Bay wastewater treatment facility on Village-owned land. This location is superior to all other potential reservoir site alternatives with respect to the siting criteria.

A new booster pump station associated with a new ground reservoir should be designed with an overall pumping capacity of 2,000 gpm with multiple pumping units (3 minimum). To minimize/eliminate pressure surging of the system during pump startup, all pump motors should include variable frequency drive units.

Figure 7-2 illustrates computer simulated 2025 water system peak hour pressures throughout the planning area assuming Alternative No. 1 is implemented. As indicated in the figure, the majority of the future service area can be adequately served by the existing HLPZ water system. Only two future service areas cannot be served adequately with a minimum pressure of 35 psi under all normal operating conditions. These areas include higher elevation land south and west of Country Lane in the far southwestern corner of the planning area; and the corridor along STH 42 northeast of LGSD No. 1. A higher pressure plane is needed in these areas to ensure that all customers can be provided with a minimum water pressure of 35 psi under all normal operating conditions as required by Wisconsin Administrative Code, Chapter NR 811.

Figure 7-3 illustrates computer simulated 2025 water system available fire flows throughout the planning area assuming Alternative No. 1 is implemented. Available fire flows were modeled assuming a minimum system residual pressure of 20 psi. Future transmission main extensions were included in the Year 2025 system model. Significant transmission main improvements would be required to serve areas north of LGSD No. 1.

As indicated in Figure 7-3, and very similar to the modeling results illustrated in Figure 7-2, the majority of the future service area can be adequately served by the existing HLPZ water system. Only two future service areas cannot be served adequately with the minimum recommended fire flows under a Year 2025 maximum day demand condition. These areas include higher elevation land south and west of Country Lane in the far southwestern corner of the planning area; and the corridor along STH 42 northeast of LGSD No. 1 and north of Seaquist Road. These areas also cannot be served with adequate pressures.

To adequately serve these areas in the future with the minimum recommended fire flows, a higher pressure plane is needed and adequate booster pumping capacity required.

A preliminary budget estimate for the Alternative No. 1 ground reservoir and booster pump station improvements is $1,075,000. Table 7-2 summarizes a preliminary budget estimate for the Alternative No. 1 water storage improvements.

7.1.4.2 Storage Alternative No. 3 Evaluation

The same criteria used to evaluate the location for a recommended ground storage reservoir were used for siting a new elevated storage tank. Based on a review of potential planning area site alternatives using the storage tank siting criteria, the recommended location for a new 0.25 MG water tower is also adjacent to the Sister Bay wastewater treatment facility on Village-owned land. This location is superior to all other potential water tower site alternatives with respect to the siting criteria.

Figure 7-4 illustrates computer simulated 2025 water system peak hour pressures throughout the planning area assuming Alternative No. 3 is implemented. As indicated in the figure, the majority of the future service area could be adequately served by the combined, single pressure zone water system. There would be only one future service area could not be served adequately with a minimum pressure of 35 psi under all normal operating conditions with Storage Alternative No. 3 implemented. This area includes the higher elevation land south and west of Country Lane in the far southwestern corner of the planning area. The vast majority of the corridor along STH 42 northeast of LGSD No. 1 could be adequately served with Alternative No. 3, with the exception of the small area in the extreme, far northeastern corner

TABLE 7-2

PRELIMINARY BUDGET ESTIMATE
GROUND RESERVOIR & BOOSTER PUMP STATION

SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

Description	Estimated Cost
Land Acquisition	See Note Below
250,000 gallon Ground Reservoir	$375,000
Booster Pump Station	$425,000
Site Work Allowance	$25,000
Total Estimated Construction Cost	$825,000
Administrative, Engineering, Financing, Legal, & Construction Contingency Costs (30%)	$250,000
Total Estimated Project Cost	**$1,075,000**

Note: Reservoir site property owned by Village.

C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\[Table 7_x.xls]Table 7-2 of the future service area. Due to the limited area involved, it is not cost effective to provide service to this area, and service is not recommended at this time. Normal system operating pressures between 90 and 100 psi could be expected in low lying areas along the Green Bay shoreline.

Figure 7-5 illustrates computer simulated 2025 water system available fire flows throughout the planning area assuming Alternative No. 3 is implemented. Available fire flows were modeled assuming a minimum system residual pressure of 20 psi. Future transmission main extensions were included in the Year 2025 system model.

As indicated in Figure 7-5, the majority of the existing and future service area would have a significant increase in available fire flows from the distribution system. Only the future service area southwest of Country Lane could not be served adequately with the minimum recommended fire flows under a Year 2025 maximum day demand condition. This area also cannot be served with adequate pressures.

To adequately serve this area in the future with the minimum recommended fire flows, a higher pressure plane is needed and adequate booster pumping capacity required.

A preliminary budget estimate for the Alternative No. 3 elevated water storage tank improvements is $995,000. This estimate includes a very short 12 inch water main that would be required to connect the recommended water tower to the existing water system, and pump modifications for the Well 1 pump. The general location recommended for the new water tower is adjacent to the wastewater treatment plant. Table 7-3 summarizes a preliminary budget estimate for the Alternative No. 3 water storage improvements.

7.1.5 Recommendations

7.1.5.1 Water Storage Approach

Alternative No. 3 is the recommended storage alternative for the Sister Bay water system. Reliability is a primary advantage of elevated storage. Because water in storage is directly connected to the water system, no mechanical devices are required to deliver water from storage to the system when it is needed.

Alternative No. 3 has additional operational and hydraulic advantages over Alternative No. 1. Increased system pressures in low lying areas of the existing Main Zone will approach the DNR Code maximum, but will allow fire flows to be greatly increased in areas where greater fire flows are needed and higher property values exist. The Utility can eliminate the operation of all seven PRV stations, and no significant changes in equipment or operation would be required at Wells 2 and 3. The Utility should consider installing individual PRVs on all customer water services in low elevations areas immediately adjacent to Green Bay. Finally, Alternative No. 3 is estimated to be less costly to implement.

Therefore, it is recommended that a new water tower be constructed to serve the Sister Bay water system to meet the current and future storage needs of the planning area.

7.1.5.2 Water Storage Location

The new tower location adjacent to the wastewater treatment plant is the recommended site of the new elevated water storage tank. However, the comparative analysis described in Section 7.1.4 was performed looking at service levels within the entire Year 2025 planning area, assuming the needed future transmission mains are in place.

TABLE 7-3

PRELIMINARY BUDGET ESTIMATE: ALTERNATIVE NO. 3
ELEVATED WATER STORAGE TANK

SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

Description	Estimated Cost
250,000 gallon Elevated Tank	$700,000
Site Work Allowance	$25,000
Well 1 Pump Modifications	$40,000
Total Estimated Construction Cost	$765,000
Administrative, Engineering, Financing, Legal, & Construction Contingency Costs (30%)	$230,000
Total Estimated Project Cost	**$995,000**

Note: Tower site property owned by Village.

P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay copy\Report\Chapter 7\Revised 11-2006\[Table 7-x revised.xls]Table 7-3

Many of the transmission mains in the northern section of the future service area may not be constructed for many years. Therefore, to better understand the probable impacts of a second water tower serving the current water system, additional computer modeling of the existing water system was performed. Two other potential tower sites were also modeled to evaluate any significant differences on existing system pressures and fire flows using the two vs. one pressure zone approach. The anticipated hydraulic impacts to the current system are graphically illustrated in the figures in Appendix E.

As illustrated in the figures in Appendix E, there are no significant differences in anticipated water system pressures between the three different tower locations using either pressure zone approach. However, as illustrated previously in Figure 7-5, anticipated fire flows throughout the entire water system are all considerably higher using the combined pressure zone approach.

A new water tower located near the wastewater treatment plant will provide the following system benefits:

- System pressures and fire flows in the LGSD No. 1 area would be increased.
- Elimination of pressure fluctuations in the LGSD No. 1 service area.
- Higher needed fire flows in high density and commercial development areas would be provided.
- Land acquisition and associated costs are eliminated. Land is available (Village-owned property).
- Site is in close proximity to largest system water mains.
- Site is very compatible with existing and proposed future distribution system hydraulics.
- Site is near areas with high fire protection needs and near future growth areas.
- Water tower aesthetics are compatible with surrounding land uses (park and wastewater plant buildings).
- There would be minimal impact of future tower maintenance activities on surrounding private property. Park open space adjacent to the wastewater treatment plant is ideal for construction and future maintenance of a water tower.

Therefore, based on these reasons, it is recommended that the Village construct the proposed new water tower on Village-owned park land adjacent to the wastewater treatment plant.

7.1.5.3 Water Storage Volume

Based on the supply and storage analysis performed for this study, the Water Utility needs an additional 250,000 gallons of storage or an additional 1,500 gpm of supply capacity (or a combination of both) to meet the projected system supply and storage needs by the end of the 2025 planning period. Implementing an additional 1,500 gpm of new supply capacity (3 new supply wells) is not a cost effective approach for meeting these projected requirements.

Providing all of the needed storage for the 20-year planning period in the new water tower is not recommended. Implementation of a new 250,000 gallon water tower will create significant operational problems in the winter months, when the overall system demand falls to below 150,000 gallons per day, while the Utility would be operating 400,000 gallons of elevated storage, plus the 100,000 gallons stored in the Standpipe. Tank water freezing problems due to lack of water turnover will be a major concern for the Utility following the construction of the new water tower, even with taking the Standpipe out of service in the winter months.

A more operationally feasible approach to address long-term storage needs for the planning area and address shorter-term operational concerns would be to construct a smaller, 150,000 gallon water tower. The increasing supply and storage needs of the system should be evaluated immediately after placing the new tower into service, and then regularly evaluated at a minimum of every 3 years.

As noted above, the additional supply/storage needs can be met by implementing a new supply well project or by adding additional ground storage. A suitable site for additional ground storage is adjacent to the Standpipe. This site is already adequately equipped to pump water from the Standpipe into the distribution system. The Utility recently had the Highway 57 Standpipe inspected to determine any maintenance needs and its current structural condition. The inspection report indicated that the 34-year old Standpipe is in excellent condition. With proper regular maintenance, this storage facility should serve the water system throughout the planning period of this study. Therefore, replacing the Standpipe with a larger ground storage facility is probably not cost effective during the planning period. Adding a second ground storage reservoir at the site is one recommended alternative.

A second alternative that could be considered following the completion of the new water tower project is implementing a new water supply well. It is usually not cost-effective to increase a municipal water system's supply capacity when there already exists sufficient capacity to meet current or projected maximum day system demands. Given the Sister Bay Water Utility's very large seasonal water demand variation between summer and winter, implementing additional supply over storage may be more operationally cost effective than constructing and maintaining additional water storage capacity. Potential distribution system water quality concerns from operating excessive storage volumes would not exist; but other operational issues would also have to be addressed in operating a 4th water supply well (wellhead protection planning and zoning issues, potential contamination concerns, additional sampling requirements, routine well and pump maintenance, etc.). Figure 7-6 illustrates the water supply and storage improvements recommended to be implemented during the planning period of this study.

It is recommended that the Utility construct a second 150,000 gallon water tower adjacent to the Village's wastewater treatment plant. Table 7-4 summarizes the budget estimate for the recommended water tower storage and combined pressure zone improvement project.

It is also recommended that the Utility plan for a future 100,000 gallon ground reservoir adjacent to the Standpipe, and begin looking for potential Well 4 sites. The Utility should plan on budgeting $350,000 for a future 100,000 gallon ground reservoir project. To minimize interference effects between the existing wells and a future Well 4, it is recommended that future well sites be planned in the western and/or southwestern areas of the Village of Sister Bay. A possible site for future Well 4 could be on Village-owned property immediately adjacent to the Jungwirth Tower. The Utility should plan on budgeting $800,000 for a future Well 4 project.

7.2 WATER SERVICE TO OUTLYING AREAS

The recommended combined pressure zone distribution system would be adequate to serve areas with ground elevations ranging up to approximately 730 feet USGS. The ground elevations proposed to be served in the outlying planning area range up to over 750 feet USGS. The combined pressure zone could only adequately serve future development within the planning area with topographic elevations less than

TABLE 7-4

PROJECT BUDGET ESTIMATE
ELEVATED WATER STORAGE TANK

SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

Description	Estimated Cost
150,000 gallon Elevated Tank	$620,000
Site Work Allowance	$25,000
Well 1 Pump Modifications	$40,000
Total Estimated Construction Cost	$685,000
Administrative, Engineering, Financing, Legal, & Construction Contingency Costs (30%)	$210,000
Total Estimated Project Cost	**$895,000**

Note: Tower site property owned by Village.

P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay copy\Report\Chapter 7\Revised 11-2006\[Table 7-x revised.xls]Table 7-4

730 feet USGS. For the Sister Bay Water Utility to serve areas with elevations greater than 730 feet USGS, additional hydraulically separate higher level pressure zones will need to be created.

The Sister Bay water system computer model created for this study was expanded to simulate Year 2025 conditions, including transmission main extensions and 2025 water demand equivalent to full development of the planning area. Full development was assumed to occur using one level greater development density as indicated on the Study Area Land Use Map previously illustrated in Figure 2-1.

7.2.1 Future System Pressures

Figure 7-4 illustrated computer simulated 2025 water system peak hour pressures throughout the planning area. As indicated in the figure, the majority of the future service area can be adequately served by the combined pressure zone water system. Only one major future service areas could not be served adequately with a minimum pressure of 35 psi under all normal operating conditions. This area includes higher elevation land south and west of Country Lane in the far southwestern corner of the planning area. A higher pressure plane is needed in this area to ensure that all customers can be provided with a minimum water pressure of 35 psi under all normal operating conditions as required by Wisconsin Administrative Code, Chapter NR 811.

7.2.2 Future System Fire Flows

Figure 7-5 illustrated computer simulated 2025 water system available fire flows throughout the planning area. Available fire flows were modeled assuming a minimum system residual pressure of 20 psi. Future transmission main extensions were included in the Year 2025 system model. Significant transmission main improvements were assumed to serve LGSD No. 1 and the northern planning area.

As indicated in Figure 7-5, and very similar to the modeling results illustrated in Figure 7-4, the majority of the future service area can be adequately served by the recommended combined pressure zone water system. The same future service area that cannot be adequately served with pressure cannot be adequately served with the minimum recommended fire flows under a Year 2025 maximum day demand condition.

To adequately serve these areas in the future with the minimum recommended fire flows, a higher pressure plane is needed and adequate booster pumping capacity required.

7.2.3 Outlying Future Service Area Recommendations

For the outlying future service area that cannot be adequately served by the recommended combined pressure zone system, it is recommended that this area be served by a small, higher level pressure zone, supplied by a booster pump station. Because of the relatively small size of these needed pressure zone, it is recommended the zone’s pressure plane be established by continuously operating variable speed booster pumps, with fire flows provided by a large capacity booster pump(s). Construction of storage facilities to serve this pressure zone is not recommended.

Service to the small area along STH 42 and north of Seaquist Road is not cost effective or recommended at this time.

Figure 7-7 illustrates a schematic of the Sister Bay water system Year 2025 that includes the recommended facilities to meet the needs of the planning area over the 20 year planning period.

7.3 DISTRIBUTION SYSTEM IMPROVEMENTS

Distribution system improvements have been recommended to strengthen the existing system, enhance supply reliability, loop water mains, and improve flow capacity and fire protection to various parts of the existing Village area.

There are several areas where the Sister Bay distribution system cannot supply the higher needed fire flows and where distribution system improvements that loop existing dead end water mains are recommended. Figure 7-8 illustrates recommended improvements to the existing water distribution system. The estimated costs of the water main segments are summarized in Table 7-5.

The existing PRV station on west STH 42 is not operational, and negates the operational benefits of the PRV station in the north Meadow Lane area. As pressures in the far western portion of the existing Village service area are not substandard (similar to the exiting Main Zone areas under a combined pressure zone system), it is recommended that the Village abandon the STH 42 PRV station, and decommission the Meadow Lane PRV station.

7.4 DISTRIBUTION SYSTEM EXPANSION

As the water system begins to expand to serve the Sister Bay future service planning area, it will be necessary to further extend the water transmission main system to adequately accommodate these new service areas.

Figure 7-8 illustrates recommended improvements to serve the future service planning service area. All major transmission mains identified in Figure 7-8 have been sized to meet projected future water system demands, and support system supply sources and storage facilities to serve outlying area land uses. Mains were sized to provide at least 2,000 gpm of flow capacity in commercial and high density residential areas and 1,000 gpm in medium and low density residential development areas at a residual pressure of 20 psi.

The mains shown in Figure 7-8 are only the recommended transmission mains. Smaller local service mains have not been shown. The transmission mains shown follow known or presumed locations for major streets or roads in the future service planning area, and have been located on a conceptual basis to run parallel to recommended trunk sanitary sewers (where feasible). Adjustments in the actual location of these mains can be expected at the time the mains or sanitary sewers are required, or as local needs dictate.

Water mains to serve developing residential land should be sized at a minimum of 8 inches in diameter. These mains should provide a minimum of 1,000 gpm at a 20 psi residual pressure in single-family areas. Fire flows of 2,000 gpm should be used as the criterion for all high density residential and commercial developments. All water mains to serve new developments should be looped; the Village should not allow dead end mains to be constructed.

The recommended improvement plan illustrated in Figure 7-8 to serve the future service area has been developed as a tool to guide the Village of Sister Bay in the siting and sizing of future system improvements. While the plan may represent the current planned expansion of the Sister Bay water system, future changes in land use, water demands, or customer characteristics could substantially alter the implementation of the plan. For this reason, it is recommended that the plan be periodically reviewed

TABLE 7-5

RECOMMENDED DISTRIBUTION SYSTEM IMPROVEMENTS
2006-2010
SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

Distribution Improvement	**General Location**	**Diameter** (inches)	**Approximate Length** (feet)	**Budget Cost Estimate**
Segment A	Sunny Court	8	400	$ 64,000
Segment B	N. Highland Road	8	250	$ 40,000
Segment C	Sister Bluff Drive	8	550	$ 88,000
Segment D	Country Walk Lane	8	200	$ 32,000
Segment E	STH 57 (near Smith Drive)	8	150	$ 24,000
Segment F	East of Smith Drive (north)	8	450	$ 72,000
Segment G	East of Smith Drive (south)	8	350	$ 56,000
Segment H	Little Sister Rd Loop	8	2,400	$ 384,000
TOTAL			**4,750**	**$ 760,000**

Notes

1. Recommended improvement locations shown in Figure 7-8.
2. Extensive rock excavation assumed.

C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\Chapter 7\Revised 11-2006\[Table 7-x revised.xls]Table 7-5 and updated using Village planning information to reflect the most current projections of Sister Bay area growth and development.

The improvement plan is a guidance document that details existing conditions and recommendations for the future. The plan is based on future conditions as perceived in 2006. As time progresses, additional information will become available and events will shape the development of the Sister Bay area. The plan must be dynamic in response; it should be studied and used but also adjusted to conform to the changes and knowledge that will come with time. Updates should be made on a regular basis. Due to the rapid rate of growth and development expected it the planning area, it is recommended that the water system master plan should be reviewed and updated (as necessary) every five years.

7.5 RECOMMENDED WATER SYSTEM CAPITAL IMPROVEMENTS

7.5.1 Estimated Cost of Water Main Improvements to Address Existing Deficiencies

The improvements to address existing deficiencies are shown in Figure 7-8. These improvements address dead end water mains in low available fire flow areas. The estimated costs to address existing deficiencies were presented in Table 7-5. These are preliminary budget estimates only, and actual costs should be determined through the competitive bidding process. The costs include anticipated contingencies and indirect project costs.

7.5.2 Estimated Cost of Supply and Transmission Main Facilities to Serve Future Growth

Preliminary cost estimates for the proposed supply and transmission main facility improvements to serve expansion areas in 2006 dollars are presented in Table 7-6. These estimates include allowances for surface restoration, construction contingencies, and indirect project costs such as engineering, finance, legal and administrative.

The linear foot costs used for the estimates may vary depending on the year the improvements are constructed. The unit costs used are based on recent projects, and make assumptions for extensive rock excavation and dewatering during construction. Actual costs may vary significantly depending upon actual conditions within the different improvement areas.

Due to the exact location of the transmission mains being unknown at this time, costs are considered preliminary. Extraordinary costs such as subsurface crossings, removal and replacement of other existing utilities, easement costs, etc., are not included in the preliminary estimates.

Table 7-6 lists the costs for the transmission mains that are needed to provide water service within the study planning area. As this study's recommendations are conceptual in nature, detailed feasibility reports and cost estimates should be prepared prior to the design and construction of any improvements.

7.5.3 Schedule of Improvements

The timing of future transmission main improvements will be influenced by a number of parameters. Items such as the location of development pressure in specific areas, aging facilities and/or facilities which are undersized, availability of funds, etc., all play a role in the timing of future transmission main sewer improvements.

TABLE 7-6

RECOMMENDED DISTRIBUTION SYSTEM EXPANSION
2007-2025
SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

Distribution System Expansion Segment *(as shown in Figure 7-8)*	**General Village Location**	**Diameter** (inches)	**Approximate Length** (feet)	**With Sanitary Sewer** (feet)	**Without Sanitary Sewer** (feet)	**Budget Cost Estimate**
Pipe Segment W100	Southwest	12	1,600	1,600	0	$ 198,400
Pipe Segment W101	Southwest	8	1,600	800	800	$ 239,200
Pipe Segment W102	Southwest	12	1,700	1,700	0	$ 210,800
Pipe Segment W103	Southwest	12	4,200	3,200	1,000	$ 585,800
Pipe Segment W104	South	12	3,600	3,600	0	$ 446,400
Pipe Segment W105	South	8	1,900	1,900	0	$ 222,300
Pipe Segment W106	South	12	2,200	1,000	1,200	$ 350,800
Pipe Segment W107	South	12	3,200	3,200	0	$ 396,800
Pipe Segment W108	Southeast	12	5,400	3,725	1,675	$ 778,500
Pipe Segment W109*	Southeast	12	1,300	0	1,300	$ 245,700
Pipe Segment W110	Southeast	12	1,000	1,000	0	$ 124,000
Pipe Segment W111	East	12	2,500	2,500	0	$ 310,000
Pipe Segment W112	East	12	1,310	1,310	0	$ 162,400
Pipe Segment W113	East	12	2,600	1,300	1,300	$ 406,900
Pipe Segment W114	East	12	1,300	900	400	$ 187,200
Pipe Segment W115	East	12	925	925	0	$ 114,700
Pipe Segment W116	East	12	1,000	1,000	0	$ 124,000
Pipe Segment W117	East	12	1,600	1,100	500	$ 230,900
Pipe Segment W118	East	12	3,600	1,700	1,900	$ 569,900
Pipe Segment W119	East	12	3,000	2,800	200	$ 385,000
Pipe Segment W120*	East	12	4,800	0	4,800	$ 907,200
Pipe Segment W121	Northeast	12	950	200	750	$ 166,600
Pipe Segment W122	Northeast	12	3,800	2,700	1,100	$ 542,700
Pipe Segment W123	North	12	1,450	1,200	250	$ 196,100
Pipe Segment W124	North	12	2,000	1,400	600	$ 287,000
Pipe Segment W125	Northeast	12	3,800	3,700	100	$ 477,700
Pipe Segment W126	North	12	2,760	2,560	200	$ 355,200
Pipe Segment W127	Northeast	12	2,600	1,490	1,110	$ 394,600
Pipe Segment W128	North	12	1,300	1,300	0	$ 161,200
Pipe Segment W129	North	12	5,300	5,030	270	$ 674,800
Pipe Segment W130	North	8	3,190	2,815	375	$ 397,600
Pipe Segment HL100	New HLPZ	12	1,800	500	1,300	$ 307,700
Pipe Segment HL101	New HLPZ	12	3,400	1,400	2,000	$ 551,600
Pipe Segment HL102	New HLPZ	12	2,200	1,425	775	$ 323,200
TOTAL			**84,885**	**60,980**	**23,905**	**$ 12,033,000**

Note: Extensive rock excavation assumed.
*Water main not installed in a common trench with sanitary sewer

P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay study\March-April 2008 Report Revisions\Chapters 1-14\Chapter 7\Tables & Figures\[Table 7-6 NEW revised.xls]Table 7-6

Because of the factors involved, it is difficult to accurately predict the timing of future improvements, especially those which may occur far into the future. However, some areas of the Village are more likely to experience rapid development than others.

Based on input from Village staff, a recommended Capital Improvement Plan (CIP) for water system improvements has been developed. The CIP is broken down into short-term and long-term improvements. Short term improvements generally include improvements that are needed to address existing deficiencies. Short term improvements can also include improvements to accommodate future development in areas where development is relatively cost effective, such as areas that do not need to be served by a new high level pressure zone. Long term improvements typically include providing service to future expansion areas that are located farther from the existing system and are more expensive to construct. The CIP for short term improvements and long term improvements is presented in Table 7-7.

7.5.4 Financing of Water System Improvements

Expanding the existing water system to accommodate future development can include construction of transmission mains, and implementing new pressure zone booster stations in areas where adequate service cannot be provided by the recommended combined pressure zone system.

It is anticipated that these improvements will either be financed by a developer, assessed to benefiting properties, paid for by the Utility, or a combination thereof. Typically, construction of future transmission main improvements will be constructed and paid for in conjunction with a development project. In some communities, the costs of transmission main extensions are the sole responsibility of the developer. In other communities, the developer has the option of allowing the transmission improvements to be constructed by the Utility with all associated costs being assessed back to the benefiting properties.

Construction of future booster stations and implementation of new pressure zones can also be treated in a similar way to transmission main extensions. If the pressure zone and booster pump station are necessary only to serve new development, the entire cost of these facilities can be passed back to the identified new development. If development is staged, it may be possible to stage the improvements to track with the development. When staging improvements is not possible, over-sizing costs can be recovered through special assessments, transmission area charges, or other means. As a last resort, over-sizing costs may need to be carried by the Utility until future development occurs within the larger service area, at which time the costs can be recovered from the development through one of the methods described above.

7.5.5 Short-Term System Improvement Impacts on Utility Revenue Requirements

Table 7-8 summarizes the results of a preliminary analysis of the probable impact on Water Utility revenue requirements (rates) of implementing the recommended short-term capital improvements. The effect of the short-term improvements with respect to each revenue requirement category has been estimated.

Table 7-9 summarizes projected increases in the Water Utility's cost of service with the implementation of the proposed short-term improvements. It is projected that the improvements will cause the Utility's revenue requirements to increase by $206,100 to approximately $578,500. This represents a 143 percent increase from the Utility's 2005 operating revenues. The actual impact on water rates would need to be determined based on the Utility's revenues in the year the improvements were constructed. Water sales are projected to increase approximately 2 percent per year during the planning period.

TABLE 7-7

WATER SYSTEM CAPITAL IMPROVEMENT PLAN
SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

SHORT TERM IMPROVEMENTS

Type of Improvement	System Location	Recommended Improvement	Planning Level Costs
Water Storage	Village Wastewater Treatment Plant	Construct New Water Tower	$840,000
Distribution System	South Central	Eliminate Dead End Water Mains in Low Fire Flow Areas	$760,000
Combine Pressure Zones	All	Modify Well 1 Pump to operate in Combine Zone System	$55,000
		Total	**$1,655,000**

LONG TERM IMPROVEMENTS

Type of Improvement	System Location	Recommended Improvement	Planning Level Costs
Water Supply or Storage	West Village area (supply) or adjacent to Standpipe (storage)	Construct water supply Well 4 or a new 100,000 gallon ground reservoir	$800,000
Distribution System Expansion	Planning Area	Construct Transmission Main Improvements to Support Growth and Development within Planning Area	$12,033,000
Implement New Southwest High Level Pressure Zone	Southwest	Construct Booster Pumping Station to Serve New Pressure Zone	$450,000
		Total	**$13,283,000**

P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay study\March-April 2008 Report Revisions\Chapters 1-14\Chapter 7\Tables & Figures\[Chpt 7 Tables 1-7.xls]Table 7-7

TABLE 7-8

CAPITAL IMPROVEMENTS IMPACT ON UTILITY REVENUE REQUIREMENTS

SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

		Estimated Increase in Utility Revenue Requirements				
Recommended Water System Capital Improvement	**Budget Cost Estimate**	Operation & Maintenance Expenses	Depreciation Expense	Tax Equivalent	Return on Rate Base*	**Total**
New Water Tower	$895,000	$22,400	$17,900	$7,000	$76,000	**$123,300**
Water Main Improvements	$760,000	$3,800	$8,000	$6,000	$65,000	**$82,800**
Total	**$1,655,000**	**$26,200**	**$25,900**	**$13,000**	**$141,000**	**$206,100**

* 8.5% return on rate base assumed

TABLE 7-9

PRELIMINARY COST OF SERVICE ANALYSIS

SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

2005 Utility Revenue Requirements:			
	Operation & Maintenance Expenses	$197,919	
	Depreciation Expense	$51,586	
	Tax Equivalent	$28,358	
	Authorized Return on Rate Base	$94,498	
	Total		**$372,361**
Plus:	Increased Revenue Requirements due to Proposed Improvements		**$206,100**
Total Projected Revenue Requirements			**$578,461**
2005 Utility Operating Revenue			$238,362
Net Service Cost Increase			$340,099
Net Service Cost Increase as a Percentage of 2005 Utility Revenue			***143%***

7.5.6 Water System Ordinance Review

As part of the analysis of future improvements, a review of the Village's existing water system regulations and ordinances was conducted. The purpose of this review was to identify any changes that could be made to the ordinances that would allow the Village to better implement the water system recommendations contained in this report.

Several documents were reviewed, including the Municipal Code of the Village of Sister Bay, and the Engineering Design Manual.

7.5.6.1 Municipal Code

The Municipal Code contains the essential rules and regulations pertaining to governance of the Village. The water system is discussed primarily in Chapter 54 (Land Division and Platting Code) and Chapter 62 (Utilities).

Chapter 54 of the Municipal Code contains Section 54.106, Water Supply Facilities. This section covers the design, installation and cost recovery aspects of constructing water distribution system facilities in conjunction with development. Section 54.106 addresses these areas quite thoroughly, and only a few suggested additions are recommended:

1. Reference this Comprehensive Utilities Plan and its role in the development review process in Section 54.106. Although this plan is conceptual and schematic in nature, this plan should be an important tool for the Village in the development review process. The plan is intended to be used as a guide, for both developers and the Village, of an efficient and economic way to construct the future distribution system. Concept plans submitted by developers should be consistent with the "spirit" of the plan, whenever possible. This may not be possible in some cases, due to unique conditions and constraints that are not known at this time. However, where it is not possible to follow the concepts identified in the plan, the developer should document why the proposed deviation would be in the best interests of the Village. Similar to the Engineering Design Manual, this plan should be kept on file at the Village Administration Building, and should be open to inspection by the public during normal office hours.

2. Section 54.105 (k) (3) states that the Village will pay some portion of the oversizing costs for water main pipes that need to be oversized to accommodate future development. Paying for and carrying this oversizing cost will be one of the major challenges that the Village will face in the future.

Chapter 62 of the Municipal Code is entitled "Utilities", and it provides rules for the Village water and sewer system, abandonment of private wells and cross connection control. Chapter 62 is quite comprehensive and thorough; no changes are needed. Section 62-7 (e) (2) identifies accommodating property owners in routing of water mains, and suggests looping of water mains "whenever possible". The Village should only approve dead-end water mains in very special circumstances – dead end water mains compromise public fire protection and distribution system water quality, and should not be allowed to be constructed unless there is a specific plan to loop the water main in the near-term future.

7.5.6.2 Engineering Design Manual

The Engineering Design Manual contains Chapter 7 that deals with water distribution system issues. The purpose of this chapter is to provide guidance to the designer regarding the Village's requirements for design of water system facilities. Comments on this document are listed below:

1. The required fire flow listed in Section C.5 is less than the flows used in this planning study. It is recommended that the required flows used in the Village be consistent with this report (1,000 gpm in low density residential areas, 2,000 gpm in higher density residential, commercial, industrial or public areas).

2. Required easement widths for water mains are listed in Section C.12 as 25 feet, whereas required easement widths in Village Ordinance Sections 54.105 and 62-7 are listed as 30 feet. The 30 foot dimension is recommended.

CHAPTER 8

EXISTING SANITARY SEWER SYSTEM FACILITIES

The sanitary sewer collection system and lift station facilities operated and maintained by the Village of Sister Bay include:

1. Four large sanitary sewer lift stations
2. Four small sanitary sewer grinder stations
3. Force mains associated with lift stations
4. A network of gravity sewer piping and manholes

The general location and layout of the sanitary sewer system facilities is illustrated in Figure 8-1. A schematic of the sewer system is illustrated in Figure 8-2. This chapter presents a summary of the design and operating characteristics of the existing sanitary sewer system and components.

8.1 DESCRIPTION OF SYSTEM

The Sister Bay sanitary sewer collection system was originally constructed in 1972, and is a combination of gravity sewers, lift stations, and force mains. Wastewater is collected in the system and conveyed through piping to the Main Lift Station No. 1 (LS 1), where the flow is pumped to the Wastewater Treatment Plant (WWTP). After the wastewater is treated, it is discharged through an outfall sewer into Sister Bay. The existing service area of the collection system is approximately 1,200 acres in size, and serves approximately 930 connections/customers. Those residents who are not connected to the system are served by private sewage disposal systems or holding tanks.

The Liberty Grove Sanitary District No. 1 is connected to the Village of Sister Bay collection system. Areas on the south side of the Village flow north and east to LS 1, and areas on the north side of the Village and Sanitary District flow south and west to LS 1. The WWTP is located east of Woodcrest Road and south of Scandia Road. LS 1 is located east of Bay Shore Drive and south of Scandia Road.

The gravity collection system piping ranges in size from 6 inch to 12 inch diameter pipe. Some 14 inch pipe also exists on the outfall line between the WWTP and Green Bay. The normal pipe size for a development is typically 8 inch diameter sewer pipe. Larger pipes (10 inch and 12 inch diameter), referred to as collectors, connect different areas of the community and convey the flows downstream. Service laterals which serve individual buildings are typically 4 inch and 6 inch diameter pipes. The system's gravity, force main and outfall sewers are summarized by size in Tables 8-1 and 8-2.

There are four large lift stations and four small lift stations (grinder stations) in the system. The large lift stations include LS 1, Little Sister Lift Station, Fieldcrest Lift Station, and Waters End Lift Station (located within the Sanitary District). The smaller grinder stations (GS) include Forest Lane GS, Sunny Court GS, Crows Nest GS, and Pheasant Court GS. Force mains connect each of the lift stations to the gravity collection system. The force mains range in size from 2 inch diameter for the grinder stations up to 12 inch diameter for LS 1. LS 1 has an 8 inch force main and a 12 inch force main.

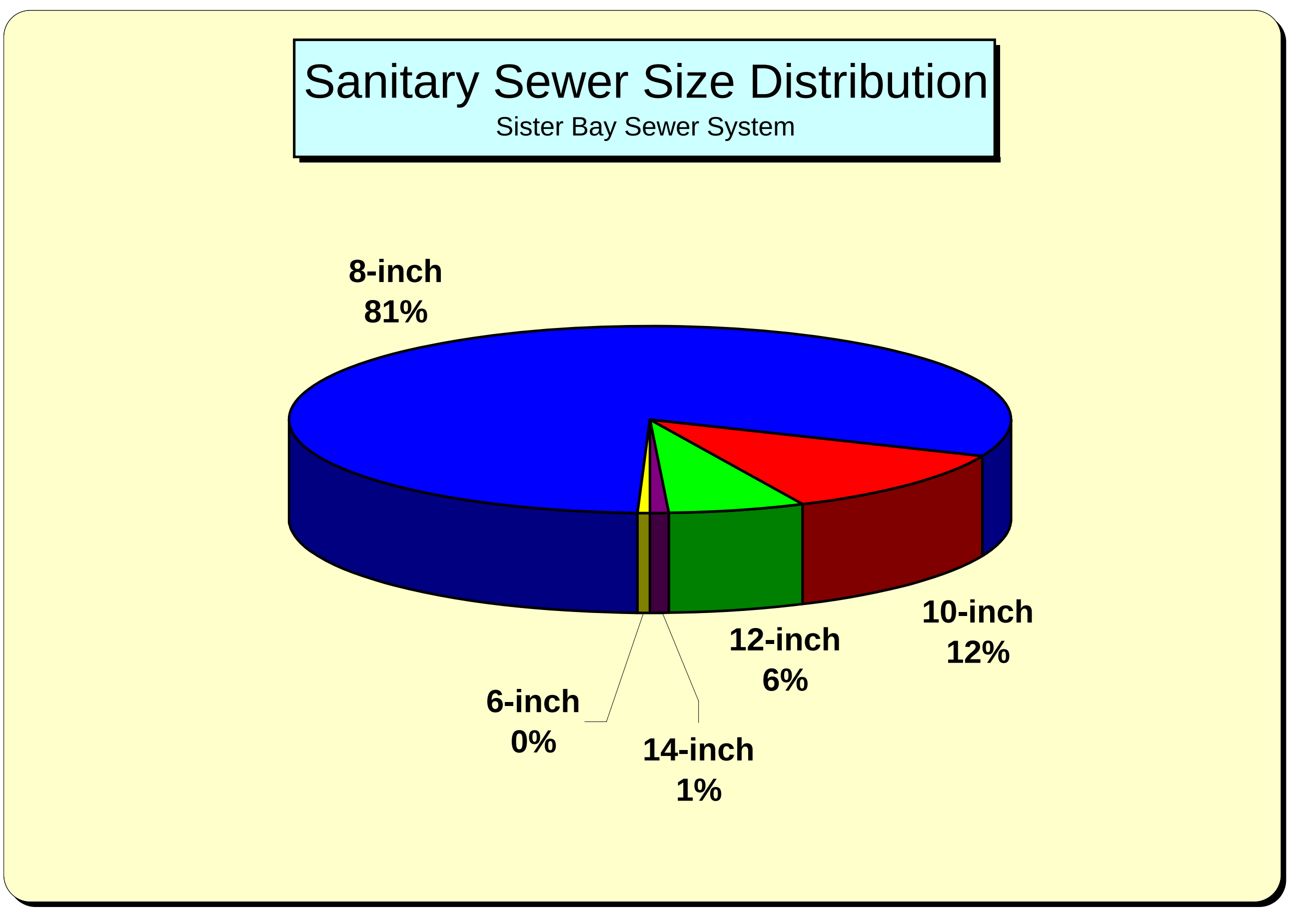
To confirm existing pumping capabilities of each lift station, Village staff conducted test pumping during the fall of 2005. The test pumping process involved manually controlling pump operation and measuring the time duration to pump a specific volume of wastewater. Adjustments were made in the calculations to

TABLE 8-1

SANITARY SEWER SIZE DISTRIBUTION
SISTER BAY SANITARY SEWER SYSTEM
VILLAGE OF SISTER BAY, WISCONSIN

Diameter (inches)	Approximate Total Length[1] (feet)	Percentage of Total
6	364	0.4%
8	65,715	81.0%
10	9,473	11.7%
12	4,956	6.1%
14	590	0.7%
Total	**81,099**	**100.0%**

[1] Source: 2005 Sister Bay sanitary sewer system computer model.



X:\S\SISTB\050200_UTILITIES\Project\Sister Bay copy\Report\Chapter 1-5 8 11\[table8-x.xls]Table 8-1

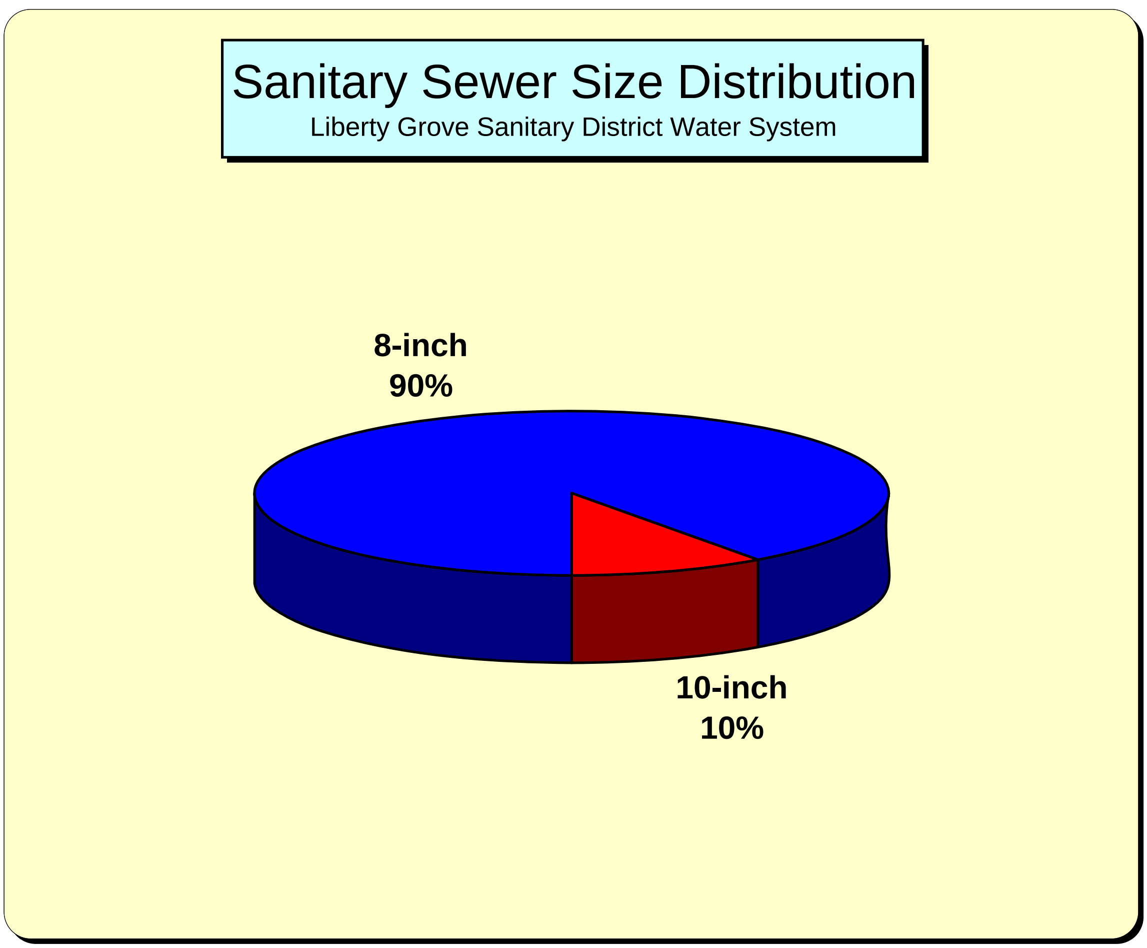
TABLE 8-2

SANITARY SEWER SIZE DISTRIBUTION

LIBERTY GROVE SANITARY DISTRICT NO. 1
TOWN OF LIBERTY GROVE, WISCONSIN

Diameter (inches)	Approximate Total Length[1] (feet)	Percentage of Total
8	19,525	89.9%
10	2,193	10.1%
Total	**21,718**	**100.0%**

[1] Source: 2005 Sister Bay sanitary sewer system computer model.



C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[table8-x.xls]Table 8-2

account for any influent discharging into the lift station wetwell during the pump test. The test pump results and other lift station data is summarized in Tables 8-3 and 8-4.

The Village's existing sanitary sewer base map denotes manhole I.D. numbers and sewer pipe sizes, but contains no information on pipe length, invert or rim elevations. To populate the sanitary sewer computer model of the Sister Bay collection system with the necessary information, record drawings of the system were reviewed and the needed information was tabulated. The system information collected was stored in geographic information system (GIS) format using DataView™ software developed by SEH. The DataView software provided a convenient way to collect and store system data in a format that allowed easy retrieval and manipulation.

8.2 CONDITION OF SYSTEM

The original Sister Bay collection system was installed in 1972, and has continued to expand on a regular basis since that time. Most of the sanitary sewers that were installed in the early 1970s were constructed of PVC pipe with glued joints. The larger sewer pipe (10 inch diameter and above) was either RCP or asbestos-cement pipe. By the late 1970s, sewer pipe installed in Sister Bay was constructed of PVC pipe utilizing a gasketed joint system. Sewer manholes are constructed of precast reinforced concrete.

The Village flushes sanitary sewer pipes each year in the spring and fall. Most areas of the system have not been televised recently, so the current condition of the pipe interior is unknown. From the available information that has been reviewed for this study, the general condition of the existing collection system is fair, although several system problems have been identified.

Infiltration and inflow (I/I) is present in the spring due to the elevated water table, snow melt and rainfall events. The WWTP experiences a significant spike in flows during spring months. While it is not readily apparent where the clear water is entering the collection system, it is likely that there are multiple sources. The State Plumbing Code calls for all homes to have floor drains adjacent to water heaters for drainage purposes. In most cases, these floor drains are connected to the sanitary sewer system. In areas of high groundwater, it is possible that building foundation drains and sump pumps are also connected to this floor drain system, and therefore are contributing clear water to the sanitary sewer system.

The area near the current intersection of State Highways 42 and 57 was originally a ravine that was filled prior to development. Some sanitary sewers in this area have reportedly settled, creating flat or negative slopes, and resulting in sewer flow capacity restrictions. The sanitary sewers have been televised in the past, and standing water and a buildup of solids was observed in the sewer pipe. An example of this condition exists between MH 47 and MH 45, where the 10 inch pipe has a visible dip between manholes.

According to Village maintenance staff, there are several areas of the sanitary sewer collection system that are known to have relatively flat slopes. These locations include:

- Woodcrest Road and Scandia Road area
- Areas north of Bay Shore Drive and Sister Bluff Drive

Areas north of Bay Shore Drive and Sister Bluff Drive (Sister Bluff Estates) have manholes with sumps. The manhole pipe inverts constructed in this area are the catch basin type that result in solids deposition and which require frequent cleaning. North Spring Road and Pheasant Court also contain these types of manholes.

TABLE 8-3

EXISTING LIFT STATION DATA

SISTER BAY SANITARY SEWER SYSTEM
VILLAGE OF SISTER BAY, WISCONSIN

	Lift Stations								
Pump Data	**Lift Station No. 1 (Main Lift Station)**			**Fieldcrest Lift Station**		**Little Sister Lift Station**		**LGSD No. 1 Waters End Lift Station**	
Type	Wetwell / Drywell			Wetwell / Drywell		Wetwell / Drywell		Submersible	
Pump Manufacturer	Gorman Rupp			Smith & Loveless		Smith & Loveless		Barnes	
Year Installed/Remodeled	1989			1990		1987		1997	
Contractor	Crane Engineering			Energenecs		Energenecs		Energenecs	
Pump Number	1	2	3	1	2	1	2	1	2
Model No.	T6A3	T8A3	T8A3	4B2B	4B2B	4B3B	4B3B	4SE	4SE
Force Main Size (inches)	8 & 12			4		6		4	
Rated Pump Capacity (gpm)	300	1000	1000	100	100	300	300	100	100
Total Dynamic Head (feet)	80	n/a	n/a	48	48	135	135	40	40
Test Pump Capacity (gpm)	216	526	435	94	94	273	238	74	53
Test Pump Capacity - Both pumps (gpm)		812		124		335		83	
Motor Data									
Horsepower	30	60	60	5	5	20	20	5	5
RPM	1765	1775	1775	1170	1170	1760	1760	1750	1750
Voltage / Phase	460/3			208/3		480/3		230/3	

X:\S\SISTB\050200_UTILITIES\Project\Sister Bay copy\Report\Chapter 1-5 8 11\[table8-x.xls]Table 8-3

TABLE 8-4

EXISTING GRINDER LIFT STATION DATA

SISTER BAY SANITARY SEWER SYSTEM

VILLAGE OF SISTER BAY, WISCONSIN

	Grinder Lift Stations							
Pump Data	**Forest Lane Grinder Station**		**Sunny Court Grinder Station**		**Crows Nest Grinder Station**		**Pheasant Court Grinder Station**	
Type	Grinder		Grinder		Grinder		Grinder	
Pump Manufacturer	Barnes		Barnes		Barnes		Barnes	
Year Installed/Remodeled	2004		2004		2004		2004	
Contractor	Energenecs		Energenecs		Energenecs		Energenecs	
Pump Number	1	2	1	2	1	2	1	2
Model No.	SGV5002L	SGV5002L	XSGV	XSGV	XSGV	XSGV	XSGV	XSGV
Force Main Size (inches)	2		2		2		2	
Rated Pump Capacity (gpm)	50	50	45	45	40	40	40	40
Total Dynamic Head (feet)	125	125	n/a	n/a	n/a	n/a	n/a	n/a
Test Pump Capacity (gpm)	51	56	43	45	40	43	41	58
Test Pump Capacity - Both pumps (gpm)	77		62		47		85	
Motor Data								
Horsepower	5	5	2	2	2	2	n/a	n/a
RPM	3450	3450	3450	3450	3450	3450	3450	3450
Voltage / Phase	240/1		240/1		240/1		240/1	

X:\S\SISTB\050200_UTILITIES\Project\Sister Bay copy\Report\Chapter 1-5 8 11\[table8-x.xls]Table 8-4

The Utility maintenance crew follows a regularly scheduled computerized plan for maintenance of the lift stations. This plan includes monthly flushing and adding degreaser to all stations. The maintenance plan also includes:

- Servicing electrodes every two months
- Servicing air release valves every three months
- Changing bearing/seal cavity oil every six months
- Checking and adjusting clearances every six months
- Greasing motors every two years
- Grinder station pumps are serviced on an annual basis (using a lift station service firm)

8.3 EXISTING SYSTEM FLOWS

Historical system flow data was collected and reviewed for this study. Average monthly flow recorded at the WWTP was approximately 170,000 gallons per day (gpd) during 2005. The flows have steadily increased over the past 5 years, and have tended to be higher during the summer tourist season. The data also shows significant flow spikes during rainfall or snowmelt events, which is an indication that some level of I/I is present in the collection system during certain periods of the year.

Lowest flows typically occur during the winter months of December, January and February. During 2005, the lowest monthly flow occurred in the month of January with an average flow of 90,000 gpd. The highest 2005 flow reported occurred in the month of July with an average flow of 280,000 gpd.

Daily flows during the maximum month, and hourly flows during the maximum day were also reviewed for 2005. A summary of 2005 flows recorded at the WWTP is summarized in Table 8-5.

A list of the top sanitary sewer flow producers in the community was not available. However, a list of the top water users was reviewed, and was previously summarized in Chapter 4 (Table 4-5). It is common that the largest water users in a community system are also the largest wastewater flow contributors.

TABLE 8-5

SUMMARY OF 2005 FLOWS AT WWTP

SISTER BAY SANITARY SEWER SYSTEM
VILLAGE OF SISTER BAY, WISCONSIN

Month	Total for Month (MG)	Average Day (MG)	Minimum Day (MG)	Maximum Day (MG)
January	2.784	0.090	0.072	0.152
February	2.514	0.090	0.072	0.118
March	2.840	0.092	0.072	0.132
April	3.901	0.130	0.104	0.183
May	5.310	0.171	0.115	0.269
June	6.471	0.216	0.171	0.261
July	8.683	0.280	0.251	0.344
August	8.255	0.266	0.205	0.311
September	5.876	0.196	0.166	0.298
October	6.589	0.213	0.138	0.323
November	4.082	0.136	0.112	0.175
December	3.727	0.120	0.089	0.189

MG: Million Gallons

P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay study\Report Final Draft\Chapter 8\Tables and Figures\[table8-5.xls]Sheet1

CHAPTER 9

SANITARY SEWER SYSTEM EVALUATION

An important component of the Comprehensive Utilities Plan was the evaluation of the existing sanitary sewer collection system and performing a deficiency analysis. This chapter summarizes the findings from this evaluation.

9.1 SANITARY SEWER SYSTEM COMPUTER MODEL

A hydraulic computer model of the existing collection system was developed to assist with the evaluation of the existing Sister Bay sanitary sewer system. The goal of the evaluation was to determine if any current deficiencies exist in the system, not including the impact of additional growth.

9.1.1 Model Setup

The modeling software selected for this project utilized the spreadsheet capabilities of Microsoft Excel. The inventory information which had been collected and stored in DataView software was transferred into the hydraulic model and populated. Populating the Sister Bay sewer model with inventory data involved computing the pipe slope between manholes by reviewing invert and length data from DataView. Using the computed pipe slopes and pipe sizes obtained from Utility records, the flow capacity of each pipe was estimated. Estimates of existing individual sewer pipe capacities were developed to compare with estimated existing flows to determine if any deficiencies existed.

9.1.2 Model Loading

The process of loading the hydraulic model with existing flows involved several steps. First, estimated sewer flows were allocated to each manhole. This was done by reviewing aerial photography and zoning maps to estimate the size and type of existing land use tributary to each manhole. In residential areas, the number of existing units was counted. In non-residential areas, the service areas were measured in acres.

Next, average sewer flows tributary to each manhole were estimated using the unit and area flow generation rates shown in Table 9-1. Cumulative average flows at each point in the system were developed by adding the flows upstream of each manhole.

Finally, a peak flow factor was applied to the cumulative average flows at each manhole. The resulting estimated cumulative peak flow at each point in the system theoretically represented the maximum flow that the piping system would experience under current development levels. The peak flow factor was based on the peaking ratios observed at the WWTP and also on the typical peaking relationships which are sited in literature (NR 110 and Recommended Standards for Wastewater Facilities, 2004 Edition).

9.1.3 Model Calibration

Calibration of the existing model was performed by reviewing Sister Bay WWTP flow records. The concept of calibration involves matching the model flows with measured flows so that the model is a reasonably accurate simulation of peak flow conditions. A number of assumptions were built into the model. These assumptions affected the magnitude of the peak flow predicted by the model. These assumptions and the associated peak flows predicted by the model were adjusted through the use of a

TABLE 9-1

UNIT AND AREA FLOW GENERATION RATES FOR EXISTING CONDITIONS
SISTER BAY SANITARY SEWER SYSTEM
VILLAGE OF SISTER BAY, WISCONSIN

		Current Zoning Densities		Parameters Used in Existing Model			
Land Use Type	**Zoning Catagories**	**Lot Size**	**Dwelling Units per Lot**	**Maximum Density in Units/Acre**	**Assumed Density in People/Unit**	**Assumed Flow Rate in Gallons per Capita per Day (GPCD)**	**Existing Flow Rate in Gallons per Acre per Day (GAD)**
Residential	(CS-1) Countryside	10 acre	1	0.10	3.00	110	33
	(R-1) Single-Family Residence	20,000 sq. ft.	1	2.18	2.50	100	545
	(R-2) Multiple-Family Residence	20,000 sq. ft.	6/acre	6.00	2.25	90	1,215
	(R-3) Large Lot Residence	5 acre	1	0.20	3.00	110	66
	(R-4) Small Lot Residence	4,500 sq. ft.	1	9.68	2.00	85	1,646
Non-Residential	(B-1) General Business	20,000/25,000 sq. ft.		2.18			1,200
	(B-2) Downtown Business Transition	4,500 sq. ft.		9.68			1,646
	(B-3) Downtown Business	4,500 sq. ft.		9.68			1,500
	(I-1) Institutional						1,500
	(P-1) Park/Recreation						60
	Liberty Grove Industrial	60,000 sq. ft.		0.73			1,500
	Liberty Grove Natural Area	15 acre	1	0.07			10

X:\S\SISTB\050200_UTILITIES\Project\Sister Bay copy\Report\Chapter 9\[Table 9-x.xls]Table 9-1 calibration factor that was applied to the flows. In this case, complete lift station pumping records and flow metering data was not available for the system. Therefore, the WWTP flow data was used as a calibration point for the existing Sister Bay model.

In reviewing the historical flow records at the WWTP, it was observed that the peak flow in a year with normal rainfall typically ranges from 300,000 to 350,000 gallons per day (gpd), and occurs in June or July. However, there have been isolated cases where flows have spiked significantly. For example, the largest event recorded in recent years was a snow melt/rainfall event which occurred in late March and early April 2004. The largest daily flow during that event was on March 29, 2004, where a volume of approximately 900,000 gallons was received at the plant. This flow volume nearly reached the plant's total treatment capacity of 945,000 gpd.

Since the maximum flow event of 900,000 gpd at the WWTP occurred quite recently (March 2004), it is a good estimate of the potential peak that could occur today. Accordingly, the computer model was calibrated to this peak event, which simulates a maximum peak flow condition with existing development.

In addition to the WWTP calibration point, a second field measurement was used to check model calibration. On May 12, 2006, a flow monitor was inserted into the 10 inch pipe between MH 17 and MH 15. A capacity test was conducted by manually starting the four upstream lift stations such that the surge in flow from each of the stations reached the flow monitor at approximately the same time. The model was modified to simulate the same conditions, and the measured flow at the monitor was compared to the flow predicted from the computer model. The resulting close comparison between the measured flow and modeled flow at this location provided further support that the assumptions used in the model were reasonable.

9.2 CAPACITY ANALYSIS

9.2.1 Pipe Capacity

After the existing model was calibrated to match existing peak flows at the WWTP and the flows measured by the flow monitor, a comparison was made between the estimated flows in each pipe and the estimated pipe capacity. The result was expressed as percent utilized. For example, a pipe having a utilization of 75 percent meant that the estimated existing flow was 75 percent of the pipe capacity, and there was 25 percent of the pipe capacity remaining for future development. Pipes with a utilization of 100 percent or more indicated a deficiency in the form of a capacity restriction.

Pipes with a capacity restriction could theoretically be flowing under surcharged conditions during peak flow periods, and could cause sewer backups and overflows. This may or may not correspond to reported backups or surcharging observations. Many times, peak flows occur during unusual hours when observations cannot be made (i.e., during or immediately after rainstorms, during early morning hours, etc.). In addition, depending on the pipe depth and number of service connections, some pipes can be periodically surcharged without causing backups or any visible sign of the surcharge. Thus, some surcharges may take place without any indication.

After final calibration, all pipes in the system were estimated to be under 100 percent utilized. The segment showing the highest utilization was the 10 inch diameter pipe from MH 15 to MH 13 on Mill Road, just west of Bay Shore Drive. This segment was estimated to be between 80 and 90 percent utilized under existing conditions. In addition, pipe segments on both sides of this segment were estimated to be between 70 and 80 percent utilized.

Table 9-2 and Figure 9-1 show the peak flow level of existing sewer pipes under current conditions in percent of maximum capacity. Table 9-2 lists the pipe segments in four categories. The categories include:

- Pipes at 25 – 50 percent of capacity
- Pipes at 50 – 75 percent of capacity
- Pipes at 75 – 100 percent of capacity
- Pipes over 100 percent of capacity

All other sewer pipe segments not listed in Table 9-2 were estimated to be below 25 percent of capacity.

The pipe segment between MH 15 and MH 13 represents a bottleneck in the collection system, as does the pipe segments on either side of this section. This affects everything upstream, including areas to the south and west. Additional development upstream of this pipe segment will be restricted unless improvements are completed to free up additional capacity. In the following chapter, a proposed diversion is described that would free up approximately 150 residential equivalent units (see Sections 10.1.1 and 10.2.6.3). Without implementing this diversion, additional development upstream of the bottleneck is not recommended. With the diversion, there would be approximately 150 units of development available upstream (south and west) of the bottleneck.

9.2.2 Lift Station and Force Main Capacity

Lift stations and force mains were also checked for capacity restrictions under existing conditions. The estimated peak flow coming to each station was checked against the pumping capacity which had been recorded during the test pumping activities. The peak flows were also checked against the force main capacities. In each case, the estimated peak flows were less than the lift station and force main capacities. This indicates that the existing lift station and force main systems appear to be adequately sized for peak flows under current conditions.

TABLE 9-2

CAPACITY LEVEL OF PIPES UNDER EXISTING CONDITIONS

SISTER BAY SANITARY SEWER SYSTEM

VILLAGE OF SISTER BAY, WISCONSIN

SANITARY SEWER PIPE CAPACITY			
Pipes at 25% to 50% of Capacity	Pipes at 50% to 75% of Capacity	Pipes at 75% to 100% of Capacity	Pipes over 100% of Capacity
MH301 - MH303	MH021 - MH019	MH015 - MH013	NONE
MH251 - MH059	MH019 - MH017		
MH055 - MH581	MH017 - MH015		
MH047 - MH045	MH013 - MH011		
MH045 - MH043	MH011 - MH009		
MH043 - MH041	MH009 - MH007		
MH085 - MH083	MH007 - MH005		
MH079 - MH077	MH005 - MH003		
MH077 - MH075	MH003 - MH001		
MH073A - MH073	MH001 - MH000		
MH073 - MH025			
MH159 - MH001			
MH033 - MH031			
MH031 - MH029			
MH025 - MH023			
MH002 - MH000			

P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay study\Report Final Draft\Chapter 9\Tables and Figures\[Table 9-2.xls]Sheet1

CHAPTER 10

RECOMMENDED SANITARY SEWER SYSTEM IMPROVEMENTS

This chapter discusses recommended improvements to address existing sanitary sewer system deficiencies as well as recommended improvements to meet future planning area needs. The nature and extent of existing system deficiencies were identified and discussed in Chapters 9 and 10. The improvements to meet future needs are based on the expansion of the existing sewer system into the undeveloped areas to facilitate growth as projected in the Village's Comprehensive Plan.

10.1 IMPROVEMENTS TO ADDRESS EXISTING DEFICIENCIES

The improvements to address existing deficiencies are shown in Figure 10-1 and summarized in Table 10-1. These improvements address three types of identified system deficiencies:

- Potential future capacity restrictions
- Pipe settlements
- Sump manholes

The remainder of Section 10.1 discusses improvements to address these existing system deficiencies.

10.1.1 Potential Future Capacity Restrictions

As shown in Figure 9-1, the sanitary sewer pipe segment from MH 015 to MH 013 is 75 to 100 percent utilized under existing development conditions. In addition, a number of pipe segments on either side of this pipe are 50 to 75 percent utilized under existing conditions. Sewer pipes that are highly utilized under present conditions have little capacity remaining to accommodate additional growth. This is particularly critical for these sewer pipe segments, because the affected sewer line is a main trunk sewer which serves the largest part of the Village service area, and could also be used to serve future expansion areas to the south. These pipes could be removed and replaced with a larger diameter size, or they could be enlarged using trenchless techniques. Either method would be quite expensive.

A more cost effective option would be to create a diversion upstream of the problem area by redirecting some of the wastewater flow around the area in question. Such a sewer flow diversion appears to be possible south of Maple Drive and west of Claflin Street. The existing sanitary sewer pipe runs from MH 39 to MH 37 along the west side of the cemetery to Maple Drive, and then turns west toward Hwy 42. Another subsystem begins east on Maple Drive at MH 193 and runs east to MH 177, and then turns north on Claflin Street. The existing manhole invert elevations would allow for a diversion to be constructed by installing a new manhole between MH 39 and MH 37, and installing a pipe from this new manhole to MH 193, thus diverting flow into sewer lines along Claflin Street, Mill Road and South Spring Road. This diversion can be seen in Figure 10-1.

10.1.2 Pipe Settlements

Inspection of television camera video footage has revealed a large settlement in the existing 10 inch sanitary sewer pipe between MH 47 and MH 45. This sewer line was constructed through an existing ravine that was subsequently filled as part of the project. This dip that was created in the sewer line causes

TABLE 10-1

IMPROVEMENTS TO ADDRESS EXISTING DEFICIENCIES
SISTER BAY SANITARY SEWER SYSTEM
VILLAGE OF SISTER BAY, WISCONSIN

System Deficiency	System Location	Description	Recommended Improvement	Alternative Improvement Option
Potential Future Capacity Restriction	MH019 to MH011	Highest Utilized Pipes in System	Construct Partial Diversion From MH039 to MH193	Enlarge pipe size from 10-inch to 12-inch using Trenchless Methods
Capacity Restriction due to Pipe Settlement	MH047 to MH045	Negative pipe slope due to settlement of sewer pipe; sedimentation of solids	Remove and replace 10-inch pipe	N/A
Sump Manholes	MH073 to MH121 MH123 to MH131 MH220 to MH228 MH198 to MH204	Sumps create maintenance problems due to sedimentation of solids	Pour concrete inverts in manholes to eliminate sumps	N/A

P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay study\Report Final Draft\Chapter 10\Old Tables and Figures\[Table 10-x.xls]Table 10-1 low flow velocities and the sedimentation of solids which results in on-going sewer maintenance problems.

This sewer pipe segment will be difficult to repair without removing and replacing the settled sections of pipe. Due to the potential for further settlement, it is recommended that the entire segment from manhole to manhole be removed and replaced. This will allow inspection of the trench bottom prior to new pipe installation, to determine if the trench bottom needs additional treatment prior to installation of a new sewer pipe.

10.1.3 Sump Manholes

Several sanitary sewer system manholes were constructed without poured inverts, and currently act as sump manholes. During periods of low flow, wastewater solids drop out of suspension into these sumps, and the sumps need to be regularly cleaned. This is an unnecessary recurring system maintenance activity that can be eliminated with proper manhole construction.

Sump manholes are found in the following locations:

- Sister Bluff Estates: MH 73 – MH 121, and MH 123 – MH 131
- North Spring Road: MH 220 – MH 228
- Pheasant Court: MH 198 – MH 204

The remedy for the sump manhole conditions is to pour concrete inverts into the manholes. The recommended improvement will require a contractor to temporarily bypass pump around the affected manholes, properly clean the sumps, pour new concrete inverts, and allow the concrete inverts to properly cure prior to removing the temporary bypass.

10.2 IMPROVEMENTS TO MEET FUTURE NEEDS

The remainder of this chapter discusses system improvements recommended to provide sewer service to the areas within the study planning boundary that are currently unsewered. The following discussion will include the methodology used, analysis conducted, results obtained, and proposed system improvement recommendations.

10.2.1 Computer Model of Future System

A hydraulic computer model of the future sanitary sewer system was developed to aid in capital improvement planning. This model was constructed similar to the existing sanitary sewer system model described in Section 9.1, but with a few differences. The primary difference between the existing system model and the future model was in the estimated future sewer flows. Instead of using aerial photography and current zoning maps to determine flows tributary to each manhole, the future model utilized the Planning Districts Map provided by the Village to estimate future flows. The Planning Districts Map is shown in Figure 2-1.

10.2.2 Flow Generation Rates

For the purposes of sizing sanitary sewer pipes to serve future development, it is prudent to be conservative when estimating future flows. Sewer pipes can typically have a useful life of 60 years or more, and there is a relatively small difference between the cost of larger pipe sizes, compared to the cost of initial pipe installation and trench restoration. Therefore, it is usually more cost-effective over the long-term to design sewer trunk facilities to be conservatively large rather than too small when planning sewer service for future long-term development.

For this reason, and based on discussions with Village staff, it was assumed that land use areas identified as “Countryside” and “Large Lot Residential” on the study Planning Districts Map may potentially be reclassified to the next higher level of development density during the life of the sewer system, and contribute higher sewer flows to the system. For example, land uses currently defined as Countryside were treated as if they were rezoned Large Lot Residential in the future system model. Similarly, Large Lot Residential land uses were treated as if they were rezoned as Single Family Residential in the future system model. All other future land uses were treated the same as they were in the existing system computer model.

Although the maximum density allowable for Single Family Residential is 1 unit per 20,000 square feet, this density was considered too high to use for estimating average sewer flows within the planning area. A number of existing rural residential developments were checked, and the average density in many of the rural settings was closer to 1 unit per acre. Therefore, it was anticipated that future Single Family Residential areas would have an average development density of 1 unit per acre within the future service planning area. This average single family residential development density would also take into account some of the more rugged terrain areas which result in large areas of unbuildable space on some lots.

The development densities and sewer flow generation rates used for the future service planning area are summarized in Tables 10-2 and 10-3.

10.2.3 Trunk Sewer Extensions

Another difference between the existing system model and future model was in the treatment of undeveloped land outside the current sewer system area but within the study planning area boundary. For the existing model, undeveloped land was not included in the flow analysis. For the future model, it was assumed that this land would develop in general accordance with the Planning Districts Map and need sanitary sewer service. As such, planned sewer extensions and lift stations to serve these future areas were located and sized to include future areas into the model. As is common for sewer system planning studies, only the planned trunk facilities were included in the future model; smaller individual sewer lines were not included.

Planned sewer extensions were located in natural valleys wherever possible to provide the maximum service by gravity to the upland areas. In areas where gravity sanitary sewers were not possible due to topographic elevation, planned lift stations and force mains were included in the model to serve the lowest areas.

In addition to topography, the proposed routes selected for trunk sewer facilities were also influenced by locations of roadways, parcel lines, and land uses. It is important to note that the size and location of planned sewers is conceptual in nature, and actual sewer pipe alignments would need to be determined through feasibility studies and final design prior to construction. The proposed conceptual locations of future sanitary sewer trunk facilities needed to adequately serve the planning area are illustrated in Figures 10-2 and 10-3.

TABLE 10-2

DEVELOPMENT DENSITIES FOR FUTURE CONDITIONS
SISTER BAY SANITARY SEWER SYSTEM
VILLAGE OF SISTER BAY, WISCONSIN

		Current Zoning Densities		Units/Acre	People/Unit
Land Use Type	**Zoning Catagories**	**Lot Size**	**Dwelling Units per Lot**	**Future Assumed Density (units/acre)**	**Future Assumed Density (persons/unit)**
Residential	(CS-1) Countryside	10 acre	1	0.20	3.00
	(R-1) Single-Family Residence	20,000 sq. ft.	1	1.00	2.50
	(R-2) Multiple-Family Residence	20,000 sq. ft.	6/acre	6.00	2.25
	(R-3) Large Lot Residence	5 acre	1	1.00	2.50
	(R-4) Small Lot Residence	4,500 sq. ft.	1	9.68	2.00
Non-Residential	(B-1) General Business	20,000/25,000 sq. ft.		2.18	
	(B-2) Downtown Business Transition	4,500 sq. ft.		9.68	
	(B-3) Downtown Business	4,500 sq. ft.		9.68	
	(I-1) Institutional				
	(P-1) Park/Recreation				
	Liberty Grove Industrial	60,000 sq. ft.		0.73	
	Liberty Grove Natural Area	15 acre	1	0.07	

C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\StormSanitary\[Table 10-x.xls]Table 10-2

TABLE 10-3

UNIT AND AREA FLOW GENERATION RATES FOR FUTURE CONDITIONS
SISTER BAY SANITARY SEWER SYSTEM
VILLAGE OF SISTER BAY, WISCONSIN

Land Use Type	Zoning Catagories	Future Flow Rate in Gallons per Capita per Day (GPCD)	Future Flow Rate in Gallons per Acre per Day (GAD)
Residential	(CS-1) Countryside	110	66
	(R-1) Single-Family Residence	100	250
	(R-2) Multiple-Family Residence	90	1,215
	(R-3) Large Lot Residence	100	250
	(R-4) Small Lot Residence	85	1,646
Non-Residential	(B-1) General Business		1,200
	(B-2) Downtown Business Transition		1,500
	(B-3) Downtown Business		1,500
	(I-1) Institutional		1,500
	(P-1) Park/Recreation		75
	Liberty Grove Industrial		1,500
	Liberty Grove Natural Area		10

C:\Documents and Settings\pplanton\My Documents\Projects\Sister Bay copy\StormSanitary\[Table 10-x.xls]Table 10-3

10.2.4 Routing of Future Flows to WWTP

The first option for conveying the future planning area sewer flows to the WWTP was to route the flows through the existing sewer system, and simulate the resulting system impact using the computer model. Simulated model results of future flow conditions were then compared to the existing pipe capacities. Existing sewer pipes with a future utilization of 100 percent or more indicated a future capacity restriction deficiency. Since critical sections of the existing sanitary sewer system are already near capacity with the current development levels, this flow conveyance option resulted in large sections of the existing system becoming overloaded and needing reconstruction.

Other options for conveying future sewer flows to the WWTP were also evaluated. For example, the feasibility of constructing new lift stations and force mains in the growth areas to pump flows directly to the WWTP was evaluated. The estimated costs for building new sewer facilities to convey flows to the WWTP were compared to the estimated costs for reconstructing sewers and upgrading existing facilities. The purpose of this comparison was to determine the most cost-effective and feasible approach to providing sewer service to the future growth areas. This comparison will be detailed further in the following sections.

To better delineate phases of construction and the associated costs, the future expansion area was broken down into “regions” for further analysis. There are six individual sewer service regions identified in the northern portion of the future sewer service expansion area (Regions A – F), and four regions identified in the southern portion of the expansion area (Regions G – J).

10.2.5 Northern Regions

The northern sewer service planning regions include Regions A – F as illustrated in Figure 10-2. Due to the flat topography and limited gravity sewer alignment options, sewer service to Regions A, B, C and D will require lift stations and force mains. Lift stations for each region would pump to the adjacent region, with the sewer flows for the four regions eventually discharging into existing system MH 298. In addition to the cost of the lift stations, these regions will be more expensive to develop and provide sewer service due to the subsurface rock and the tight working conditions.

Future development located on the eastern end of Region C will be a difficult area to serve due to the existing flat terrain and the need for a lift station. The costs for providing sanitary sewer service to this relatively small area will be high. As a result, this area (indicated in Figure 10-2) is not recommended to receive sanitary sewer service at this time due to the high cost per acre of providing service.

On the other hand, Regions E and F are not expected to need lift stations, and can be served by gravity from the existing sewer system. Region E would discharge into the north end of the existing system, and Region F would discharge into the area north of the WWTP. These areas will be less expensive to serve per acre because of the lower infrastructure costs, the closer proximity to the existing collection system, and the less developed conditions which will require less restoration.

10.2.6 Southern Regions

10.2.6.1 Regions G, I and J

The southern sewer service planning regions include Regions G – J as illustrated in Figure 10-3. Region G can feed directly into the existing sewer system near the Little Sister Lift Station.

The small area inside the planning boundary southeast of Lift Station H would require a lift station to be served. Because this area is small, outside the Village corporate limit, difficult to serve, and the cost per acre to provide sanitary sewer service would be high, extending service to this area is not recommended at this time.

A lift station is anticipated to be needed south of Plateau Road on Woodcrest Road to serve Region I. This region is a naturally low lying area that drains to the east. Proposed Lift Station I would pump north and discharge near the intersection of Plateau Road and Woodcrest Road.

Region J is lower in elevation than the receiving area at the WWTP, and therefore will need a lift station located near the lowest point in the region. The likely location would be in the park near the intersection of Woodcrest Road and Autumn Court. From proposed Lift Station J, wastewater flows would be pumped a short distance east to the WWTP.

10.2.6.2 Region H

Region H is a large area that can be served by a single lift station located immediately north of the intersection of Country Lane and Hwy 57. This station may need to be relatively deep to serve the low-lying area northwest of this intersection. This approach to serving Region H would also allow the abandonment of the existing Fieldcrest Lift Station because a new gravity sewer line (P134) would be installed south along Fieldcrest Road to Proposed Lift Station H. This sewer alignment would divert the current Fieldcrest Lift Station service area flow out of the existing system, which would provide additional existing system capacity for the anticipated flows that would be added from serving Region G.

Multiple options were evaluated to determine the best way to serve Region H. After many discussions with Village staff, a preferred alternative was established. The preferred alternative will consist of running the future force main from Lift Station H north along Hwy 57 to the crest of the hill north of Country Walk Drive. At the crest of the hill, the force main will discharge into a new gravity trunk sewer that will run down Hwy 57 and Bay Shore Drive to Lift Station 1. Flows at Maple Drive and Mill Road will be picked up by this new pipe. In addition, the existing sewer pipe located through the back yards east of Bay Shore Drive can be abandoned once all existing service laterals are reconnected to the new pipe. Lift Station 1 will need to be upgraded, but the existing force mains pumping to the WWTP appear to have sufficient capacity to accommodate the additional flow. The route of the proposed trunk sewer through the downtown area to serve Region H is shown in Figure 10-4.

Under this alternative, the recommended force main pipe from Lift Station H to the crest of the hill would be 8 inches in diameter. The downstream trunk sewer to Lift Station 1 will steadily increase in size as it proceeds downstream, and will range from approximately a 10” pipe to an 18” pipe, depending on the location along the route and the number of connections to the existing system which are made.

Lift Station 1 will need to be upgraded by increasing the pump capacity. More study would be needed to determine the exact improvements needed at the station, but at this time, it is estimated that the total pumping capacity at the station would need to increase by approximately 50 percent. As the service area fills in and the existing system expands in other areas, Lift Station 1 may need to be upgraded even further in the long-term future.

A number of alternatives were studied prior to determining the preferred option for serving Region H. Information pertaining to these other options can be found in Appendix F. The primary advantages to the preferred alternative are listed below:

- This alternative may allow more development to occur faster in the southern part of the service area, because it would not be dependent on development in Regions I and J.
- It would provide additional sewer capacity in the downtown area.
- It would provide a level of redundancy that currently does not exist in the downtown area. This would allow some of the existing sewer mains to be taken out of service for maintenance if needed, while still providing sewer service to customers.
- If the project is done concurrently with the State's Hwy 57 reconstruction project, there should be cost sharing opportunities with the State on the surface restoration.

Detailed feasibility studies should be conducted to confirm the preliminary findings before any improvements are designed or constructed.

10.2.6.3 Region H – Interim Lift Station

Development pressure currently exists in the northeast part of Region H. Based on the model assumptions; it is projected that the existing system could accommodate a small interim lift station located in the northeast part of Region H as an interim way to provide sewer service in this area. This interim station would only be a temporary solution until such time as Lift Station H and the associated downstream improvements were constructed.

If the diversion were constructed from MH 39 to MH 193, it is estimated that an interim lift station with duel 100 gpm pumps could be constructed somewhere in the northeast part of Region H. If the diversion were not constructed, there would not be capacity in the Mill Road sewer to accommodate this additional flow, so the diversion must be constructed first. The maximum pumping rate of the station should not exceed 150 gpm with both pumps operating simultaneously. The flow could be temporarily pumped to MH 317, which is located on the south end of Smith Drive. For comparison, this interim station would be the approximate size of the existing Fieldcrest Lift Station.

As part of the design on this interim lift station, a detailed feasibility study would be required to confirm that the downstream system is adequate to accept the flows resulting from the specific pumps and pumping conditions being proposed for that lift station.

If the interim station were to be constructed, it is recommended that flow monitors be periodically installed in the downstream system to monitor the flow levels after the station comes on line. This would allow the Village to observe the flow levels and take appropriate action if the levels become too high. The monitors should be installed on the critical segments of both sides of the diversion (i.e., the Maple/Mill Road side and the Claflin/South Spring side). Lift Station 1 should also be monitored for any capacity issues.

Using the flow rates assumed in the this report, the maximum pumping rate of 150 gpm translates into approximately 150 single family homes. Although the minimum lot size for R-1 zoning is 20,000 square feet (approx. 0.5 acre), the amount of land typically attributed to each new lot is significantly more if roadways, ponding areas, and open spaces are also included. If the diversion is constructed, it is estimated that approximately 150 residential equivalent units of capacity would be available for development upstream of the Mill Road restriction. This would include areas to the south and west of the Mill Road location.

Figure 10-5 shows one option for serving the northeast part of Region H. The approximate sewer service boundary is shown based on the existing topography. Areas beyond the limits shown would be difficult to serve without additional lift stations. This option illustrates providing service to only the undeveloped land north of Hwy 57. The interim lift station is located near Northwoods Drive in the lowest part of the service area to save costs on sewer pipe depth. The force main would pump northeast along Hwy 57 to MH 317.

Planning level costs for the interim station are summarized in Table 10-4. The force main cost is based on an assumed 6 inch diameter force main, and contains a significant allowance for rock excavation, restoration and contingencies. A detailed feasibility study should be conducted to confirm the preliminary findings before any improvements are designed or constructed.

For the purposes of this cost estimate, it was assumed that the size of the lift station would be similar to the Fieldcrest Lift Station. A less costly grinder pump system was considered due to the smaller design flows, but it was determined that the flows could exceed the recommended threshold for a grinder pump. In addition, using a 100 gpm duplex station for each option would provide the Village with the most flexibility for providing sanitary sewer service to this developing area.

10.2.7 Impact of Future Expansion on Existing Facilities

The computer model was used to evaluate the impact to the existing system of adding the flows from proposed Regions A, B, C, D, E, F, G, H and I. Other than the issues already mentioned, the results of this analysis indicated that only a single sewer pipe segment would have a capacity restriction with the addition of these flows. The affected segment is an 8 inch pipe from MH 106 to MH 104 which has a very flat slope. The model results indicate that this pipe segment would significantly exceed its capacity due to the additional flows from the north. Accordingly, it is recommended that this segment be removed and replaced with a larger diameter sewer pipe prior to the full build-out of Regions A – D. The location of this pipe segment is shown in Figure 10-6.

10.3 RECOMMENDED SANITARY SEWER CAPITAL IMPROVEMENTS

10.3.1 Estimated Cost of Trunk Facilities to Address Existing Needs

The improvements to address existing deficiencies are shown in Figure 10-1 and summarized in Table 10-1. These improvements address three different types of deficiencies: potential future capacity restrictions, pipe settlements and sump manholes.

The estimated costs to address existing deficiencies are presented in Table 10-5. These are preliminary budget estimates only, and actual costs must be determined through the competitive bidding process. The costs are in 2006 dollars and include anticipated contingencies and indirect project costs.

TABLE 10-4

PLANNING LEVEL COST FOR INTERIM LIFT STATION IN REGION H

SISTER BAY SANITARY SEWER SYSTEM

VILLAGE OF SISTER BAY, WISCONSIN

ITEM	Size (in.)	Length (ft.)	Cost per Foot	Estimated Construction	Restoration Unimproved (ft.)	Restoration Partial Improved (ft.)	Restoration Cost*	Rock Excavation Cost**	Subtotal Construction	Contingencies and Engineering (30%)	Total Estimated Cost
F.M.	6 FM	2,750	$40	$110,000		1,900	$57,000	$165,000	$332,000	$99,600	$431,600
L.S.	100 gallons/minute			$90,000					$90,000	$27,000	$117,000
TOTAL											**$549,000**

***Restoration cost based on $.60 per linear foot for unimproved and $30 per linear foot for partially improved areas.**

**** Rock Excavation estimated at $60 per linear foot, assumed to be an average area of 4' wide x 6' deep.**

P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay study\March-April 2008 Report Revisions\Chapters 1-14\Chapter 10\Table 10-4.xls

TABLE 10-5

ESTIMATED COST OF IMPROVEMENTS TO ADDRESS EXISTING DEFICIENCIES
SISTER BAY SANITARY SEWER SYSTEM
VILLAGE OF SISTER BAY, WISCONSIN

System Deficiency	System Location	Description	Recommended Improvement	Approximate Quantities	Planning Level Costs (2006 Dollars)
Potential Future Capacity Restriction	MH019 to MH011	Highest Utilized Pipes in System	Construct Partial Diversion From MH039 to MH193	75 LF of 10 inch pipe plus bypass pumping, manholes, restoration, and traffic control	$50,000
Capacity Restriction due to Pipe Settlement	MH047 to MH045	Minimal pipe slope due to settlement of sewer pipe; sedimentation of solids	Remove and replace 10-inch pipe	Remove and replace 240 LF of 10 inch pipe, plus bypass pumping, restoration and traffic control	$60,000
Sump Manholes	MH073 to MH121 MH123 to MH131 MH220 to MH228 MH198 to MH204	Sumps create maintenance problems due to sedimentation of solids	Pour concrete inverts in manholes to eliminate sumps	34 manholes plus cleaning, bypass pumping and traffic control	$40,000

P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay study\March-April 2008 Report Revisions\Chapters 1-14\Chapter 10\Table 10-5.xls

10.3.2 Estimated Cost of Trunk Facilities to Serve Future Growth

Preliminary cost estimates for the proposed trunk facility improvements to serve expansion areas in 2006 dollars are presented in Table 10-6. These estimates include allowances for surface restoration, construction contingencies, and engineering.

The linear foot costs listed in the table for each pipe size are approximate. They are based on recent projects, and make assumptions for some rock excavation and dewatering averaged out across the system. In areas where sewer and water are located in a common trench, the rock excavation and restoration costs have been split between the sewer and water categories. Actual costs may vary significantly depending upon actual conditions within each region and within each sub-region.

Two different restoration categories are used to address anticipated restoration costs. The first restoration category is for unimproved areas. In this category, it is assumed that pipes will be installed across an undeveloped, “unimproved” area. Restoration costs for this category include only turf establishment within the work area. The second category of restoration is for “partially improved” areas. This category assumes that the pipes will be installed along and near a street (perhaps in a ditch area), and that part of the pavement and shoulder may need to be replaced. Table 10-6 lists the assumed restoration type and length for each pipe run.

An allowance was included in the cost estimate for rock excavation and dewatering. The rock excavation allocation represents the cost of about 6 foot of rock excavation depth across all of the piping. Since actual depth of rock excavation at each location is unknown, this allowance may be high or low, depending on the actual conditions at each location. It should be noted that costs for rock excavation were split between the sewer and water categories in areas where the utilities were installed in a common trench.

Due to the exact location of the trunk facilities being unknown at this time, costs are considered preliminary. Extraordinary costs such as subsurface crossings, removal and replacement of other existing utilities, easement costs, etc., are not included in the preliminary estimates. However, the 20% contingency may cover a number of these miscellaneous costs.

Table 10-6 lists the costs for the facilities that are needed within each of the 10 future sewer service regions. Since some flows are conveyed from one region through another region, some facilities would need to be designed larger to accommodate flows from other regions. As a result, if the Village wishes to pass these costs back to the regions in some manner, there is an important distinction that is worth noting. The costs spent within any particular region to construct the facilities may be higher that the costs that are incurred as a result of serving that particular region. This will be true for regions that are located downstream of other regions.

As this study’s recommendations are conceptual in nature, detailed feasibility reports and cost estimates should be prepared prior to the design and construction of any improvements.

Operating costs for the future collection system facilities primarily involve electricity to run the new lift station pumps. The annual operating cost for any lift station will depend on the peak flow rate and head conditions at the station. It will also depend on the cost of electricity. Since pumps slowly wear over time, electrical costs will increase over time, due to increased friction, decreased efficiency, and the need for more power to run the pumps. Operating costs could also include the cost to operate portable generator sets, which are needed to run the stations if electrical power is interrupted. Due to the many variables

TABLE 10-6

ESTIMATED COST OF TRUNK FACILITIES TO SERVE EXPANSION AREAS
SISTER BAY SANITARY SEWER SYSTEM
VILLAGE OF SISTER BAY, WISCONSIN

ITEM	Size (in.)	Length (ft.)	Cost per Foot	Estimated Construction	Restoration Unimproved (ft.) [3]	Restoration Partial Improved (ft.) [3]	Restoration Cost	Rock Excavation Cost [4]	Engineering & Contingencies (30%)	Total Estimated Cost (2006 Dollars)
REGION A										
P101	8	1,100	$40	$44,000		1,100	$33,000	$66,000	$42,900	$185,900
P102[1]	8	1,530	$40	$61,200	205	1,325	$19,900	$45,900	$38,100	$165,100
P103	8	930	$40	$37,200		930	$27,900	$55,800	$36,300	$157,200
F.M. - A	6 FM	1,200	$40	$48,000		1,200	$36,000	$72,000	$46,800	$202,800
L.S. - A	20 gallons/minute			$60,000					$18,000	$78,000
REGION A Total										**$789,000**
REGION B										
P104[1]	8	2,015	$40	$80,600		2,015	$30,200	$60,500	$51,400	$222,700
P105[1]	8	800	$40	$32,000		800	$12,000	$24,000	$20,400	$88,400
F.M. - B	6 FM	300	$40	$12,000	300		$200	$18,000	$9,100	$39,300
L.S. - B	15 gallons/minute			$60,000					$18,000	$78,000
REGION B Total										**$429,000**
REGION C										
P106[1]	8	5,700	$40	$228,000		5,700	$85,500	$171,000	$145,400	$629,900
P107[1]	8	1,490	$40	$59,600		1,490	$22,400	$44,700	$38,000	$164,700
P111	8	2,270	$40	$90,800		2,270	$68,100	$136,200	$88,500	$383,600
P111[2]	8	1,400	$40	$56,000		1,400	$21,000	$42,000	$35,700	$154,700
P112	8	4,150	$40	$166,000		4,150	$124,500	$249,000	$161,900	$701,400
P112[2]	8	1,900	$40	$76,000		1,900	$28,500	$57,000	$48,500	$210,000
P113	8	1,700	$40	$68,000		1,700	$51,000	$102,000	$66,300	$287,300
P113[2]	8	2,600	$40	$104,000		2,600	$39,000	$78,000	$66,300	$287,300
P114[1]	8	560	$40	$22,400		560	$8,400	$16,800	$14,300	$61,900
F.M. - C	8 FM	905	$45	$40,700	905		$500	$54,300	$28,700	$124,200
L.S. - C	220 gallons/minute			$144,000					$43,200	$187,200
REGION C - Total										**$3,193,000**

Note: All Region Totals Rounded to the Nearest $1,000

[1] Entire trench common with water piping, costs split for entire lengths

[2] Part of trench common with water piping, costs split for common lengths

[3] Restoration cost based on $0.60 per lineal foot for unimproved and $30 per lineal foot for improved

[4] Rock excavation estimated at $60 per lineal foot, assumed be be an average of 4' wide by 6' deep

TABLE 10-6

ESTIMATED COST OF TRUNK FACILITIES TO SERVE EXPANSION AREAS
SISTER BAY SANITARY SEWER SYSTEM
VILLAGE OF SISTER BAY, WISCONSIN

ITEM	Size (in.)	Length (ft.)	Cost per Foot	Estimated Construction	Restoration Unimproved (ft.) [3]	Restoration Partial Improved (ft.) [3]	Restoration Cost	Rock Excavation Cost [4]	Engineering & Contingencies (30%)	Total Estimated Cost (2006 Dollars)
REGION D										
P115	8	1,300	$40	$52,000		1,300	$39,000	$78,000	$50,700	$219,700
P115[2]	8	2,500	$40	$100,000		2,500	$37,500	$75,000	$63,800	$276,300
F.M. - D	8 FM	330	$45	$14,850	180	50	$1,608	$19,800	$10,900	$47,200
L.S. - D	250 gallons/minute			$144,000					$43,200	$187,200
REGION D - Total										**$731,000**
REGION E										
P116[1]	8	1,200	$40	$48,000		1,200	$18,000	$36,000	$30,600	$132,600
P117	8	1,680	$40	$67,200		1,680	$50,400	$100,800	$65,500	$283,900
P118	8	1,380	$40	$55,200		1,380	$41,400	$82,800	$53,800	$233,200
P119	8	1,300	$40	$52,000		1,300	$39,000	$78,000	$50,700	$219,700
P119[2]	8	700	$40	$28,000		700	$10,500	$21,000	$17,900	$77,400
REGION E - Total										**$947,000**
REGION F										
P120[1]	8	2,800	$40	$112,000		2,800	$42,000	$84,000	$71,400	$309,400
P121	8	525	$40	$21,000		525	$15,800	$31,500	$20,500	$88,800
P122	8	750	$40	$30,000		750	$22,500	$45,000	$29,300	$126,800
P122[2]	8	1,100	$40	$44,000		1,100	$16,500	$33,000	$28,100	$121,600
P123	8	1,250	$40	$50,000		1,250	$37,500	$75,000	$48,800	$211,300
P124[1]	8	3,600	$40	$144,000		3,600	$54,000	$108,000	$91,800	$397,800
P125	8	860	$40	$34,400		860	$25,800	$51,600	$33,500	$145,300
P126[1]	8	925	$40	$37,000		925	$13,900	$27,800	$23,600	$102,300
REGION F - Total										**$1,504,000**

Note: All Region Totals Rounded to the Nearest $1,000

[1] Entire trench common with water piping, costs split for entire lengths

[2] Part of trench common with water piping, costs split for common lengths

[3] Restoration cost based on $0.60 per lineal foot for unimproved and $30 per lineal foot for improved

[4] Rock excavation estimated at $60 per lineal foot, assumed be be an average of 4' wide by 6' deep

TABLE 10-6

ESTIMATED COST OF TRUNK FACILITIES TO SERVE EXPANSION AREAS
SISTER BAY SANITARY SEWER SYSTEM
VILLAGE OF SISTER BAY, WISCONSIN

ITEM	Size (in.)	Length (ft.)	Cost per Foot	Estimated Construction	Restoration Unimproved (ft.) [3]	Restoration Partial Improved (ft.) [3]	Restoration Cost	Rock Excavation Cost [4]	Engineering & Contingencies (30%)	Total Estimated Cost (2006 Dollars)
REGION G										
P127	8	2,200	$40	$88,000		2,200	$66,000	$132,000	$85,800	$371,800
P127[2]	8	800	$40	$32,000		800	$12,000	$24,000	$20,400	$88,400
P128	8	450	$40	$18,000		450	$13,500	$27,000	$17,600	$76,100
P128[2]	8	1,000	$40	$40,000		1,000	$15,000	$30,000	$25,500	$110,500
P129	8	300	$40	$12,000		300	$9,000	$18,000	$11,700	$50,700
P129[2]	8	2,500	$40	$100,000		2,500	$37,500	$75,000	$63,800	$276,300
P130[1]	8	1,100	$40	$44,000		1,100	16,500	33,000	$28,100	$121,600
REGION G - Total										**$1,096,000**
REGION H										
P131[1]	8	3,800	$40	$152,000		3,800	$57,000	$114,000	$96,900	$419,900
P132	8	2,650	$40	$106,000		2,650	$79,500	$159,000	$103,400	$447,900
P133	8	2,350	$40	$94,000		2,350	$70,500	$141,000	$91,700	$397,200
P134[1]	8	3,000	$40	$120,000		3,000	$45,000	$90,000	$76,500	$331,500
P135	8	4,000	$40	$160,000		4,000	$120,000	$240,000	$156,000	$676,000
P135[2]	8	600	$40	$24,000		600	$9,000	$18,000	$15,300	$66,300
P136	8	2,100	$40	$84,000		2,100	$63,000	$126,000	$81,900	$354,900
P136[2]	8	1,600	$40	$64,000		1,600	$24,000	$48,000	$40,800	$176,800
P137	8	300	$40	$12,000		300	$9,000	$18,000	$11,700	$50,700
P137[2]	8	1,000	$40	$40,000		1,000	$15,000	$30,000	$25,500	$110,500
P138	8	500	$40	$20,000		500	$15,000	$30,000	$19,500	$84,500
P138[2]	8	1,000	$40	$40,000		1,000	$15,000	$30,000	$25,500	$110,500
P139	8	2,325	$40	$93,000		2,325	$69,800	$139,500	$90,700	$393,000
P140	8	5,400	$40	$216,000		5,400	$162,000	$324,000	$210,600	$912,600
P141	8	1,100	$40	$44,000		1,100	$33,000	$66,000	$42,900	$185,900
P141[2]	8	2,500	$40	$100,000		2,500	$37,500	$75,000	$63,800	$276,300
P142	8	1,200	$40	$48,000		1,200	$36,000	$72,000	$46,800	$202,800
P143[1]	8	1,125	$40	$45,000		1,125	$16,900	$33,800	$28,700	$124,400
P144	8	1,125	$40	$45,000		1,125	$33,800	$67,500	$43,900	$190,200
P144a[5]	12 - 18	3,600		$342,000					$103,000	$445,000
F.M. - H	8 FM	6,260	$45	$281,700		6,500	$195,000	$375,600	$255,700	$1,108,000
L.S. 1 Upgrade	500 gallons/minute			$176,000					$52,800	$228,800
L.S. - H	340 gallons/minute			$160,000					$48,000	$208,000
REGION H - Total										**$7,502,000**

Note: All Region Totals Rounded to the Nearest $1,000

[1] Entire trench common with water piping, costs split for entire lengths

[2] Part of trench common with water piping, costs split for common lengths

[3] Restoration cost based on $0.60 per lineal foot for unimproved and $30 per lineal foot for improved

[4] Rock excavation estimated at $60 per lineal foot, assumed be be an average of 4' wide by 6' deep

[5] Does not include street restoration costs due to state resurfacing project

TABLE 10-6

ESTIMATED COST OF TRUNK FACILITIES TO SERVE EXPANSION AREAS
SISTER BAY SANITARY SEWER SYSTEM
VILLAGE OF SISTER BAY, WISCONSIN

ITEM	Size (in.)	Length (ft.)	Cost per Foot	Estimated Construction	Restoration Unimproved (ft.) [3]	Restoration Partial Improved (ft.) [3]	Restoration Cost	Rock Excavation Cost [4]	Engineering & Contingencies (30%)	Total Estimated Cost (2006 Dollars)
REGION I										
P145[1]	8	2,325	$40	$93,000		2,325	$34,900	$69,800	$59,300	$257,000
P146	8	1,360	$40	$54,400		1,360	$40,800	$81,600	$53,000	$229,800
F.M. - I	6 FM	1,260	$40	$50,400		1,260	$37,800	$75,600	$49,100	$212,900
L.S. - I	100 gallons/minute			$90,000					$27,000	$117,000
REGION I - Total										**$817,000**
REGION J										
P147[1]	8	2,700	$40	$108,000		1,210	$18,150	$81,000	$41,430	$248,600
P147a	10	1,210	$45	$54,450		2,700	$81,000	$72,600	$41,610	$249,700
P148[1]	8	3,810	$40	$152,400		3,810	$57,200	$114,300	$64,780	$388,700
P149	8	1,160	$40	$46,400		1,160	$34,800	$69,600	$30,160	$181,000
P150[1]	8	1,300	$40	$52,000		1,300	$19,500	$39,000	$22,100	$132,600
P151	8	2,460	$40	$98,400		2,460	$73,800	$147,600	$63,960	$383,800
P152	8	3,205	$40	$128,200		3,205	$96,200	$192,300	$83,340	$500,100
F.M. - J	8 FM	1,900	$45	$85,500		1,900	$57,000	$114,000	$51,300	$307,800
L.S. - J	380 gallons/minute			$165,000					$33,000	$198,000
REGION J - Total										**$2,591,000**

Note: All Region Totals Rounded to the Nearest $1,000

P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay study\March-April 2008 Report Revisions\Chapters 1-14\Chapter 10\Tables & Figures\Used in March-April Final Draft\Excel\Table 10-6.xls

[1] Entire trench common with water piping, costs split for entire lengths

[2] Part of trench common with water piping, costs split for common lengths

[3] Restoration cost based on $0.60 per lineal foot for unimproved and $30 per lineal foot for improved

[4] Rock excavation estimated at $60 per lineal foot, assumed be be an average of 4' wide by 6' deep

involved in estimating operating costs, such costs are not included in this report. However, approximate operating costs could be estimated by reviewing the electrical records from the Village's existing stations, and by relating these records to similarly sized future stations.

10.3.3 Schedule of Improvements

The timing of future trunk sanitary sewer improvements will be influenced by a number of parameters. Items such as the location of development pressure in specific areas, aging facilities and/or facilities which are undersized, availability of funds, etc., all play a role in the timing of future sanitary sewer improvements.

Because of the factors involved, it is difficult to accurately predict the timing of future improvements, especially those which may occur far into the future. However, some areas of the Village are more likely to experience rapid development than others.

Based on input from Village staff, an estimated Capital Improvement Plan (CIP) for sanitary sewer has been developed. The CIP is broken down into short-term and long-term improvements. Short term improvements generally include improvements that are needed to address existing deficiencies. Short term improvements can also include improvements to accommodate future development in areas where development is relatively cost effective, such as areas that do not need lift stations. Long term improvements typically include providing service to future expansion areas that are located farther from the existing system and are more expensive to construct. The CIP for short term improvements is presented in Table 10-7.

The first and second stages of Region H improvements have been listed under short term improvements. There is development interest in this area, but construction of the full lift station at location H is not necessary or cost effective at this time. The first stage of Region H improvements would include an interim lift station and force main located near the intersection of pipes P135 and P139. This interim station would allow development to occur north of pipe P135, as is currently proposed. The interim force main would connect to the existing system at MH 317. The second stage of Region H improvements would include construction of a small lift station at location H that would be designed to have its pumping capacity increased in the future. An interim force main could be installed along Hwy 57 which would connect to the downstream system.

The recommended construction of P144a, a new trunk sewer down Bay Shore Drive, is also included in the short term improvements. The State plans to do a reconstruction project of Bay Shore Drive in the near future, and the installation of this trunk sewer should be done in conjunction with the State project. Upgrades to Lift Station No. 1 are also part of the short term improvements.

Proposed sanitary sewer Regions E, F and G do not require lift stations, and can be served by gravity sewers discharging into the existing collection system. In addition, sewer service provided to these regions will not cause adverse impacts on the existing system. Accordingly, sanitary sewer improvements in these regions have been identified as short term improvements.

Recommended long term system improvements consist of all remaining facilities identified in the CIP. These projects include the remainder of Region H improvements as well as improvements to provide service to Regions A, B, C, D, I and J. Improvements also include the reconstruction of an existing sewer pipe segment that is undersized to handle the increase in flows for the expansion areas. The 8 inch pipe

TABLE 10-7

SANITARY SEWER CAPITAL IMPROVEMENT PLAN - SHORT TERM IMPROVEMENTS
SISTER BAY SANITARY SEWER SYSTEM
VILLAGE OF SISTER BAY, WISCONSIN

Type of Improvement	System Location	Recommended Improvement	Planning Level Costs (2006 Dollars)
Potential Future Capacity Restriction	MH019 to MH011	Construct Diversion From MH039 to MH 193	$50,000
Pipe Settlement	MH047 to MH045	Remove and replace 10 inch pipe	$60,000
Sump Manholes	MH073 to MH121 MH123 to MH131 MH220 to MH228 MH198 to MH204	Pour concrete inverts in manholes to eliminate sumps	$40,000
Future Expansion	Trunk Sewer P144a	Construct Trunk Sewer in Bay Shore Drive	$445,000
Future Expansion	Lift Station 1	Upgrade Station Capacity	$229,000
Future Expansion	Stage 1 of Region H Improvements	Construct Interim Lift Station and force main (at intersection of P135/P139)	$549,000
Future Expansion	Stage 2 of Region H Improvements	Construct first phase of Lift Station H, interim force main	$2,000,000
Future Expansion	Region E Improvements	Construct Gravity Facilities (no lift stations required)	$947,000
Future Expansion	Region F Improvements	Construct Gravity Facilities (no lift stations required)	$1,504,000
Future Expansion	Region G Improvements	Construct Gravity Facilities (no lift stations required)	$1,096,000
		Total	**$6,920,000**

P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay study\March-April 2008 Report Revisions\Chapters 1-14\Chapter 10\Tables & Figures\Used in March-April Final Draft\Excel\[Tables 10-7.xls]Table 10-7 from MH 106 to MH 104 will need to be replaced with a 10 inch pipe to accommodate future anticipated flows from development in the northern planning area.

The proposed CIP for long term sanitary sewer improvements is presented in Table 10-8.

10.3.4 Financing of Sewer Improvements

Expanding the existing trunk sanitary sewer system to accommodate future development can include upgrading existing lift stations and force mains, constructing relief sewers in certain areas, constructing new gravity extensions, and constructing new lift stations and force mains in areas where gravity service is not feasible.

It is anticipated that these improvements will either be financed by a developer, assessed to benefiting properties, paid for by the Utility, or a combination thereof. Typically, future trunk extensions will be constructed and paid for in conjunction with a development project. In some communities, the costs of trunk system extensions are the sole responsibility of the developer. In other communities, the developer has the option of allowing the trunk improvements to be constructed by the Utility with all associated costs being assessed back to the benefiting properties.

Construction of future lift stations and force mains can be treated in a similar way to trunk extensions. If the lift stations and force mains are necessary only to serve new development, the entire cost of these facilities can be passed back to the identified new development. If development is staged, it may be possible to stage the improvements to track with the development. When staging improvements is not possible, over-sizing costs can be recovered through special assessments, impact fees, connection charges, or other means. As a last resort, over-sizing costs may need to be carried by the Utility until future development occurs within the larger service area, at which time the costs can be recovered from the development through one of the methods described above.

Upgrading existing lift stations/force mains, and constructing relief sewers in developed areas are more difficult to tie to specific developments, and therefore, they can be more difficult to finance. In addition to a Sanitary Sewer Utility, some communities utilize a municipal Sewer Availability Charge (SAC), a Sewer Connection Charge, a Sewer Trunk Area Charge, or an Impact Fee to build funds that can be used to upgrade existing facilities. These are one-time fees that can be collected at the time of final plat, or at the time of building permit, to supplement the Utility's revenue stream. As significant development is expected to occur in the future, Sister Bay may wish to implement one of these options to help offset the larger demand that the new development will place on the existing sewer utility's customer base.

Although the Utility's current balance in the Sanitary Sewer Fund may comfortably operate the Sewer Utility today, it is insufficient to operate the Utility and also pay for all of the above listed improvements that will be needed prior to full development of the system. Additional funds will be needed. The sewer user charge system and the payment of charges are described in Sections 62-9 and 62-10 of the Village's ordinances. A complete rate analysis and review of the Utility's current account balances is outside of the scope of this study. It is recommended that a separate rate study be performed to compare the projected revenues to the projected costs listed in the Tables 10-7 and 10-8.

TABLE 10-8

SANITARY SEWER CAPITAL IMPROVEMENT PLAN - LONG TERM IMPROVEMENTS

SISTER BAY SANITARY SEWER SYSTEM
VILLAGE OF SISTER BAY, WISCONSIN

Type of Improvement	System Location	Recommended Improvement	Planning Level Costs
Provide Service to Expansion Areas	Stage 3 of Region H Improvements	Enlarge L.S. H, Extend Force Main, Region H Trunk Mains	$4,829,000
Provide Service to Expansion Areas	Region J Improvements	Construct Lift Station and Trunk Mains in Stages	$2,591,000
Provide Service to Expansion Areas	Region I Improvements	Construct Lift Station and Trunk Mains	$817,000
Increase Capacity of Existing Pipe	MH106 to MH104	Upgrade 8 inch Pipe to 10 inch Pipe	$45,000
Provide Service to Expansion Areas	Region D Improvements	Construct Lift Station and Trunk Mains	$731,000
Provide Service to Expansion Areas	Region C Improvements	Construct Lift Station and Trunk Mains	$3,193,000
Provide Service to Expansion Areas	Region B Improvements	Construct Lift Station and Trunk Mains	$429,000
Provide Service to Expansion Areas	Region A Improvements	Construct Lift Station and Trunk Mains	$789,000
		Total	**$13,424,000**

P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay study\March-April 2008 Report Revisions\Chapters 1-14\Chapter 10\Tables & Figures\Used in March-April Final Draft\Excel\[Tables 10-8.xls]Table 10-8

10.3.5 Short-Term System Improvement Impacts on Utility Revenue Requirements

Table 10-9 summarizes the results of a preliminary analysis of the probable impact on Sewer Utility revenue requirements of implementing the recommended short-term capital improvements. The effect of the short-term improvements with respect to each revenue requirement category has been estimated.

Table 10-10 summarizes projected increases in the Sewer Utility's 2005 revenue requirements with the implementation of the proposed short-term improvements. The Sewer Utility's 2005 revenue requirements were obtained from a review of the Village's 2005 Sewer Rate Study (Revision 5). It is projected that the improvements would cause the Utility's annual revenue requirements to increase by $608,800 to approximately $994,810. This represents a 258 percent increase from the revenue requirements listed in the 2005 Sewer Rate Study report. The actual impact on sewer rates would need to be determined based on the Utility's revenues in the year the improvements were constructed.

10.3.6 Wastewater Collection System Ordinance Review

As part of the analysis of future improvements, a review of the Village's existing wastewater collection regulations and ordinances was conducted. The purpose of this review was to identify any changes that could be made to the ordinances that would allow the Village to better implement the collection system recommendations contained in this report.

Several documents were reviewed, including the Municipal Code of the Village of Sister Bay, and the Engineering Design Manual.

10.3.6.1 Municipal Code

The Municipal Code contains the essential rules and regulations pertaining to governance of the Village. The wastewater collection system is discussed primarily in Chapter 54 (Land Division and Platting Code) and Chapter 62 (Utilities).

Chapter 54 of the Municipal Code contains Section 54.105, Sanitary Sewerage System. This section covers the design, installation and cost recovery aspects of constructing sewer facilities in conjunction with development. Section 54.105 addresses these areas quite thoroughly, and only a few suggested additions are recommended:

1. Reference this Comprehensive Utilities Plan and its role in the development review process in Section 54.105. Although this plan is conceptual and schematic in nature, this plan should be an important tool for the Village in the development review process. The plan is intended to be used as a guide, for both developers and the Village, of an efficient and economic way to construct the future collection system. Concept plans submitted by developers should be consistent with the "spirit" of the plan, whenever possible. This may not be possible in some cases, due to unique conditions and constraints that are not known at this time. However, where it is not possible to follow the concepts identified in the plan, the developer should document why the proposed deviation would be in the best interests of the Village. This plan should be kept on file at the Village Hall, and should be open to inspection by the public during normal office hours.

2. Section 54.104 (k) (3) states that the Village will pay some portion of the oversizing costs for sewer pipes that need to be oversized to accommodate future development. Sewer connection fees are mentioned in 54.104 (m) and these fees may be a source of cost recovery for the

TABLE 10-9

CAPITAL IMPROVEMENTS IMPACT ON UTILITY REVENUE REQUIREMENTS

SISTER BAY SANITARY SEWER SYSTEM
VILLAGE OF SISTER BAY, WISCONSIN

Recommended Sewer System Capital Improvement*	Budget Cost Estimate	*Estimated Increase in Utility Revenue Requirements* Operation & Maintenance Expenses	Equipment Replacement Cost	Debt Service**	Total
Existing Deficiency Improvements	$150,000	$800	$0	$12,000	**$12,800**
Future Expansion:					
Trunk Sewers	$3,992,000	$20,000	$0	$320,000	**$340,000**
Lift Stations/F.M.	$2,778,000	$27,800	$5,000	$223,000	**$255,800**
Total	**$6,920,000**	**$48,600**	**$5,000**	**$555,000**	**$608,600**

* as taken from Table 10-7

**Debt service calculated based on 5% at 20 year term.

TABLE 10-10

PRELIMINARY COST OF SERVICE ANALYSIS

SISTER BAY SANITARY SEWER SYSTEM
VILLAGE OF SISTER BAY, WISCONSIN

2005 Sewer Rate Study (rev. 5) Revenue Requirement Data

	Operation & Maintenance Expenses	$286,965	
	Equipment Replacement Costs	$56,123	
	Debt Service	$43,122	
	Total		**$386,210**
Plus:	Increased Revenue Requirements due to Proposed Improvements		**$608,600**
Total Projected Revenue Requirements			**$994,810**
Net Revenue Requirement Increase as a Percentage of 2005 Revenue Requirement Data			***258%***

P:\PT\S\SISTB\050200_U P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay study\March-April 2008 Report Revisions\Chapters 1-14\Chapter 10\Tables & Figures\Used in March-April Final Draft\Excel\[Table oversizing. Paying for and carrying this oversizing cost will be one of the major challenges that the Village will face in the future. Section 10.3.4 of the report discusses various options for financing of sewer improvements. In addition to the connection fees which are currently being used, the Village should consider the possibility of implementing some type of an impact fee or trunk area charge to help pay for the pipe oversizing. An impact fee study should be conducted to determine the best way to proceed.

Chapter 62 of the Municipal Code is entitled "Utilities", and it provides rules for the village sewer and water system, and the abandonment of private wells. Again, Chapter 62 is quite comprehensive and thorough, and only a few suggested additions are recommended:

1. Section 62-7 is the same as Section 54.105 (c) – (n), so the comments indicated above for Section 54.105 also apply to Section 62-7.

2. Sections 62-9 and 62-10 discuss Sewer User Charge System and the method of paying these charges. The Capital Improvement Plan contained in this Comprehensive Utilities Plan should be utilized to estimate costs for repair and replacement of existing facilities. The future costs for repair and replacement of existing facilities can be programmed into the rate structure in advance, and the user rates adjusted so that funds can be generated to complete the repairs. Although a complete rate study is outside the current scope of work for this Comprehensive Utilities Plan, it is recommended that the Village consider performing such a rate study as a next step toward implementing the planned improvements.

10.3.6.2 Engineering Design Manual

The Engineering Design Manual contains Chapter 6 that deals with sanitary sewer issues. The purpose of this chapter is to provide guidance to the designer regarding the Village's requirements for design of sanitary sewer facilities. Comments on this document are listed below:

1. The design flows listed in Section B.6 appear to be higher than what is normally used for the type of land use that is expected in Sister Bay. It is recommended that the average flows used in the Village be consistent with Tables 9-1 and 10-3 of this Comprehensive Utilities Plan.

2. Required easement widths for sanitary sewer are listed in Section C.5 as 25 feet, whereas required easement widths in Village Ordinance Sections 54.105 and 62-7 are listed as 30 feet. The 30 foot dimension is recommended.

3. It is recommended that the minimum slopes listed in Section C.6 be changed to match the slopes listed in the Recommended Standards for Wastewater Facilities (10-States Standards), current edition.

CHAPTER 11

EXISTING STORM WATER SYSTEM

11.1 GENERAL DESCRIPTION OF SYSTEM

There are three types of existing storm water systems in the Sister Bay planning area. The rural areas have little or no defined conveyance systems. Runoff follows the natural topography. As development increases, a combination of shallow open channel swales paralleling adjacent roadways with connecting culverts at intersections carries runoff. Interspersed within these drainage areas are channels of significantly greater cross-section and occasional manhole and storm sewer pipe. Finally, in some of the areas of concentrated development, there are several underground storm sewer systems. A number of interior depressed or low gradient areas are served by storm sewer inlets.

The general location and layout of the existing storm water system is illustrated in Figure 11-1. This chapter presents a summary of the observed design and operating characteristics of the existing storm water system and components.

Most open channels are grass lined but some show erosion. Culvert pipes are a variety of materials including reinforced concrete, corrugated metal, and HDPE. Storm sewer pipes are primarily reinforced concrete and corrugated metal. Storm sewer outfalls are generally in good condition.

A unique feature of the Sister Bay system is a storm sewer lift station pump. The pump is located directly adjacent to the large wetland area between Bay Shore Drive and North Spring Road, north of Scandia Road. This wetland area has been used to store and detain storm water runoff. Excess water is pumped to an outfall on Green Bay.

The topography of the area also has numerous natural closed depressions that detain runoff. When these areas pond with water it is often viewed as a problem by landowners. Standing water is often viewed as a nuisance and if features have been constructed too low there is the potential for flood damage. If these features are upstream of other similar areas along the same drainage route, increasing the drainage efficiency will increase downstream flooding. Likewise, development of contributing areas will increase flooding problems for those downstream along the system.

A common feature of the Sister Bay region is the presence of shallow, highly fractured bedrock. The geological formation is susceptible to the formation of large voids, also known as sinkholes or karst formations. When coupled together with the closed depressions discussed above, this creates a direct path for the contamination of the drinking water aquifer from contaminated runoff. On-site private septic systems are also a major pollution concern and are discussed later in this report.

Several areas of residential development are occurring on a lot by lot basis. This type of development falls beneath the one acre threshold for state storm water regulation. As development in these areas fills in, the storm water management issues are the same as a larger residential subdivision development. But without effective storm water management planning, these issues can become problems. In some planning area locations, the natural drainage areas have been obstructed by homes, structures or other landscaping. Eventually, flooding of these areas could create significant problems for the Village and surrounding areas.

11.2 OBSERVED AREAS OF CONCERN

As part of developing the inventory of the planning area's storm water infrastructure, a general review of existing problem areas was investigated. The following summarizes a review of the known existing storm water areas of concern within the Village and surrounding planning area. The locations of these problem areas are indicated in Figures 11-2, 11-3 and 11-4.

11.2.1 Location #1: Area Bounded by Country Lane, Fieldcrest Road, and South Bay Shore Drive (STH 42)

The area of the Village of Sister Bay bounded by Country Lane, Fieldcrest Road, and South Bay Shore Drive is area is poised to experience significant land use changes as residential subdivision and other development of the land is currently occurring. Storm water drainage collected from this area currently flows to the north under South Bay Shore Drive through twin 30 inch diameter culverts. The culverts appear to be undersized, which may be the cause for localized flooding upstream of the culverts. A 3 foot deep, low gradient ditch has been constructed upstream of the culverts as a means to improve the flows to culverts. However, visual evidence of road overtopping has been observed.

As the storm water runoff water flows under South Bay Shore Drive, it follows a series of ditches, culverts, and a pond as it passes through the Bay Ridge Golf Course, then along Little Sister Road until the water flow eventually discharges into a depression formed by a beach ridge near the shore of Sister Bay. There are surface erosion and flooding events below the golf course.

The entire catchment discharges to a closed depression on private property that is ready for potential development. Increased runoff from upstream developed areas may cause increased flooding and erosion to this private property. As lots are sold and houses and other structures built upstream, the storm water problems in this area can be expected to worsen. A policy needs to be developed for dealing with this storm water issue.

Other concerns in this area include the presence of sink holes and properties served by private wells and on-site sewage disposal systems.

11.2.2 Location #2: Westwood, Woodland, and Forest Road Area

This Village area drains to closed depressions formed between the beach ridge near the shore of Sister Bay and the bluffs similar to the area described above. There are similar concerns with continued conveyance of upstream runoff to outfalls to private property.

11.2.3 Location #3: Sunnyside, Admiral, and Sunny Road Area

This area is experiencing a rapid increase in single family home construction. Driveways currently exist without culverts, ditches are present without outlets, and homes have been constructed within the natural drainage ways.

11.2.4 Location #4: STH 42 corridor from Fieldcrest to Gateway

The property along STH 42 in this area is a growing commercial corridor, with a limited defined storm water drainage system and controls. Although individual sites will restrict their storm water runoff discharge rates and volumes, this area as a whole will be impacted by the overall increased volume of storm water as impervious surfaces increase with development.

In addition, contamination of groundwater is a concern due to proximity of the fractured bedrock to the ground surface. Current drainage patterns indicate infiltration maybe prevalent in the area. A regional system with a planed collection and conveyance system may be appropriate for this area to protect groundwater quality and control storm water quantity as development of the corridor increases.

11.2.5 Location #5: Gateway Drive

The drainage upstream of Gateway Drive collects some of the STH 42 commercial corridor storm water flow and all of the flow from the upstream development. The existing conveyance system alternates between pipes and ditches. The pipe sizes under STH 57 have been recently increased during DOT roadway reconstruction and are presumed adequate for the flows to be conveyed.

Development to the west of this area raises concerns over the adequacy of pipe sizes and continued erosion in the ditches along Gateway Drive as well as South Bay Shore Drive. The upper basin has a storm sewer pipe network that is in deteriorating condition that will need replacement.

11.2.6 Location #6: STH 42 North of Harbor Shores

The existing ditch system along STH 42 north of Harbor Shores has been paved in an apparent effort to limit surface erosion. A portion of this paved area was observed to be failing and new areas of erosion are evident. There appears to be available space in this location for structural storm water controls.

11.2.7 Location #7: Storm Sewer Lift Station Pump

As noted above, the wetland area north of Scandia Road and east of Bay Shore Drive is served by a small storm water lift station. The station pump was originally installed to remove excess water from the wetland and pump it west to an outfall structure on the shore of Green Bay. Village Utility staff have reported that there are maintenance and operation cost issues with this pump system. The equipment is old and appears to be near the end of its useful life. In addition, the pump uses an oil drip feed system for lubrication, and frequently traces of the pump oil can be observed being discharged from the storm sewer outfall pipe creating a source of pollution.

11.2.8 Location #8: Ponds and Beach Ridge/Bluff Depressions East of Downtown (between downtown and the bluffs)

These Village areas absorb storm water runoff to a certain extent, but do become full or saturated and then cause localized flooding. The frequency of this localized flooding occurrence has increased with development of the area. The area may also be accumulating depositions of silt.

It appears that this area has been filled in over time to increase developable land in the downtown area. These depressions may have some water quality benefits since they may act like Best Management Practices. The slope to the Sister Bay shoreline is low gradient from these depressions. Few direct connections currently even exist between the bay and the depressions. Improving the direct connections may be able to reduce localized flooding, but the runoff water discharging into the bay may be of poorer quality.

CHAPTER 12

STORM WATER MANAGEMENT EVALUATION

The final component of the Sister Bay Comprehensive Utilities Plan was the evaluation of storm water runoff and drainage within the planning area, and the evaluation of the existing storm water system and performing a deficiency analysis. This chapter summarizes the findings from this evaluation.

12.1 GENERAL

Throughout this chapter several key terms and phrases are used to identify or describe important storm water management activities, facilities or natural features. The following terms/phrases are defined for use in this section:

- **Best Management Practices (BMP):** Structural storm water management practices such as detention or retention basins, swales, bio-retention, infiltration, etc. that mitigate peak flows and/or attenuate pollutants.
- **Natural Closed Depression (NCD or basin):** Naturally occurring hollow that is defined by concentric 2 foot closed contours.
- **Storm Water Storage Zone:** A proposed zoning unit for the horizontal area extent of the flooded area in storm water storage areas – most often natural closed depressions - during the 100-year event. It is defined by the Flood Protection Elevation.
- **Flood Protection Elevation:** The elevation of flooding plus a defined freeboard height in a Storm water Storage Zone during the 100-year event.
- **Watershed:** The total contributing area of the landscape directing storm water runoff to a point of interest or outfall. All Study Area watersheds have been given individual identification numbers (e.g., #xx00).
- **Subwatershed:** A subset of a watershed contributing runoff to a feature of interest such as a natural closed depression, a storm water inlet, culvert, etc. Subwatershed basins are labeled using a sub-series of numbers of the associated watershed (e.g., #xx10, #xx11).
- **Dry Detention Basin:** A manmade basin that has the characteristics of holding a water volume with an outlet control such as a pipe or weir. The outlet control is typically at or near the bottom elevation of the basin allowing it to drain dry between rainfall events.
- **Wet Retention Pond:** Similar to a dry detention basin except the outlet control is higher and the basin holds water to promote deposition of sediment and associated pollutants on the pond bottom in the still waters held back by the outlet. Detention and retention are often used interchangeably - wet vs. dry is the critical difference in how the two differ.

12.2 STORM WATER STUDY METHODOLOGY

A computer model of the planning area was created to assist with the hydrologic and hydraulic analysis of the storm sewer and storm water drainage/runoff systems. The model was developed based on the planning area data provided by the Village and from readily available information sources. In addition, existing data was supplemented by field investigations performed over a five day period in

November 2005. The existing storm sewer infrastructure is limited to a very small portion of the overall planning area. The vast majority of the planning area has no distinct or identifiable drainage system.

The topography for the Sister Bay study planning area is unusually complex with numerous naturally closed depressions capturing runoff. The planning area geology is conducive to the formation of sinkholes and karst. A more typical ground surface landscape would contain obvious concentrated flow areas and 7.5 minute USGS quadrangle maps would show first order intermittent (dashed blue) streams.

The natural drainage system of the study planning area is poorly developed. There is a lack of well-defined channels; the storm water runoff is captured upslope in natural closed depressions or basins.

Many of the significant NCDs represent storm water outfalls. For every basin there exists a rainstorm (unlikely as it may seem) that can cause flooding or overtopping. But for the normal range of rainfall events up to and including the 100-year or 1 percent chance occurrence in any one year, it is anticipated that many of the NCDs in the study area will contain the runoff that reaches them. Therefore, for the purpose of this study, these NCDs are outfalls. They replace the lake or stream surface water that is usually associated with the final destination of surface water runoff.

The significance of NCDs as outfalls is in the perspective of how they are viewed. Instead of viewing them simply as low points or temporary holding areas in the topography that are normally dry, as outfalls they are the ultimate receiving point for surface runoff. This is how they normally function; the natural final destination of the runoff is to the Natural Closed Depressions. It is imperative that this function be allowed to continue. The alternative is to pump and convey a large portion of the study area runoff to Green Bay. As noted above, the topography of this area does not drain to Green Bay. Instead it naturally rises and falls. Existing development often lies along the best drainage route. To install a gravity drainage system would require deep trenches well into the bedrock at significant cost. Minimizing depth would require piping constructed across parcels which would create the need for easements and easement purchase costs. While this may be the best practical alternative if conveyance is necessary (as in the case of sanitary sewer collection). it is not the least cost alternative when compared to utilizing the storm water collection and storage system that naturally exists.

In the hydrologic cycle, the NCDs are collectors of surface runoff. Under predevelopment conditions, the collected water either infiltrated into the ground water or evapo-transpired. After development occurred, area roads are often found adjacent to these basins. The possibility for culverts or ditches to drain the basins is introduced incidentally to roadway construction. There are several storm water routing possibilities at road crossings adjacent to significant closed depressions. These include:

- Pure surface storage - storm water collected in a basin will pond on the surface. Infiltration rate is low and the water is eliminated mainly through evapo-transpiration.
- Drain off through culvert - storm water collected in basin discharges across roadway through existing culvert.
- Drain off down road ditch – storm water collected in basin reaches ponding depth allowing some water to drain away by gravity along the ditch (provided road ditch profile is not in a deep sag curve).
- Seepage into underground aquifer - storm water infiltrates into the ground through soil voids and rock fractures and discharges into dolomite aquifer.

To implement specific storm water infrastructure it will be necessary to conduct a detailed site survey to determine which of these situations may be occurring. A complete culvert survey will accurately determine the runoff routing for each watershed. It will determine if runoff is diverted by roadside swales or ditches to a different drainage system. A detailed survey will also accurately establish the available storage volume, elevation of overtopping or escape route of excess runoff water, and the elevation of existing development.

For the purpose of this study, the probable drainage system routes were established using GIS maps. The GIS maps included topographic features, contour lines, and point elevations. Engineering judgment was used to establish the connectivity of adjacent subwatersheds across roadways and the demarcation of significant storage basins.

12.3 INFRASTRUCTURE DATA AND FIELD OBSERVATIONS

Detailed infrastructure information on the existing storm sewer system was obtained from record drawings of the recent East Mill Road and South Bay Shore Drive project (2005); Bluff View to Green Bay drainage system project (1981); and the lift station project (1973) that lies within Watershed #2700, and the storm system draining NCD found in Watershed #6300 (1973).

Input data for the storm water computer models was based primarily on information available from Door County and Village of Sister Bay. Construction/as-built drawings were available only for some areas, which limited the accuracy of the models in the remaining study areas. However, using knowledge of specific areas prone to flooding, the modeling results do provide insight into targeting solutions to the storm water concerns.

Existing GIS-based data was available for planning area wetlands, soils, parcels, topographic elevations (2 foot contours), land use/zoning, and roads, along with digital ortho photos from Door County. The Village of Sister Bay provided storm sewer record drawing information where available.

12.3.1 Data Collection Methods

Field data was collected by using a hand-held Trimble GPS unit that included a data collector for hand entry of system characteristics. System characteristics collected included pipe or channel dimensions and depth, flow direction, structure type, condition, material type, etc. Following collection of this information, the data was downloaded to a computer and was integrated into the GIS attribute database for each feature.

Field information was collected on the three primary storm water drainage features existing in the planning area, including storm sewer piping, manholes, inlets/catch basins, culverts and roadside drainage ditches and channels.

12.4 STORM WATER INFORMATION DEVELOPMENT AND ANALYSIS

Hydrologic analysis is used to determine peak runoff and volumes from infrequent storm events. Key parameters used in the analysis are land cover, soil types, topography, and drainage systems. Watershed boundaries are defined by the topography that drains runoff to a point of interest. Figure 12-1 is a Study Area planning map showing all of the watersheds down to the delineated subwatershed level. Planning area GIS elevation contour maps were used to define drainage subwatersheds. It should be noted that 2 foot contour intervals are limited to an accuracy of plus or minus half the contour interval, or in this case, 1 foot. In turn, all calculations based upon the contours have a degree of uncertainty. Although adequate for a planning level study, a more accurate base map should be developed for analysis and design of projects. Soil map information was combined with the sub-basin boundaries and land cover information to determine runoff characteristics.

XP-SWMM, version 9.14 was used to model the water hydrologic and hydraulic characteristics for rainfall events of the 2, 10, and 100-year return intervals. The model evaluated storm sewer and channel capacity, storage, and discharge values.

ESRI ArcMap GIS computer software was used to process information found in the public domain from NRCS, Village of Sister Bay, and Door County. This software creates GIS features and uses the attributes to generate the data used in the analysis of both the water quantity and quality modeling.

The second step was to convert the zoning codes to a corresponding land use code. Land use is keyed to both water quantity and quality runoff. The SLAMM Standard Land Use (SLU) Codes are more detailed than the zoning codes. The SLU codes were selected and entered into the GIS tables and used to generate the parameter values used in both types of analyses. . To complete the conversion from zoning code to SLU code, the ortho photos were viewed and used to assign specific SLU codes to each zoning/land use area.

The third step was to create a GIS map of SLU areas that differentiated areas that were currently undeveloped vs. potential development. This information was used in the H&H analysis for the development of the curve number (CN) used in the SCS (NRCS) hydrologic method. These features were also used to develop output files for the SLAMM computer model.

The modeling software employed to determine runoff allows the choice of entering either a composite CN that includes the impervious area, or to enter the percentage of imperviousness and a CN for the remaining areas. In this effort, the latter method was chosen. Lacking information on specific land cover, it was decided to choose a pervious open space CN from the range of available CN's. This range, where categorized by soil Hydrologic Soil Group (HSG), was relatively narrow for the typical range.

The percentage of impervious surface was determined starting with the typical values given by NRCS and using engineering judgment to adjust these values according to evidence found in the ortho-photos for the Sister Bay study area. The NRCS land use description is keyed to the SLU code and determines the percent imperviousness. The hydrologic soil group determined the CN.

The time of concentration was developed from watershed sub-basin specific route calculations or engineering judgment.

In some cases, CN was adjusted to account for available storage areas. The topography of the study area is marked by a high number of closed depressions that function as hydrologic storage or detention areas. Watershed sub-basins with significant closed depression areas were modeled as storage. Most computer models do not account for soil infiltration within these depressions. This is useful because it provides discharge values that would occur without infiltration. As discussed later, it would be beneficial to prevent infiltration to avoid aquifer contamination. If this is the case, then model result discharge values can be used as the initial basis for sizing storm sewer system improvements.

It should be noted that hydrologic analysis is an inexact science. There are several different methodologies recognized in the industry as valid for use in this type of study. The SCS or NRCS method (TR-55) used for this project is used most often. When comparing results from different methods or in selecting equally valid parameters using engineering judgment, the results obtained can vary. This should be kept in mind when comparing study results with work done by others that used more detailed information. The value of a hydrologic study of this level is in the conceptual planning level information it provides. The results of this study identify issues or areas requiring more detailed review, and analysis. The study findings and recommendations provide a basis for planning and implementing future projects.

12.5 FUTURE DEVELOPED CONDITION MODELS

The developed condition model can effectively define the increase or change in runoff rates and volumes from a given area for a given storm recurrence interval. However, it is not practical to design storm water management systems for hypothetical developments. The future condition models provide information demonstrating the change in runoff that would occur at the same point of interest without mitigating practices.

The existing condition land use includes relatively large areas of sparsely developed or semi-rural areas. Parcel maps indicate that outside of the already intensely developed areas, residential lots are relatively large. For the purpose of this storm water study, the direction given by Sister Bay for future development patterns in these areas is the same as is used for the water and sanitary sewer studies to project the density of future residential development.

12.6 HYDRAULIC AND HYDROLOGIC (H&H) ANALYSIS

12.6.1 Objectives

The purpose of the analysis is to provide planning level guidance for specific storm water conveyance improvements to relieve flooding, to provide a process for guiding future development to prevent increased flooding to downstream areas, and to develop strategies for improving water quality in Green Bay to prevent beach closures and promote tourism.

Some of these objectives can often be in conflict. If an upland landowner wants to reduce ponding on their property, they are inclined to look for a way to drain water off faster. This often promotes the delivery of pollutants and water volume rates downstream to other landowners. As is well known, new development typically converts soft absorptive land covers into hard impervious surfaces. The result is more storm water runoff and increased chances of flooding.

Another challenge for the Sister Bay region is the shallow dolomite bedrock and associated karst. Its presence increases the cost of storm sewer construction, and can either physically prevent infiltration in some areas, or accelerate it through fractures in other areas and potentially contaminate the aquifer.

12.6.2 Planning Level Recommendations

In the discussion below each watershed is described, the results of the analysis reported, and recommendations specific to addressing watershed issues are presented. The planning level recommendations provide background for a better understanding of the specific recommendations for each watershed.

In general, existing storage areas within the planning boundary are recommended to be preserved for flood storage. No further development should be allowed within the Storm Water Storage Zones. Preservation of the storage areas is recommended for two primary reasons: flood control and protection of surface waters from storm water pollutants. First, modifying a drainage system can lead to conflict between the landowner properties draining and those receiving storm water. Often those receiving are likely to be in a poor position to accommodate the change in runoff. Wisconsin water law in this regard is based on precedents set in case law. It uses the concept of reasonable use. As such, each case becomes unique. To rely on the courts to settle disputes arising from changes in the natural drainage system would result in chaos, discourage development, and lower land values.

The second reason to preserve natural closed depressions is to trap pollutants and prevent their transport to Green Bay. The water along the area beaches has already experienced degradation which would be exacerbated by increasing developed area delivery of pollutants. Likewise, the preservation of storage detains runoff and reduces downstream flooding.

To achieve this, Sister Bay must create the authority through zoning ordinance adoption to require the preservation of natural closed depressions to be dedicated to flood storage. The ordinance should allow for the alternative to replace their function through other means. The language of the zoning ordinance should also allow for floodproofing existing structures. For the purposes of this study, the natural storage areas are referred to as **storm water storage zones**. The storm water storage zones will be bounded by the **flood protection elevation**.

The flood protection elevation as used in Wisconsin's Floodplain Management Program and NR 116 is the elevation 2 feet above the regional (100-year event) flood elevation. Floodplain zoning restricts development at the floodplain boundary. Development within the floodplain boundary if allowed at all, is required to be above the flood protection elevation or 2 feet above the floodplain elevation. The difference between the water surface and the flood protection elevation is referred to as freeboard. Sister Bay has the option of setting the criteria for the flood protection elevation at a different height. The freeboard used for the difference between the modeled 100-year ponding elevation and a buildable ground surface elevation should be set with consideration for the level of risks for potential harm to health, safety, and property.

The flood elevations provided by this study and found in Appendix C are based on approximate data. Given the level of uncertainty, the 2 foot freeboard height is recommended for the Flood Protection Elevation to set the Storm Water Storage Zones. However, it is recognized that 2 feet of freeboard in a relatively flat area can prevent development in large areas which will affect property values and tax revenue. Reduced uncertainty could lead to less freeboard. To reduce uncertainty the system should be inventoried and analyzed at a more detailed level.

This would require an on-the-ground survey of the proposed Storm Water Storage Zone to accurately define the available storage and executing the computer H&H models with the new data to accurately define the ponded elevation for the 100-year event.

An additional step to reduce uncertainty is to create an escape path for flood water to a relatively safe route such as a roadway leading to Green Bay or to designated drainage routes. Realize that the circumstances for this occurring are less than 1 percent in any given year, but increase over a longer period. If circumstances do not permit this, then the freeboard should remain at 2 feet. Another common practice is to compute the Storage Zone volume necessary to contain back-to-back 100-year runoff events.

The ponded elevations found for each of the NCDs does not consider infiltration. It is broadly understood that karst is common in this area and the working assumption should be that all NCDs overlie karst and bedrock fractures. So in the existing condition with the probability that large volumes of runoff can enter the aquifer at each of these NCDs, the modeled flood elevations will appear unreasonably high and the resulting size of the Storm Water Storage Zones unreasonably large. But as will be discussed in Chapter 13, if the drinking water aquifer is to be protected, the karst short circuit for runoff must be cut off. Prevention of ground infiltration will result in dramatic changes in ponded elevations approaching the results provided in this study. Unless site specific conditions show adequate treatment for infiltrating runoff, the choice is between flood damage protection of dwellings at the surface, or drinking water contamination in the ground. If adequate soils provide aquifer protection, then the ponded elevations are more likely to be similar to the current condition due to slow infiltration.

Another consideration is the potential for wet basements due to lateral movement of groundwater. It may not be enough to have the low adjacent grade for a dwelling be above the ponding elevation if the floor level below that is prone to water damage. A restriction on the lowest floor level with respect to the ponding level should be considered. Some flexibility for site specific knowledge of groundwater conditions and engineered solutions should be built into the ordinance.

However, property subject to development and ownership of the property encompassing the storage area do not necessarily coincide. The ordinance should contain language preventing landowners from increasing runoff volume or rates to other properties including rights of way without a formal agreement with the receivers. This protects not only those downstream of existing storage depressions, but also the owner of the storage basin from increased runoff from changes in land use and cover from contributing area land owners.

A final consideration in the designation of a Storm Water Storage Zone is that existing development can become a nonconforming structure. There will likely be opposition to creating a zone that results in some structures becoming nonconforming. Designation of the Storm Water Storage Zone will reduce the value of the property and the value of existing development. There are cases where development is so low in the NCD that there is no practical means of diverting or removing runoff without the occurrence of flooding. In such a case, it is imprudent to let further development to take place.

Model ordinances, found in Appendix D, will provide language for water quality similar to the model ordinance found in NR 152. In addition, and of interest to the discussion of this chapter, is the language requiring flood control via peak flow mitigation in all new development for the large storm events. This is discussed in greater detail in Chapter 13 of this report.

12.6.3 System Hydraulic Performance

A primary objective of this study was the performance analysis of the storm system conveyance at the 10-year event discharge and for ponds at the 100-year event discharge level. The computer models determine the runoff discharges for each contributing area and routes these through the storm water conveyance system. Flow bypass at inlets, capacity of the street curb and gutter system, localized depressions not recorded by the 2 foot contours, and perhaps most importantly, the infiltration of rainfall into the fractured bedrock are considerations not captured by the H&H models. Each watershed system should be reviewed in light of these characteristics to determine the actual impact indicated by the computer model results.

The discharge for each watershed for both the existing and future build-out condition at the 2, 10 and 100-year recurrence intervals is summarized in Appendix C of this report. The discharge values listed are for the peak discharge from the landscape and not as modified by storage areas or conveyance capacities. Also listed are the important elevations for the watersheds NCDs including; Existing Low Adjacent Grade or the elevation of the contour closest to visible structures, the maximum modeled ponded elevation of flooding in the NCD during the runoff event for the existing and future developed conditions, and the elevation of a surface feature over which ponded water can escape should it reach that elevation. Of particular note is the difference between the Low Adjacent Grade and the Escape route Elevation. The preferred condition is that the escape elevation is lower than the low adjacent grade. Should runoff event exceed the modeled volume, the ponded elevation will exceed the value given and reach a structure if there is no escape route below the dwelling's low adjacent grade elevation. A second table is included in this appendix that shows the area and elevation of the Storm Water Storage Zones as shown in the figures of each of the Watersheds discussed below.

Each watershed number refers to a corresponding number as discussed under Section 12.1 for the watersheds and sub-basins. Each watershed discussion includes a probable cost of construction and engineering of recommended improvements. Shallow rock excavation costs for the storm sewer is included at $40/lf. The costs for modifying NCDs to function as water quality BMPs are presented separately in Chapter 13. The potential costs for easements or property purchase is not included. The following summarizes the system performance analysis findings.

12.6.3.1 Watershed #900 – Meadow Wood Lane

Watershed Description. There is relatively dense development within the study area that lies within the NCD of this almost 200 acre watershed. Most of the watershed lies outside of the study area boundary and is agricultural. Figure 12-2 illustrates the location of the watershed boundaries and surface water flow direction.

Storm System Description. There is no identified storm water system in the area other than roadside ditches.

10-Year Event Storm System Analysis. Since there is no storm water system within the watershed, the 10-year event storm water system analysis was not performed.

100-Year Event Pond System Analysis. Under current development conditions, Basin #900 overtops and flows west beyond the study area boundary. The escape elevation is between 720 and 722 feet NGVD 29. The flooded elevation for the developed condition is approximately 722 feet NGVD 29.

Watershed Recommendations. The ortho photos indicate development has already occurred in the low area of the NCD, precluding the designation of a **storm water storage zone.** To construct a storm sewer would require traversing topography that drops below elevation 712 feet NGVD 29 and rises to elevation 730 feet NGVD 29 before steadily dropping to Green Bay. The developer's drainage plan should be reviewed to see how the watershed runoff was planned to be handled. If homeowners are placing fill as each home is constructed, it will raise the ponded elevation eventually the increasing the number of homes that will be flooded. Watershed 900 drains to an NCD to the west. The NCD is approximately 6 feet deep and has a high potential for karst conditions. Added runoff from Watershed 900 to the adjacent landowners has the potential to create conflict from increased flooding. The Town of Liberty Grove or Door County Zoning Administrator should be contacted to apprise them of the situation before added development exacerbates the problem.

12.6.3.2 Watershed #1000 – Beach Road South of Seaquist Road

Watershed Description. This 230 acre watershed is composed of low-density residential, agricultural and wooded land. The topography is steep with a relatively continuous positive drainage system. Development exists within the NCD along Beach Road and to the west fronting Green Bay. Figure 12-3 illustrates the location of the watershed boundaries and surface water flow direction.

Storm System Description. There is no identified storm water system in the area other than roadside ditches.

10-Year Event Storm System Analysis. Since there is no storm water system within the watershed, the 10-year event storm water system analysis was not performed.

100-Year Event Pond System Analysis. Under current development conditions, NCD #1002 overtops and flows west across Beach Road, inundating existing development and spilling across developed property. The escape elevation is between 646 and 648 feet NGVD 29. The flooded elevation for the developed condition is approximately 646.3 feet NGVD 29. Existing development is shown at or below the 644 foot elevation contour.

The above analysis does not include any infiltration. Use of detention basins in the upland areas in accordance with the proposed ordinance will maintain the present rate of runoff but will increase volume unless infiltration can be used. The existing condition runoff discharge to the NCD is 140 cfs. The existing development is too low in the basin to effectively utilize it for storage.

Watershed Recommendations. The NCD is outside of the wellhead protection zones. The extension of a municipal water supply to residents in this area will allow them to use the municipal water system instead of private wells with the potential for contamination from any karst activity within the NCD. Since the NCD is close to Green Bay, the likely direction of ground water flow is to the Bay. For this situation the best course of action is to monitor flooding during high runoff events to determine the true level of risk of flooding.

It is still recommended to designate this NCD as a **Storm Water Storage Zone** to prevent additional development in a flood prone area.

12.6.3.3 Watershed #1400-#1900-#2000 – Beach Road and STH 42 North of Wildwood Road

Watershed Description. This 150 acre watershed is composed of low density residential, agricultural and wooded land. Watersheds #1400, #1900, and #2000 were analyzed together as the H&H analysis results revealed that they were connected. A natural closed depression straddles Wildwood Road at the western (lower) end of the watershed. Parcel mapping shows that the watershed is extensively subdivided in Watersheds #1400 and #1900. Figure 12-4 illustrates the location of the watershed boundaries and surface water flow direction.

Storm System Description. There is no storm water system in the area other than roadside ditches.

10-Year Event Storm System Analysis. Since there is no storm water system within the watershed, the 10-year event storm water system analysis was not performed. However, the natural closed depressions on either side of Wildwood Road were modeled separately to determine if Basin #1900 overtopped.

Under current conditions Basin #1900 (natural closed depression) appears to reach the approximate road centerline estimated at approximate elevation 637 feet NGVD 29. Storm water runoff overtopping Wildwood Road is a possibility. The peak 10-year water level within Basin #2000 reaches elevation 629.2 feet NGVD 29 and surface runoff is contained within the basin.

100-Year Event Pond System Analysis. Under current development conditions, Basin #1900 overtops and flows into Basin #2000 to the south. Basin #1900 floods to 637.3 feet NGVD 29 or 0.2 feet above the approximate road centerline elevation. During the 100-year rainfall event, water in Basin #2000 reaches elevation 632 feet NGVD 29, which is approximately 8 feet below the crest. The topographic elevation contour near the only structure shown (Mary Smith) on the ortho-photo is 642 feet NGVD 29, or 10 feet above the modeled flood elevation.

Watershed Recommendations. Designate both of the natural closed depressions as **storm water storage zones** with the boundaries set at the future conditions flood protection elevation. The future condition flooded elevation is approximately 637.5 feet NGVD 29 for Basin #1900, and 634 feet NGVD 29 for Basin #2000. More detailed analysis should be developed prior to setting zoning boundaries.

12.6.3.4 Watershed #1500 Muffin Road Cul de Sac

Watershed Description. This 43 acre watershed is currently includes light residential development with some orchards. It is projected to be developed up to a medium density residential. Current development at the west end of the Muffin Road cul-de-sac shows a home constructed in one of the lowest parts of the watershed in an NCD. The parcel is owned by Laura Rush. **Based upon the available data, this home is at a high risk for flooding**. Even the existing condition 10 year event floods the NCD to elevation 684 feet NGVD 29. Figure 12-5 illustrates the location of the watershed boundaries and surface water flow direction.

Storm System Description. There is no storm sewer system.

10-Year Event Storm System Analysis. There is no storm water system within the watershed (other than roadside ditches). Therefore, the 10-year event storm water system analysis was not performed.

100-Year Event Pond System Analysis. The NCD ponded depth is elevation 685 feet NGVD 29. The 684 foot contour surrounds the Rush property structure. Based upon the 2 foot contours the surrounding surface drainage elevation is at or above 688 feet NGVD 29. The likely surface water runoff route is north along Beechwood east to Green Road. The 690 foot elevation contour crosses Muffin Road to the east.

Watershed Recommendations. Unless the affected structure noted above can be flood-proofed, there is minimal benefit in creating a Storm Water Storage Zone. Development on surrounding lots will doubtless further encroach upon the NCD. The area should be investigated to determine the true risks of flood damage. This NCD is not suitable as a **storm water storage zone**. An area developer's drainage plan should be reviewed to see how the watershed runoff will be handled. If ponding does not occur, there is a greater than usual likelihood of karst short-circuiting the drainage directly to the groundwater. A parcel owned by Douglas and Merisue Haas is a possibility for a storage basin, but given the bedrock conditions the outlook is not favorable.

Without some means of detention, a storm water system capable of handling the future condition 100-year event discharge of 52 cfs is needed to prevent runoff from entering the aquifer or flooding homes. If homeowners are placing fill as each home is constructed, eventually the ponded elevation will rise flooding more homes. This area is a concern for future potential legal action regarding flooding. The Town of Liberty Grove or Door County Zoning Administrator should be contacted to apprise them of the situation before added development worsens the problem.

12.6.3.5 Watershed #1600

Watershed Description. This 70 acre watershed contains primarily agricultural land uses with some light residential development. A large NCD lies in the middle of the watershed. Future development is projected to be medium density residential. Figure 12-6 illustrates the location of the watershed boundaries and surface water flow direction.

Storm System Description. There is no storm sewer system.

10-Year Event Storm System Analysis. There is no storm water system within the watershed (other than roadside ditches). Therefore, the 10-year event storm water system analysis was not performed.

100-Year Event Pond System Analysis. The watershed's NCD has no development within its boundaries. The future development ponded elevation is 694.3 feet NGVD 29. The upper boundary of the NCD is at elevation 696 feet and would be an appropriate Flood Protection Elevation. It encompasses 12 acres while the 694 foot elevation contour is 6 acres.

Watershed Recommendations. To maximize developable area, some grading can modify grades to the extent necessary to make a practical **storm water storage zone**. Zoning action should be accomplished as soon as possible to prevent a situation similar to Watershed #1500.

12.6.3.6 Watershed #2300 – Beach Road and STH 42 North of Waters End Road

Watershed Description. This 20 acre watershed is largely wooded and subdivided into parcels that typically range in area from ¾ to 2 acres in size. About 40 percent of the area is developed, primarily with single family dwellings. A significant natural closed depression lies within the watershed that prohibits runoff from entering surface waters. No wetland areas have been previously mapped. The majority of the depression is labeled on the area soil maps as containing sandy soil. It is highly likely that runoff infiltrates to the groundwater at this location.

It was reported that there are no significant culverts crossing STH 42. Therefore, it was determined that area surface runoff from Watershed #2600 runs downhill along STH 42, and crosses Waters End Road

Figure 12-7 illustrates the location of the watershed boundaries and surface water flow direction.

Storm System Description. There is no storm water system within the watershed (other than roadside ditches). Therefore, the 10-year event storm water system analysis was not performed.

100-Year Event Pond System Analysis. The lowest existing adjacent dwelling appears to belong to George and Eileen True and has a parcel with a residence at or above elevation 632 feet NGVD 29. The natural detention area ponds up to 623 feet NGVD 29 which provides almost 9 feet of freeboard during the existing condition 100-Year rainfall event.

Watershed Recommendations. Designate the natural closed depression as a **storm water storage zone** with the boundary set at the future conditions flood protection elevation. The future condition flooded elevation is approximately 625 feet NGVD 29.

12.6.3.7 Watershed #2500 – Beach Road and STH 42 North of Waters End Road

Watershed Description. This 11 acre watershed is largely wooded and subdivided into 5 parcels that vary widely in size. It is mostly developed with commercial and residential land uses. A significant natural closed depression lies within the watershed that prohibits runoff from entering surface waters. No wetland areas have been previously mapped. Figure 12-8 illustrates the location of the watershed boundaries and surface water flow direction.

Storm System Description. There is no storm water system within the watershed (other than roadside ditches). Therefore, the 10-year event storm water system analysis was not performed.

100-Year Event Pond System Analysis. The lowest existing adjacent dwellings appear to belong to Bruce and Carol Bell and Peter Jr. and Janet Peterson on parcels with a residence below elevation 590 feet NGVD 29. The NCD ponds up to 589 feet NGVD 29 during both the existing and future conditions 100-year rainfall event. There is little or no freeboard and the difference is within the tolerances of the contour mapping.

Watershed Recommendations. Designate the natural closed depression as a **storm water storage zone** with the boundary set at the future conditions flood protection elevation. Encourage residents to flood proof structures to the flood protection elevation. Given the proximity of Green Bay to the NCD, the possibility of adding an outlet and pipe to Green Bay to control flooding should be explored. A detailed study and investigation should explore this and/or if it is feasible to enlarge and deepen the storage volume of the NCD.

12.6.3.8 Watershed #2600 – Waters End

Watershed Description. The watershed land use is a combination of commercial and low density residential. No natural closed depressions exist. Figure 12-9 illustrates the location of the watershed boundaries and surface water flow direction.

Storm System Description. The storm water system consists of a roadside ditch, culverts along the lowest part of STH 42 with pipe and manhole leading down to Green Bay from STH 42. These channels are in fair-to-poor condition showing signs of erosion.

10-Year Event Storm System Analysis. The storm system capacity is exceeded by the 10-year discharge.

100-Year Event Pond System Analysis. There are no ponds within the watershed to analyze.

Watershed Recommendations. The contributing area north of Waters End should be diverted into a new conveyance down Waters End Road to Green Bay. This will relieve the existing system south of Waters End Road. The inlet on the east corner of the Waters End-STH 42 intersection should be situated in a sump condition to effectively capture storm water runoff as it travels from the northeast along STH 42.

An analysis of the remaining runoff peak flow velocities is recommended to determine the type of lining for the channel, followed by channel reconstruction. If at all feasible, a grassed swale should be constructed to act as a pollutant removal BMP.

Public Capital Improvement Cost Estimate

Diversion Storm Sewer $125,000

12.6.3.9 Watershed #2700 – Storm Water Wetland Lift Station

Storm System Description. This watershed is without a storm system in the upland areas of the watershed. Storm water drains to a wetland lying between North Bay Shore Drive, North Spring Road, Scandia Road, and Hill Road. The water outlet used for this wetland area is a Village-owned lift station. No information was available on the existing pump performance capabilities. The existing lift station pump is beyond its useful life and is recommended to be replaced. The pump discharge capacity has been reported to be acceptable.

Figure 12-10 illustrates the location of the watershed boundaries and surface water flow direction.

10-Year Event Storm System Analysis. The existing storm system is limited to the outlet side of the lift station pump. There is no storm water system within the upland area of the watershed. The pump discharges to a system of storm sewer piping with the outfall on the Green Bay shoreline. The first storm sewer pipe segment has the flattest slope, and limits the total flow that can be delivered to the outfall. This segment is a 30 inch CMP at 0.1 percent slope, which has a gravity flow capacity of 14.4 cfs, well in excess of the capacity of most typical storm water lift station pumps.

100-Year Event Pond System Analysis. Without the storm water lift station operating, water in the wetland area would reach an elevation of approximately 587 feet NGVD 29 during the 100-year rainfall event, which would cause flooding problems for adjacent homes. The ortho photo shows trees obscuring many dwellings, but it appears that several homes including Jeanette Tveten, John and Janet Sharp, and James and Lois Pellegrini have low adjacent grades below the 584 foot elevation. Flooding in the area has not been reported to be a current concern. The operation of the lift station and/or downstream storm sewer system are apparently adequate for the existing condition and for the rainfall events experienced since the lift station was installed. It has been noted that areas along North Spring Road have experienced flooding in the past due to high runoff rates from the bluffs to the east. A single culvert reportedly exists that limits the effectiveness of storm water runoff to drain from the east side of North Spring Road.

However, the modeling for the 100 year existing condition shows that in order to maintain an existing conditions ponded elevation at or below 583 (no freeboard) a discharge of 100 cfs (45,000 gpm) is necessary. This is far in excess of any lift station pump. Since the lift station has been effective at preventing flooding to-date, there is very likely an alternative discharge route such as diversion into a karst feature. To maintain a future conditions ponded elevation at or below 583 feet NGVD 29 (no freeboard) a discharge of 120 cfs is necessary during the 100 year event.

Even when disregarding Subwatersheds #2701 and #2702, Subwatershed #2703 alone will discharge 246 cfs at peak flow and raise the ponded elevation to above 586 feet NGVD 29 with no outlet. This demonstrates that even with upslope detention or diversion, the NCD capacity and outlet conditions pose a serious threat of flooding.

Watershed Recommendations. Remove and replace the existing lift station pump with a new pump. Improve conveyance of storm water across North Spring Road. This will require a detailed analysis of alternatives for capturing runoff and directing it to the existing culvert, or the installation of addition culverts under North Spring Road. As noted in the discussion below for Watersheds #2900 and #3000, runoff from #2900 and #3000 may be contributing to the problem. If as suggested below, a storm sewer draining the #2900 NCD is connected to the #2700 storm sewer piping, the capacity of the total drainage system (both pipe and surface channels) should be confirmed. Analysis to determine if the new pump is sufficient to handle the added water brought in by improving drainage across North Spring Road is recommended.

The necessary discharge capacity (100 cfs) of the lift station is impractically high for a pump if it is the only outlet for the 2703 NCD. It would be need to be even higher if Scandia Road is diverting the other watersheds to it. The 2703 NCD lies within the primary Wellhead Protection Zone. If substantial or significant discharge is occurring through infiltration and karst drainage it may be contaminating the aquifer. Well 1 is within a few hundred feet of the lowest area of the NCD. Monitoring of this well is vital given the apparent direct connection between the runoff entering the NCD and the unexplained outlet for much of the water entering it. The best course of action may be to continue to allow runoff to outlet from the wetland in this manner until it is determined to be a source of contamination. If contamination becomes an issue, the alternatives are to find and seal the karst voids and fractures or abandon the well. If the karst outlet is successfully sealed, the development properties are at risk of flooding during large events. As noted earlier, a lift station pump is not a practical solution since the pump's capacity would need to be at least 100 cfs.

Since pumping is not a practical alternative, a surface outlet alternative has been modeled. Such an alternative will require the shortest possible route to Green Bay. If the flood protection elevation must be held at 583 feet NGVD 29 (little or no freeboard) to protect existing development, there is really no practical way to drain water to the Bay when the Lake Michigan water level is at historically high levels (582.3 feet IGLD). Even when normal water levels exist, a surface drainage outlet with a crest at 582.5 feet (0.5 feet above the lowest contour and 0.5 feet below the assumed low adjacent grade) would need to be at least 150 feet wide. There is no undeveloped corridor to Green Bay for such a channel. The difference in datums (IGLD and NGVD 29) is discussed in the summary.

There is little opportunity to detain or divert the runoff from Subwatershed #2703 due to the existing topography. Even if a system of storm sewers were to be installed, the overland flow during a large event would bypass the piped system and follow the ground surface to the NCD. It still is best to divert runoff from Subwatersheds #2701 and #2702 to the extent possible.

Subwatershed #2702 is approximately 100 acres of what is projected to become low and medium density land use. Generally 5 percent of the contributing area is needed for a detention basin of sufficient size (5 acres) to control flooding. The logical location for such a basin is on the property of Edgar and Nancy Hillner just north of Hillcrest Lane.

Subwatershed #2701 is largely outside of the study area. But it has similar opportunities that can be implemented when developed.

Public Capital Improvement Cost Estimates

Culverts for North Spring Road	$16,000

New Lift Station Pump	$30,000
Subwatershed #2702 Retention Basin	$950,000
Replace Lift Station Storm Sewer Outlet	$200,000
Total	**$1,196,000**

12.6.3.10 Watershed #2800 – Harbor Shores

Watershed Description. The land use within the watershed is primarily commercial with approximately 25 percent low density residential. No natural closed depressions exist within the watershed. There is approximately 1,000 feet of storm sewer pipe along North Bay Shore Drive terminating in an outfall discharging into Green Bay. Figure 12-11 illustrates the location of the watershed boundaries and surface water flow direction.

Storm System Description. The existing storm sewer system along North Bay Shore Drive (STH 42) is predominantly 18 inch diameter concrete pipe with manholes that connects to a 24 inch diameter outfall pipe.

10-Year Event Storm System Analysis. Based on the hydraulic modeling completed, and the area of contribution defined by the 2 foot elevation contour plans, the existing pipe system does not have sufficient capacity for the anticipated 10-year rainfall event discharge. The proposed or fully developed condition storm discharge increases from 52 to 58 cfs.

100-Year Event Pond System Analysis. There are no ponds within the watershed to analyze.

Watershed Recommendations. With regard to the storm sewer pipe system, if there is a swale in addition to the pipe system to carry runoff, it should be sufficient to carry the runoff in excess of the pipe capacity parallel to the road. If there is no swale, it is recommended that a more detailed study be undertaken to determine the changes to the pipe size necessary to adequately convey storm water. A review of current problems (erosion, flooding) should be made before implementing any infrastructure modifications. Should it become necessary to increase the size of the pipe, the 30 inch pipe leading down to the outfall should be increased to a 36 inch diameter, while the pipes leading up to it should be 30 inch with the upstream pipe at 24 inch diameter.

Public Capital Improvement Cost Estimate

Increase Pipe Sizes	$100,000

12.6.3.11 Watershed #2900 – Scandia Road and Sunset Road

Watershed Description. The land use within this 85 acre watershed is a mixture of light to medium residential and agricultural, with some limited multifamily residential. It is largely wooded with steep topography. A 4+ acre NCD straddles Scandia and Sunset Roads. The NCD intersects as many as a dozen parcels most of them already developed. It is likely that roadside drainage along the east-west roads drains the NCD before it causes any flood issues, although a very large storm may cause some flooding. Figure 12-12 illustrates the location of the watershed boundaries and surface water flow direction.

Storm System Description. None.

10-Year Event Storm System Analysis. There is no storm water system within the watershed (other than roadside ditches). Therefore, the 10-year event storm water system analysis was not performed.

100-Year Event Pond System Analysis. The existing NCD has minimal volume and depth and is not significant enough to consider as storage. No pond analysis was performed.

Watershed Recommendations. Since flooding has not been noted as an issue in the area, the current conditions are anticipated to be adequate. However, based on modeling results, the future conditions discharge is expected to increase by 50 percent. It is recommended that this area be given further detailed study to determine a feasible means of routing of runoff and to observe conditions in the area of the NCD during a large rainfall event. It may become necessary to install piping in this area connecting to the storm sewer system draining from the lift station. Flooding along North Spring Road may be partially related to the discharge from the #2900 NCD.

The future conditions 10 year peak discharge for this watershed is 27 cfs, requiring a 30 inch RCP pipe for a 0.5 percent slope. This will exceed the capacity of the downstream system which is about half this and must also carry the lift station discharge. Sunset Drive provides a lower escape route elevation than Scandia Road, but the route would require water running through more developed area to reach it. If Scandia Road can be modified so that the roadside ditches maintain a maximum elevation of 616.0 feet NGVD 29 it would help to divert large runoff events.

There is a location in the upland area for a detention basin on the property of Rodger Swenson that would mitigate development from 30-40 percent of the 85 acre watershed. A 2 acre basin should be an adequate area for minimizing increased runoff. But the lower part of the watershed with the steeper topography remains unmanageable with the level of existing development cutting off options for detention and storage.

This NCD is not suitable for designating as a **storm water storage zone**.

Public Capital Improvement Cost Estimates

30 inch Storm Sewer	$170,000
Retention Pond	$360,000
Total	**$530,000**

12.6.3.12 Watershed #3000 – East Scandia Road

Watershed Description. The land use within this 83 acre watershed is a mixture of light to medium residential, agricultural, and institutional. The non-agricultural area is largely wooded with steep topography. A 6 acre NCD contains the runoff without surface discharge. Figure 12-13 illustrates the location of the watershed boundaries and surface water flow direction.

Storm System Description. None.

10-Year Event Storm System Analysis. There is no storm water system within the watershed (other than roadside ditches). Therefore, the 10-year event storm water system analysis was not performed.

100-Year Event Pond System Analysis. The future conditions NCD ponded depth is 631 feet NGVD 29. The existing conditions ponded depth is 630 feet NGVD 29. A dwelling within the parcel owned by

James and Kristine Johnson lies on the 630 foot elevation contour, below the future flooded elevation. Several other parcels without visible structures are within the flood elevation contour and are apparently owned by the Johnsons as well.

Watershed Recommendations. Since flooding has not been noted as an issue, the current condition is anticipated to be adequate. There are approximately 22 acres of agricultural land that could be open to development. Implementation of flood ordinance protection can prevent increases in peak runoff but does little to control the increased volume if infiltration is not a feasible BMP.

Once again as in Watershed #2900, Scandia Road may actually divert much of the runoff from the contributing area to the west. If this is the case, the NCD is ineffective in terms of storing runoff.

Although there are challenges due to the presence of a dwelling and directing the runoff to the NCD, it is recommended that this NCD be utilized for a **storm water storage zone** if the Johnson's are agreeable. It is also recommended that this location be investigated for use as a wet retention pond with a surface drainage system (including storm sewer pipe). An underdrain can be included to dry the area after sufficient detention period is allowed so that the pond functions as a wet detention pond.

Public Capital Improvement Cost Estimates

15 inch Storm Sewer with underdrain $100,000

12.6.3.13 Summary: Watersheds #2700, #2900, and #3000

It is apparent from the preliminary analysis performed that the area encompassed by Watersheds #2700, #2900, and #3000 needs additional detailed investigation to accurately determine the route and destination of storm water runoff. It appears possible that Watershed #2900 and #3000 runoff is diverted by Scandia Road before reaching the NCDs, to either the system draining the lift station or even on to North Spring Road. In either case the downstream storm system is incapable of handling the 10 year event, the benchmark capacity for storm sewers. The pipe size required to handle the 10 year event from those two watersheds alone would be at least 48 inches and would be slightly undersized for the 51 cfs necessary. If these systems are contributing to the Watershed #2700 NCD at the lift station, their runoff needs to be captured and diverted to the assumed NCD outfall in the case of Watershed #3000 and to the proposed detention basin for the future development of Watershed #2900.

Subwatershed #2703 NCD served by the lift station poses a serious challenge for flood relief. Without details on the current flood concern noted on the map at the September 2005 CUPAC meeting, it is difficult to conclude if there is really a flooding problem as indicated by the results of the modeling. But if the reason no problem exists is due to karst drainage, the threat to the aquifer is real and has to be balanced by any action that increases the likelihood of flooding existing development when options for protection from flooding have limited effectiveness.

The most costly course if reducing karst drainage becomes necessary is to purchase properties at greatest risk and create a surface outlet to Green Bay that would require a gate to prevent inundation from Green Bay during extreme high water levels. Changes in water levels due climate change are unpredictable but the possibility of water levels above the 1986 elevations are certainly possible.

12.6.3.14 Watershed #3100 - Pheasant Park, Village Park to South of Maple Drive

Watershed Description. This 580 acre watershed is made up of five separate Subwatersheds of which three have significant storage in natural closed depressions. The contributing area extends almost 1 mile east of Woodcrest Road and as far as a ½ mile south of Maple Drive (CTH ZZ). Much of the watershed area is agricultural but also includes multifamily, single family, and commercial land uses in approximately one-quarter of the area. Located within the watershed area is the new Village/Town fire station and Pheasant Park development; each property has its own storm water management system but each property area only contributes a very small portion of the overall watershed storm water runoff.

Watershed runoff ultimately discharges to a very large natural depression which overlaps with an existing wetland. It is reported that there is a high groundwater infiltration rate in the north end of the wetland where runoff reappears in Subwatershed #6301 in a closed depression. The rate of soil infiltration at this depression is evident to casual observation.

It has been reported that the Village park area immediately east of Woodcrest Road and south of Autumn Court has experienced flooding in the past. This area is relatively flat and lacks proper drainage.

Figure 12-14 illustrates the location of the watershed boundaries and surface water flow direction.

Storm System Description. The fire station property includes an existing detention pond that discharges to the Subwatershed #3101 natural closed depression. Pheasant Park development uses an open channel infiltration trench drainage system. Runoff drainage west across Woodcrest Road from the east is controlled by an existing culvert.

East Mill Road has a discharge pipe available for discharge to the Subwatershed #3101 basin.

10-Year Event Storm System Analysis. The Pheasant Park channel system is the only storm water conveyance system besides the existing culverts. The Pheasant Park channel system has excess capacity for the 10-year rainfall event.

100-Year Event Pond System Analysis

Subwatershed Basin #3101. The existing development east of the north end of Subwatershed #3101 basin is situated at approximately elevation 600 feet NGVD 29. The final NCD is a wetland. A high rate of discharge through what is probably a karst feature leading to the Subwatershed #6300 NCD has been reported. The basin was modeled with a high level of infiltration and without infiltration to test the variability of the flood elevation. With significant infiltration, the pond tops out at 596.4 feet NGVD 29. With some pond overtopping (modeled as a 75 foot weir at a crest at elevation 599.5 feet), the basin tops out at 599.6 feet. With no outlet, the basin is just under capacity at elevation 600 feet NGVD 29. Observation by Village personnel during flood conditions indicates that the NCD has reached very near capacity.

Subwatershed Basin #3102. The flood elevation for the natural closed depression found immediately east of Woodcrest Road is at or slightly above storage capacity (elevation 616 feet NGVD 29) including the discharge through a 24 inch culvert under Woodcrest Road. The 100-year modeling results agree with reports of flooding onto the pavement without overtopping the road.

Subwatershed Basin #3105. Existing agricultural buildings are the lowest elevation structures exposed to potential flooding at approximately elevation 630 feet NGVD 29. This elevation would allow close to 2 feet of freeboard. The ortho photos indicate that the existing residential dwellings are an additional 2 feet higher in elevation.

Watershed Recommendations. Due to the large undeveloped/agricultural areas of Subwatersheds #3104 and #3105, there will be dramatic increases in peak runoff after full development. Both of these areas feed into Subwatershed #3102 and to the Woodcrest Road area already experiencing flooding. Flood control of developing areas - much of which is outside of the current municipal boundary - is vital to avoid a steadily increased problem with flooding.

The Village is currently considering the construction of a new soccer field in the park that could act as a storm water detention basin. There are some limitations to this concept. Creating an outlet for the pond is difficult since the area is flat. One option is an underground pipe but it may have to run a long distance to achieve daylight. The most obvious route is west along Autumn Road to the wetland in Subwatershed #3100. Considerations should be given to treating storm water prior to discharge. A soccer field would function as a dry detention basin to restore its normal function as a recreation facility. However, no water quality benefits result from dry detention basins.

An alternative possibility is to utilize the existing NCD at the east side of Woodcrest. The landscape already drains to this location. The parcel mapping indicates that the Village already owns this property. The poor drainage near the park facilities could be remedied with an open channel leading to the basin. The basin bottom could be lined as discussed in Chapter 13 and the outlet modified to create a wet detention pond with flood storage. The dual purpose of soccer field as detention basin savings would be lost and is a drawback, but this alternative has fewer challenges from the landscape and the outlet is readily available. It would not re-route water from the existing drainage patterns. Its feasibility depends on whether the volume of the available storage can be increased with grading and/or excavation to enlarge it into the hillside to the east or deepen it. The parcel to the south owned by the Curtis B and Roxann J. Wiltse Trust might need to be purchased. The possibility of raising Woodcrest Road in the low section could also be considered.

The general rule for a flood control structure is 5 percent of the contributing area. The combined contributing area for Subwatersheds #3102 and #3104 is 160 acres. At 5 percent, the required area of the detention basin is 8 acres. The approximate total of the two parcels is approximately 8 acres. To be effective, the soccer field as detention basin would need to be 5 percent of the Subwatershed #3104 contributing area of 120 acres, or 6 acres.

The runoff from #3105 and #3106 is mitigated by the NCD south of Maple Drive (CTH ZZ). This NCD should be designated as a **storm water storage zone**.

The Pheasant Park channels are below capacity and therefore capable of receiving additional storm water discharge. It is recommended that the existing Pheasant Park storm water channel, detention system and outlet be evaluated using existing construction record drawings (if available) or accurate site survey information to determine if routing additional storm water flow through the existing detention system outlet is feasible without flooding adjacent structures.

The Village is considering installation of a new storm sewer pipe for Subwatershed Basin #3101 crossing Sunset Road that would discharge to an existing open ditch on the north side of the road. Storm water runoff would flow toward the existing storm sewer system that currently receives flows from the NCD lift station. A review of the existing topography indicates that there is sufficient elevation for this new storm sewer to gravity feed the existing system. A 24 inch reinforced concrete pipe carrying 25 cfs would maintain the level of the NCD below 600 feet NGVD 29. However, given the potential for the connection of Watersheds #2900 and #3000 to this same system it is recommended that the combined impacts of the entire system should be evaluated prior to pursuing this project.

Public Capital Improvement Cost Estimates

Channel	$12,000
#3102 retention pond	$2,041,000
#3105 outlet storm sewer	$41,000
Total	**$2,094,000**

12.6.3.15 Watershed #3200

Watershed Description. This watershed of less than 30 acres is primarily golf course which is not expected to change land use. An NCD is indicated by the 2 foot elevation contour plan. Immediately below the NCD is a multi-family or condominium development. One of the structures lies within the footprint of the NCD near the edge below the 626 foot elevation contour. Figure 12-15 illustrates the location of the watershed boundaries and surface water flow direction.

Storm System Description. There is no storm sewer system.

10 Year Event Storm System Analysis. There is no storm water system within the watershed (other than roadside ditches). Therefore, the 10-year event storm water system analysis was not performed.

100 Year Event Pond System Analysis. The NCD floods to elevation 628.4 feet NGVD 29. This is above the elevation of the structure noted above.

Watershed Recommendations. Flood proofing around the structure encroaching upon the NCD would allow this area to become a **storm water storage zone** which is compatible with its use as a golf course. A safe outlet should be installed to drain off water past the downhill development should infiltration be the current primary outlet. Modifying this NCD for aquifer protection is unnecessary unless private wells are in the vicinity. The wellhead protection zone is up-gradient and the NCD is reasonably close to Green Bay. Encouraging a nutrient and pest management program for the golf course (if not already in place) would help to minimize pollutant runoff impacts to Green Bay.

12.6.3.16 Watershed #3600 - Country Lane, Fieldcrest, & Golf Course

Watershed Description. This 860 acre watershed is one of the largest in the study area. Existing land uses run the spectrum from commercial, park (golf course), and all residential density types. Storm water flow is directed through a series of culverts crossing STH 42 and adjacent frontage roads down through a wet golf course hazard pond, then through another series of culverts and channels to a natural closed depression adjacent to the shore of Green Bay.

There are only two other significant natural closed depressions in this system. One is very shallow and the other has development at a relatively low ground elevation. Figure 12-16 illustrates the location of the watershed boundaries and surface water flow direction.

Storm System Description. The storm water conveyance system consists of series of channel segments and culverts. The channel segments on the east side of the watershed along Bay Shore Drive were determined to carry insignificant discharge in the upland areas with little contributing area.

The culverts across STH 42 appear to be undersized. During large rainfall events flooding is a likely occurrence on the southeast side of the road. Although the culverts across STH 42 limit the flow rates to the golf course pond area, these flow rates exceed the capacity of the system of channels and culverts downstream of from the golf course pond. This system appears to be grossly undersized, given that the areas around the golf course pond generate significant runoff in addition to the runoff from discharge, the culverts from the contributing area north of the highway alone is much more than it can carry.

100-Year Event Pond System Analysis.

Subwatershed Basin #3601. The home of George and Arlene Schiestel sits along the 648 foot elevation contour providing less than 1 foot of freeboard to the flood protection elevation. Parcels immediately to the south and east are all situated on land with substantial areas at or below elevation 648 feet NGVD 29.

Subwatershed Basin #3603. Development on the parcel owned by Lester Hammersmith is shown on topographic mapping to be below elevation 642 feet NGVD 29 and is at least 3 feet below the flood elevation. Open Hearth Lodge is not much higher in elevation.

Golf Course Condominium Area. This area has experienced flooding issues in the past. Subsequent to the most recent major flooding event (Spring 2004), Village staff had discovered that one of the existing drainage culverts was plugged with debris. Despite correction of the culvert plugging issue, computer modeling results of the 100-year rainfall event indicate that current detention by existing upstream basins and the capacity of downstream culverts is inadequate to contain the runoff within the system and roadway overtopping is likely. The overflow elevation of the golf course is 622 feet. It appears that nearby adjacent structures are within 2 feet or less of this level. Therefore, there will be little to no freeboard during a large rainfall event.

Subwatershed Basin #3605. The capacity of the "ridge depression" at the outfall to Green Bay was exceeded in computer simulations for the 100-year event. However no infiltration was modeled. This NCD should be designated a **storm water storage zone**.

Watershed Recommendations. Options for reducing flooding are very limited due the placement of existing development low in the landscape in subwatershed #3603 and adjacent to the golf course hazard pond. The two primary options are to move water downstream more quickly from the area of potential flood damage or to detain it upslope. The recommended method of alleviating flooding is to increase storage and detention in the existing NCDs of subwatersheds #3601 and #3603 and prevent increased flooding through added detention BMPs in the upland areas contributing runoff as development occurs in accordance with the proposed ordinance. Additional flood storage should be constructed in the two NCDs of Subwatershed Basins #3601 and #3603 if excavation is permitted by existing bedrock and/or groundwater conditions. The feasibility of flood protection of existing development should be explored using berms or other measures to protect structures. These NCDs and the Watershed #3605 NCD should be designated as **storm water storage zones**.

Part of the flooding may be alleviated by increasing the size of pipe culverts leading to and across Little Sister Road. The downstream NCD would receive little additional runoff volume except for what presently infiltrates. The peak discharge rate should be reduced somewhat by the detention added to the system. The property owner of the subwatershed #3605 NCD, West Capitol Incorporated, has been receiving water from the system for at least as long as the culvert crossing under Little Sister Road has existed. However, during high water events, some of the water was probably diverted by the road when the culvert capacity was exceeded.

The STH 42 culvert system consists of a pair of existing 30 inch diameter corrugated metal pipe culverts crossing STH 42. They are preceded by a single 30 inch diameter culvert under Country Lane. Little Sister Road has a similar culvert condition. These culverts should be replaced by a continuous culvert from south of Country Lane to the north of Little Sister Road, preferably even beyond the private drive. Runoff from between the road could be captured by inlets connected to the culvert. Whether or not they are adequately sized is a matter of perspective. The development above STH 42 could be better served by larger culverts. The development below STH 42 would be better served if there were no culverts.

The existing condition 100-year event peak discharge through the STH 42 culvert system is over 80 cfs. The peak discharge from the landscape downstream of STH 42 is over 120 cfs. Added detention capacity in the Subwatershed #3601 and #3603 NCDs will reduce the 80 cfs discharge. Protection of the area downstream of STH 42 can be improved by increasing detention capacity in the golf hazard pond, floodproofing the adjacent development, and eliminating culverts where possible and lowering and enlarging them where they must remain.

It is recommended that the golf course pond be modified to function as a true detention pond. The drainage from the nearby buildings should be diverted downstream of the pond so that the high flood elevation of the modified pond isolates the proposed pond from the building and creates freeboard. As this area is privately held, the responsible party should undertake the necessary engineering evaluation and design.

The culverts both in the privately held area and under Little Sister Road are undersized. But there are practical limitations to what can be modified. To prevent flooding the drainage needs to be lowered dramatically. In the extreme case where upland storage cannot be modified for Subwatersheds #3601 and #3603, the following would be required to safely pass the discharge from the existing condition development runoff. The description below summarizes modeling results. Each element was modeled as it currently exists. Each element could be modified to a limited degree without changing the flood elevations, but the description below is essentially what would be required.

The culverts under STH 42 would need to be replaced by a continuous set of culvert pipes as described earlier. In order to maintain a headwater below 642 feet NGVD 29 to protect the existing development, a 6 foot by 3 foot high concrete box culvert is required. The inverts would be close to the existing elevations.

The receiving channel should be shaped using a 4 percent slope, would be trapezoidal, and have a 10 foot bottom, be 2 feet deep and have 4-to-1 sideslopes and lead to the golf course pond.

The golf course hazard pond should be modified to the extent possible. Modeling used a theoretical proposed detention basin pond in place of the existing golf course hazard pond for a maximum area approximating an ellipse of 400 feet long, 160 feet wide at the axes (about 1.2 acres of maximum surface area) with a top elevation of 624 feet NGVD 29.

The outlet of the pond would be a channel with a 20 foot bottom at a pond outlet invert elevation of 620 feet NGVD 29, or 2 feet lower than presently indicated on the contours.

The next culvert downstream, an existing 40 foot by 28 foot elliptical corrugated metal pipe should be replaced by a 6 foot by 3 foot high box culvert lowered to elevation 618 feet from the presently estimated invert of 619 feet.

The channel leading west to Little Sister Road would need to be reconstructed at an invert elevation of 614 feet, 6 feet below the existing contour elevation.

The culvert presently crossing Little Sister Road should be replaced south of the present crossing, aligned with the east-west channel using a 6 foot by 4 foot box culvert with an upstream invert elevation of 613 feet. The culvert would be 140 feet long, discharging to the proposed Storm Water Storage Zone of the Subwatershed #3605 NCD. A safe outlet should be constructed for discharge from the NCD. The NCD is close to the Bay, outside of the Wellhead Protection Zones and does not require a liner.

Alleviating flooding by means of increasing conveyance from STH 42 through Little Sister Road is not a recommended alternative, although some elements will improve the situation. The culvert locations discussed above crossing Country Lane, STH 42 and Little Sister Road should be followed when it becomes convenient to replace them. The new location of the Little Sister Road crossing eliminates unnecessary additional flow restrictions and the extension of the STH 42 culverts eliminates the obstructions created by the culverts crossing the frontage roads.

Country Lane and Fieldcrest Road Areas. Storm water flows from the Country Lane and Fieldcrest Road areas will increase with imminent development. The current low gradient channel south of STH 42 that drains the area is inadequately sized for these flows. It is recommended that the Village develop a plan for a 10 foot wide, 1.5 foot deep channel with a greenway flood buffer (these areas are typically used for recreation paths as well) where shown on Figure 12-16. Future development in the area should be required to conform to the plan as a condition of plat approval.

Private Capital Improvement Cost Estimates

Floodproofing	$27,000
Golf Hazard retention pond	$360,000
Channel& Culvert Replacement/Reconstruction	$100,000
Total	**$487,000**

Public Capital Improvement Cost Estimates

Culvert Replacement/	$120,000
Capacity Expansion of NCDs in Subwatersheds #3601 and #3603	$3,800,000
Total	**$3,920,000**

12.6.3.17 Watershed #3900 – STH 42 and STH 57 Convergence

Watershed Description. The existing watershed land use is a mixture of commercial and residential uses and is approaching full development. There are no known natural closed depressions or retention/ detention ponds. Figure 12-17 illustrates the location of the watershed boundaries and surface water flow direction.

Storm System Description. The upper parts of the watershed are made up of isolated networks of culverts, channels, and storm sewer. A 42 inch pipe is connected to a downstream 36 inch pipe on Bluffside Lane between Gateway Drive and North Bay Shore Drive. A channel connects this to a downstream system of smaller pipes.

10-Year Event Storm System Analysis. The existing 36 and 42 inch diameter pipe sizes feed into a smaller diameter system downstream. There is no apparent available storage to mitigate the runoff peak except for the connecting channel. The computer model results start at the pair of 18 inch CMP parallel to Bluffside Lane. The succeeding pipes are single 24 inch diameter leading across Maple Drive to a 20 foot wide, 3 foot deep open channel, followed by a series of 24 inch CMPs and finally to a 30 foot wide, 3 foot deep open channel discharging to the bay.

While the outfall channel appears to have excess capacity, the storm sewer upstream as far as North Bay Shore Drive does not have capacity for the total runoff. The large diameter pipes are capable of feeding water to the smaller downstream system beyond the system's capacity. While the large diameter pipes most likely have the necessary capacity, based on the modeling completed and the area of contribution defined by the 2 foot elevation contour plans, the smaller downstream pipes do not have sufficient capacity for the anticipated 10-year rainfall event discharge.

100-Year Event Pond System Analysis. There are no ponds within the watershed to analyze.

Watershed Recommendations. No flood issues have been reported although the modeling results suggest otherwise. There is an opportunity for a detention/retention system for Subwatershed #3901 on what is already publicly-owned property. The parcel is apparently owned by the Village described as *"that part of Government Lot 4 Section 5-31-28 that forms a triangle where Hwy 57 meets Hwy 42 DESC 14P121 DCR"*. STH 57 appears to effectively channel water from the south and east toward a 42 inch pipe leading to the parcel with the potential pond. The inlet conditions may need to be modified to allow more efficient entry to avoid road overtopping. This pond could be constructed as a lined wet pond to treat polluted runoff and protect groundwater, although it is just outside the wellhead protection management zone. This pond would capture the Subwatershed #3901 runoff from 30 acres. The site is steep which severely limits the possible area of the pond. The pond should be as large as physically possible to achieve water quality benefits. Due to the limited area, large events should simply bypass the BMP to avoid flooding out the pond and resuspending solids.

Downstream the topography flattens in the area south of E. Larson where a channel leads to a pipe system. The transition from open channel to pipe is likely to back up water during large events - assuming that the contributing area is not diverted during overland flow along streets such as Maple Drive. The ortho photos do not indicate development on some of these properties including that of William and Louise Robbins where much of the channel is located. If flooding has occurred, it may not have been observed. The property owned by Dwight and Mary Jo Anderson is landlocked and adjacent properties have irregular boundaries. This suggests the possibility of purchase by the Village for a detention/retention facility for water quality treatment and if large enough, for peak discharge reduction. The contributing area is approximately 45 acres suggesting a pond of 2-2.5 acres to be effective at flood reduction. But it does not appear that there is sufficient undeveloped area for a facility of this size.

Gateway Drive. Current storm water flows are eroding the existing ditch along Gateway Drive. The current system of alternating pipes and ditches results in high energy, concentrated flows exiting the existing pipes and eroding the ditches. The system should be converted into an all piped system and tied into the STH 57 drainage network recently updated as part of the WDOT work.

Public Capital Improvement Cost Estimates

Gateway Drive Retention Pond	$100,000
Larson Road Retention Pond	$100,000
Increase Storm Sewer Size	$32,000
Total	**$232,000**

12.6.3.18 Watershed #4000 - North Bay Shore Drive and East Mill Road

Watershed Description. The existing watershed land use is a mixture of commercial and residential uses and is approaching full development. Figure 12-18 illustrates the location of the watershed boundaries and surface water flow direction. Although a storm sewer system will intercept runoff for the design storms, bypass from larger storms may be reaching the area northeast of Mill Street. The elevation in this area bottoms out at a contour elevation of 586 feet NGVD 29. The mapping of problem areas in the fall 2005 CUPAC meeting indicated flood problems in this vicinity, but the nature of the flooding was not defined.

The upper half (south end) of Subwatershed #4002 is the development known as Scandia Village which is served by an engineered detention basin.

The downstream end of Subwatershed #4002 is north of East Mill Road and South Spring Road. A small NCD (4002B) is found there. The 1 foot contours provide added definition as shown in Figure 12-18a. Most of Subwatershed #4002 is intercepted by streets and would not reach the interior of the block north of East Mill Road. For the purpose of mapping the flooded area, only the area lying within the block that would not normally be graded to drain to the street was considered as part of the contributing area. The volume of water that might bypass the storm system and reach the interior of the area is indeterminant. The only outlet for the NCD is to the natural closed depressions to the north (Subwatershed #6301).

The larger portion of the watershed drains overland and through storm sewers to the bay.

Storm System Description. The storm water system consists of a privately-owned detention basin serving the upper 2/3 of Subwatershed #4002, which primarily incorporates the Scandia Village development area. The detention basin discharges to a storm sewer system running north along Claflin Street to East Mill Road. The storm sewer continues northwest to North Bay Shore Drive, where it connects to the existing system that drains Subwatershed #4001. This existing storm sewer originates uphill approximately 750 feet from the afore mentioned junction at Mill Road from southwest along Bay Shore Drive, draining the area south of Maple Drive. The existing downtown storm sewer system runs approximately 200 feet west of the Mill Road/Bay Shore Drive intersection along West Mill Road, before turning north to the outfall discharge to the Sister Bay portion of Green Bay.

The entire system described above (except for the segment leading from the Scandia Village detention pond to the intersection of Claflin Street and East Mill Road) is less than 2 years old.

The engineering details of the recently constructed storm sewer work in this area are documented on existing project record/as-built drawings. The information from these documents was used as the basis for input data the H&H computer model. Input data for the older, non-documented part of the system was based on field GPS data, GIS topographic elevation data, selected field measurements, and field observations. Figure 12-18 illustrates the location of the watershed boundaries and surface water flow direction.

10-Year Event Storm System Analysis. The minor drainage system requires supplemental capacity to meet the discharge requirements of the drainage area Subwatersheds. Based on the modeling completed, and the area of contribution defined by the 2 foot elevation contour plans, the existing pipe system does not have sufficient capacity for the anticipated 10-year rainfall event discharge. The Scandia Village detention basin does reduce the discharge of the 10-year storm from 32 cfs to 17 cfs, but the basin is at the point of overtopping. The existing basin it is not capable of containing the 100-year storm event.

The Claflin Street-East Mill Road segment of the storm sewer system is close to design capacity and appears adequate for the 10-year event. The North Bay Shore Drive storm sewer pipe lies on a significant grade; it is anticipated that there is substantial bypassing of the inlets by storm water running northeast down the North Bay Shore Drive pavement/gutters. Therefore the existing pipe is believed to be adequate for the runoff quantity captured by inlets and discharged into the storm sewer system. Village staff have noted street flooding to be a problem along North Bay Shore Drive between Mill Road and Sunset Drive, a distance of approximately ¼ mile with only one set of inlets between on what is essentially very flat terrain.

100-Year Event Pond System Analysis. As noted above, computer model results indicate that the Scandia Village storm water detention pond (4002A) would be at capacity during the 10-year storm event. Its storage volume would be exceeded during more intense storms such as the 100-year event.

The NCD north of East Mill Road has a volume of runoff that exceeds the volume of storage below the 585 foot contour elevation. The area will flood to an elevation above the overland escape elevation (between 585 and 586 feet) in order to discharge. Runoff volume (0.98 acre-feet), storage volume below 585 contour (0.41 acre-feet).

Watershed Recommendations. Since the system is new, it has not been tested by time for rainfall events leading up to the 10-year event. Real world runoff characteristics and behavior must be observed to determine if the system needs additional infrastructure to meet the drainage system flooding goals.

The design for the Scandia Village detention pond should be reviewed. The design was probably done with more accurate topography and more detailed watershed modeling, but it should be checked to see what occurs during the 100 year event. A safe outlet should exist in the event of either overtopping or a plugged outlet.

The area northeast of East Mill Road should be monitored to see if overland flow from large events does reach the low area and cause problems. East Mill Road should act as a diversion to direct runoff downhill towards Bay Shore Drive. If flooding does pose a problem, it can be alleviated with minor grading to improve the hydraulic connection to the NCD of Subwatershed #6301. The Storm Water Storage Zone for this area should be at least at elevation 585 feet. All development should be required to have a low space elevation in excess of elevation 586 feet.

The excess runoff generated by Watershed #4000 that routes along the street curb and gutter system to North Bay Shore Drive can be captured in Watershed #6300 with additional inlets in the flat stretch of North Bay Shore Drive.

12.6.3.19 Watershed #4400

Watershed Description. This 160 acre watershed has a current land use that is primarily agricultural along with the majority of the remaining area being low density residential. It is expected that the area will fully develop at a medium residential density.

The watershed discharges away from Green Bay and is not contributing to the Sister Bay watershed. Figure 12-19 illustrates the location of the watershed boundaries and surface water flow direction.

Storm System Description. There is no storm sewer system.

10-Year Event Storm System Analysis. There is no storm water system within the watershed (other than roadside ditches). Therefore, the 10-year event storm water system analysis was not performed.

100-Year Event Pond System Analysis. Development already exists in the vicinity of the #4401 NCD. The top elevation contour is at 684 feet USGS, and the existing condition modeled flooded elevation is 683 feet USGS.

Similarly, development is already encroaching on the NCD in Subwatershed #4402. The surface route escape elevation contour is 674 feet USGS, the flooded elevation is 673.4 feet USGS for the existing condition.

Watershed Recommendations. It is apparent that the NCDs cannot be used for the added flood storage required when the watersheds are in the developed condition. It will also be necessary to create an outlet. It will be important that all future development accommodate all increases in runoff.

For the NCD in Subwatershed #4401 some flood proofing may be necessary around the structures on the parcels of Leslie Boden and the parcel owned by JM Northwoods Properties on the corner of STH 57 and Northwoods Road.

The Subwatershed #4402 NCD covers 16 acres and represents a considerable area to designate as unbuildable. The alternative to upland site development flood control BMPs is for <u>any</u> development within the watershed to make a legal arrangement with landowners downstream.

These NCDs should be designated as **storm water storage zones**.

12.6.3.20 Watershed #5200

Watershed Description. This 11 acre agricultural watershed drains to an NCD with no known outlet except for road overtopping to the west outside of the Green Bay watershed. Figure 12-20 illustrates the location of the watershed boundaries and surface water flow direction.

Storm System Description. There is no storm sewer system.

10-Year Event Storm System Analysis. There is no storm water system within the watershed (other than roadside ditches). Therefore, the 10-year event storm water system analysis was not performed.

100-Year Event Pond System Analysis. The top ground elevation contour is at 732 feet USGS and the modeled flooded elevation for the existing condition is 731.6 feet.

Watershed Recommendations. The NCD crosses out of the planning area study. This area should be designated as a **storm water storage zone**. The flooded elevation under future conditions is at or above the containment elevation of the NCD. Upland development should be required to control storm water for flooding and, as in the case of Watershed #4400, be prepared to make legal arrangements with downstream landowners if necessary.

12.6.3.21 Watersheds #5800 and #5900 – Sunnyside, Admiral, and Sunny Road Area

The Sunnyside, Admiral, and Sunny Road area does not have adequate drainage for future development. A neighborhood drainage plan should be developed and adequate drainage required as a condition of the building permits issued for this neighborhood. A ditch network with culverts under driveways would be adequate.

12.6.3.22 Watershed #6300 – South Spring Road and STH 42 (North Bay Shore Drive)

Watershed Description. This watershed is largely composed of commercial and dense residential development. A natural closed depression lies within Subwatershed #6301. It is reported that this area receives groundwater from the wetland of Subwatershed #3101 due to the development of an artesian condition (water is visibly flowing vertically out of the ground). The source area is known to infiltrate water at a high rate and there is an elevation difference of over 20 feet. Figure 12-21 illustrates the location of the watershed boundaries and surface water flow direction.

The area between Scandia Road and Mill Road, Bay Shore Drive and South Spring Road is defined by 1 foot contours from a survey specifically for that area. Datum for the STS survey was not provided. It is assumed to be consistent with the County furnished contours.

Storm System Description. The storm system consists of a 36 inch x 22 inch arch feeding into a 30 inch CMP discharging to the bay according to the 1973 Robert E. Lee drawings. The upstream pipe invert at Casperson's pond is 581.75. The system drains the basin just west of South Spring Road and the only other feed into the system are two inlets at North Bay Shore Drive.

10-Year Event Storm System Analysis. The system has capacity in excess of the 100-year event for the Watershed #6300 contributing area. As discussed under the description of Watershed #4000, bypass from #4000 is probable and likely to flow along North Bay Shore Drive which is flat with few inlets. As described by Village staff, there are insufficient inlets to capture runoff without road flooding.

100-Year Event Pond System Analysis. The 1 foot contours show a pond bottom of 579 feet, or about 2.8 feet below the outlet elevation. The existing basin has a closed contour capacity that is reached at elevation in excess of 585 feet NGVD in Subwatershed #6301. The 100-year water surface or flood elevation reaches 582.4 feet NGVD. An outlet control structure exists on Casperson Pond which can be manipulated. The study assumed that the outlet pipe invert would control and that control structure stop logs are removed to allow maximum outflow and minimize flooding. There is no information available on the outlet structure.

Watershed Recommendations. The invert of the storm sewer as it crosses North Bay Shore Drive is 581.2 feet USGS. The GIS mapping 2 foot contours indicate a ground elevation of approximately

586 feet. Allowing 3 feet for inlet height and assuming a surface elevation of 585 feet (due to contour tolerances), the storm sewer falls 0.8 feet over a distance of 160 feet with a 0.5 percent slope. It is 600 to 800 feet to the nearest adjacent storm sewer systems. The system on Sunset Drive crossing North Bay Shore Drive should be at approximately elevation 581.5 feet, while the system in Watershed #4000 is already at capacity. It may be necessary to route piping directly to the bay in a manner parallel to the existing system. A separate system would also preserve the pipe system's capacity to drain the basin at the present rate.

The runoff from Watershed #4000 exceeding the modeled capacity of the storm sewer is estimated at approximately 25 cfs for the 10 year storm. With the runoff from the local area, the southwest system shown in Figure 12-21 should use a 30 inch RCP pipe main at 0.5 percent leading to the Bay with 24 inch diameter pipes leading from the inlets. The northeast system should be adequate with 24 and 18 inch diameter pipes, respectively.

Serious consideration should be given to establishing a Storm Water Storage Zone in Subwatershed #6301. Because information on the outlet structure is unknown, and unless there is a plan in place to remove the stoplogs during large runoff evens, it would be prudent to establish the Storm Water Storage Zone at the overland escape elevation, which exceeds elevation 585 feet. The attached Figure 12-21a shows the extent of the 585 foot contour by shading. Other unknowns are the rate of discharge into the pond from Subwatershed #3101 via underground flow and the rate of discharge through the ground towards the bay. The survey indicates a ponded elevation of 579.2 feet but the water surface of the bay on that date is not noted. The bay may have some control on the pond pool elevation should a cobble substrate be present. These factors may well over-ride the simple runoff/discharge results. All development should be required to have a low space elevation in excess of elevation 586 feet.

Presently the basin is drained near the bottom of the basin, which precludes using it as a wet retention pond. The outlet structure could be modified to provide the minimum depth of 5 feet of undrained permanent pool if the discharge rate prevents flooding adjacent homes. If an Operation and Maintenance plan that calls for an operator to remove stoplogs in the event of large storms is practical, added protection would be provided to adjacent development. But unless these measures are taken and with the minimal present capacity, it appears that creating a water quality BMP is not feasible at this location without compromising the 100-year event flood protection.

Eroding and undersized storm water flow channels are a common problem in the Sister Bay area. Many existing channels have eroded to the underlying bedrock or flood routinely. As impervious surface area and the associated storm water runoff volumes upstream of the channel sections increase with future development, the existing problems will be intensified and new problems will occur in channels that are currently adequate. The following channel improvements are recommendations:

Public Capital Improvement Cost Estimate

New Storm Water System $143,000

12.7 SUMMARY OF STORM WATER IMPROVEMENT RECOMMENDATIONS

Table 12-1 summarizes the recommended storm water management planning capital improvements for the Sister Bay planning area over the planning period. The improvements are listed by watershed.

TABLE 12-1

RECOMMENDED STORM WATER PLANNING IMPROVEMENTS

STORM WATER MANAGEMENT PLAN
SISTER BAY, WISCONSIN

Watershed No.	Improvement Description	Budget Estimate
2600	New Storm Sewer along Waters End Road	$125,000
2700	North Spring Road Culverts; New Lift Station Pump; Subwatershed #2702 Retention Basin; Lift Station Storm Sewer Outlet	$1,196,000
2800	Replace Storm Sewer System with Larger Pipe	$100,000
2900	New Storm Sewer and Retention Pond	$530,000
3000	New Storm Sewer with Underdrain	$100,000
3100	Channel Improvements; Subwatershed #3102 Retention Pond; Subwatershed #3105 Outlet Storm Sewer	$2,094,000
3600	**Private Improvements:** Floodproofing Structures; Golf Hazard Retention Pond Improvements; Channel and Culvert Replacement and/or Reconstruction	$487,000
	Public Improvements: Culvert Replacements; Subwatershed #3601 and #3603 NCD Capacity Expansion	$3,920,000
3900	Gateway Drive Retention Pond; Larson Road Retention Pond; Storm Sewer Capacity Improvements	$232,000
6300	New Storm Sewer Improvements	$143,000
Total		**$8,927,000**

Note

1. Estimates include engineering and contingency costs.

X:\S\SISTB\050200_UTILITIES\Project\Sister Bay copy\Report\Chapter 12\[Table 12-1.xls]Table 12-1

The recommended first steps before implementing the proposed improvements is to enact zoning, secure easements, and purchase property necessary before additional development in the flood prone areas and BMP locations occurs.

12.7.1 Natural Closed Depressions

The NCDs need to be treated as outfalls receiving runoff. To protect development from flooding, development should be prohibited within the Storm Water Storage Zone unless other provisions are made to prevent flooding of both the proposed development and others hydrologically and hydraulically connected to the NCD.

This study provides the mapping necessary to create the Storm Water Storage Zones. The boundary of the zones is recommended to be at a flood protection elevation of 2 feet above the ponded depth given in Appendix C. The figures in this chapter show the approximate area of the zones at the 2 foot elevation contour above the ponded elevation which is not necessarily the same as the 2 foot freeboard.

Due to the uncertainty of the contour data, which is plus-or-minus 1 foot accuracy, it is recommended that each Storm Water Storage Zone be determined based on the process outlined below. The investigation of NCDs is also discussed in Chapter 13 of this report in the context of water quality.

1. Conduct a topographic survey up to an elevation of at least 2 feet above the currently modeled ponded elevation.
2. Survey to identify existing development by use (dwelling, garage, outbuilding, commercial business, etc.), including the low adjacent grade and floor elevations.
3. Convert the survey data into a contour CAD drawing.
4. Re-execute the H&H models to determine the ponded depth for the 100 year events for existing and future development, using the updated, accurate, topographic data.
5. Map the area at ponded depth, 1 foot of freeboard, and 2 feet of freeboard with corresponding elevations and area values.
6. Village to review the results to determine how existing development is affected and review possible alternatives such as:
    a) Can the existing development be flood-proofed?
    b) Can the NCD be regraded to increase storage to minimize the ponded elevation?
    c) Can storage be created in the upland contributing area to supplement the available storage of the NCD?
    d) Can an outlet be created to reduce the ponded elevation without causing flood damage downstream?

12.7.2 Ditches

The roadside ditches should continue to be the primary means of conveying storm water for runoff reaching road right-of-ways. Ditches have greater conveyance capacity than pipes at a lower capital cost and do not need to be as low in elevation which is an important consideration due to shallow bedrock. They have the added benefit of acting as water quality swale BMPs.

12.7.3 Storm Sewer

In existing urbanized areas where storm sewers are necessary their conveyance capacity is limited by a combination of size, slope, and the inlet characteristics. They are typically designed for a 10 year storm, but the surface topography will affect the efficiency of transferring surface runoff in ditches or gutters into the pipe below. Ditches and gutters are both the primary and tertiary conveyance system. They collect water, directing it to catch basins or inlets which capture some of the water and drain it into the underground pipe system. But their efficiency is determined by whether or not they lie on a slope or at a low point. Inlets on slopes typically allow a significant quantity of flowing water to bypass the inlet. Therefore the gutter or ditch continues to carry runoff supplementing the pipe system.

Recognizing this means that water will continue to be on the surface during large events and will accumulate at low points in the system. These sump areas will act as dry detention basins until the system can drain it down. Development in low areas will be flooded during extreme events unless they are above a surface "escape route" that allows water above the "escape elevation" to drain off. For development along the Bay Shore Drive, this may mean finding opportunities between existing developments to allow water to drain away from the street to the Bay. To determine this will require elevation information with accuracy to the nearest 0.1 foot vs. the 2 foot contour information available for this study.

The reconstruction of STH 42 provides an opportunity to adjust grades enough to optimize drainage as much as possible. Inlets should be located at low points. Escape routes can be situated so they are higher than inlets to avoid becoming the primary runoff route, but low enough to protect existing development.

12.7.4 Outfalls

Lake Michigan ranges in elevation (International Great Lakes Datum 1985) from 576.05 to 582.35 feet over the course of the historical record since 1918. The listed normal range is from 578.5 - 579.5 feet IGLD, but it was only a relatively short time ago in 1986 that it reached the high level noted above. To maximize the capacity of a storm sewer it should have a free outfall. For pipes with a low slope it is even more important to have a free outfall, otherwise the lake level will reach a significant distance up the pipe and decrease its conveyance efficiency.

This is of concern for Bay Shore Drive between Mill Street and Sunset Drive where the street elevation is in the vicinity of 586 feet USGS. The depth of an inlet structure alone can require 2.5 – 3 feet of height when a 12 inch pipe is used, higher for larger pipes. Special inlet structures such as the existing drainage pipe bisecting this area will be necessary.

This is also of concern for the development surrounding the wetland NCD of Subwatershed #2703 where it appears there are low adjacent grades below the 584 foot elevation contour, or within less than 2 feet of the high water level of Lake Michigan. This is why the lift station is in place.

The elevation differences in this area are so tight that even the datum becomes an issue. As noted above, the Lake Michigan water surface elevations are in International Great Lakes Datum 1985. There is a datum difference of 0.43 feet between IGLD 1985 and the NGVD 29 datum used for the flight to develop the 2 foot contours used in this study. In other words, the high water elevation of 582.35 feet IGLD 1985 converts to 582.78 feet NGVD 29.

CHAPTER 13

STORM WATER QUALITY MANAGEMENT EVALUATIONS AND RECOMMENDATIONS

This Chapter of the Sister Bay Comprehensive Utilities Plan evaluates storm water runoff quality within the planning area and recommends steps to reduce surface water pollutant loading while minimizing contamination of ground water. This chapter summarizes the findings from this evaluation.

13.1 GENERAL

Throughout this chapter several key terms and phrases are used to identify or describe important storm water management activities, facilities or natural features in addition to those found in Chapter 12. The following terms/phrases are defined for use in this section:

- **Developed** areas are generally those that were not subject to the post-construction performance standards (s. NR 151.12 or NR 151.24, Wis. Adm. Code).
- "**MS4**" means a conveyance or system of conveyances, including roads with drainage systems, municipal streets, catch basins, curbs, gutters, ditches, constructed channels or storm drains, which meets all the following criteria:
  - Owned or operated by a municipality.
  - Designed or used for collecting or conveying storm water.
  - Not a combined sewer conveying both sanitary and storm water.
  - Not part of a publicly owned wastewater treatment works that provides secondary or more
  - Stringent treatment.
- **P-** Phosphorus
- **TSS** – Total Suspended Solids

13.2 OBJECTIVES

Sister Bay has undertaken a Comprehensive Utilities Plan in part to improve water quality. The study is to "assess current non-point source pollution discharges into State waters, and establish pollution control elements that target the performance standards as defined in CH. NR 151 Wis. Adm. Code".

Sister Bay is voluntarily developing a plan to meet the permit requirements. The NR 151 performance standards do not apply directly to Sister Bay since the municipality is not required to secure a Municipal Separate Storm Sewer System (MS4) Wisconsin Pollution Discharge Elimination System (WPDES) permit. Although communities are encouraged to voluntarily apply for permits Sister Bay has not indicated that they intend to do so.

NR 151.13 Developed urban area performance standard is the most applicable section of the rule. It sets goals of 20 percent reduction of TSS by 2008 and a 40 percent reduction by 2013.

These goals are set for the level of development in effect at the time when the "new development" rules of NR 151.11 and 12 went into effect in 2004. The best available information for the level of development is the 2005 digital ortho photos available from Door County. Development taking place after that time must meet the new rules for post-construction storm water; typically to reduce TSS by 80 percent compared to loading with no-controls. In effect, the scope of the water quality study is limited to the currently developed area, since future development is required to meet other goals and is explicitly excluded by NR 151 language. SEH's analysis with SLAMM however does include the loadings generated by future development, but assumes that the future development will meet the 80 percent reduction rule of the law.

Two of Sister Bay's stated primary concerns are to reduce and prevent beach closings and to prevent ground water contamination. However, the tasks requested in the scope of this study do not address how to prevent beach closings and protect ground water. This analysis and recommendations are focused on surface water pollutant loadings using SLAMM as directed by Sister Bay. SEH has made recommendations that provide some level of protection for ground water but a comprehensive analysis of ground water and Best Management Practice designs that treat water for infiltration are beyond the scope of this study.

13.2 STORM WATER QUALITY STUDY METHODOLOGY

The water quality analysis uses information developed as described in Chapter 12 to input to the Source Loading and Management Model (SLAMM). SLAMM version 9.1 evaluates the TSS and Phosphorus loading generated by the developed condition. SLAMM is strictly a surface water model for developed areas only which evaluates pollutant loadings. It is intended for developed areas only and is not intended for agricultural or natural undeveloped conditions. It is not a hydrologic model.

NR 151 compares annual pollutant loading of developed areas for the "no-controls" condition to the "with controls" condition. This is different from the H&H comparison of "undeveloped" compared to "developed" since water quality considers only the developed condition. The no- controls pollutant loading establishes the baseline of comparison. No-controls is defined as when no Best Management Practices (BMPs) are employed in the model to mitigate runoff pollutant loading. The drainage system is assumed to be curb and gutter in good condition regardless of actual conditions. The drainage system control for Sister Bay supplies some level of TSS/P reduction through the shallow roadside ditches that function as swales, a recognized BMP control in SLAMM.

13.4 SURFACE WATER QUALITY ANALYSIS

13.4.1 Water Quality Analysis

SLAMM was used to create a model that quantifies pollutant loadings from outfalls identified during the system inventory. Pollutant loadings included the amount of TSS and Phosphorus in the water for three stages. These three stages include (1) baseline (without pollution controls); (2) drainage controls; and (3) recommended BMPs to obtain the target amount of pollutant removal.

The analysis will present the pollutant load:

1. coming off the landscape with no controls, regardless of destination for the existing conditions;
2. coming off the landscape with no controls regardless of destination for the future condition;
3. reaching the surface waters with no controls for the existing condition;
4. reaching the surface waters with drainage control for the existing condition.

The difference between result #1 and #2 represents the added load due to full planned buildout. Twenty percent of that difference represents the added load allowed to reach the surface waters of the state. The twenty percent is the load result due to the construction site WPDES requirements.

The difference between result #1 and #3 represents the load measured by the state. The difference is the load diverted to NCDs.

The table below presents the results for the pollutant loading conditions, 1-4 noted above.

Annual Pollutant Loading (lbs)					
	1. Existing Baseline	2. Future Baseline	3. Existing 'No Controls' Excluding NCD loading	Difference due to NCDs (1 minus 3)	4. Existing with Drainage Control
TSS	418,860	788,000	296,352	122,508	262,052
Phosphorus	1,286	2,527	973	313	890

The existing condition reduction due to drainage control and capture by NCDs (1 minus 4) is already close to 40 percent. Although the unimproved NCDs cannot be counted, the improved NCDs can.

When measuring reduction for a permit, if the NCD is improved to treat the runoff, the influent load is added to the baseline amount and the effluent (water reaching a surface water) load, if any, is added to the "with treatment" result.

According to NR151 guidance correspondence:

> *"Areas and loadings that shall not be included: Any internally drained area with natural infiltration. (This does not include engineered or constructed infiltration areas.) However, an internally drained area that discharges to a karst feature is not likely to be receiving adequate treatment prior to any contact with the groundwater. The municipality is encouraged to look at the area for possible treatment options."*

Of the runoff reaching surface waters, the current treatment occurs in swales, otherwise referred to as drainage controls. In this case, swales refer to existing roadside grass ditches. Typical swales in Sister Bay appear to be shallow grass channels. The model swale geometry reflects this characteristic. The length of road in each non-NCD watershed was compared to the total acreage to create a typical "Swale Density" for each residential land use/soil type combination. This foot/acre swale density is used by the SLAMM software, along with assumed typical swale geometry to calculate the amount of TSS removed in each watershed.

The third treatment condition requires the design of additional BMPs to achieve final necessary treatment. These BMPs may include wet detention ponds, infiltration, additional swales, or street sweeping. BMPs will need to be implemented to achieve remaining treatment.

13.5 PLANNING LEVEL RECOMMENDATIONS

13.5.1 Storm Water Management Ordinance

Appendix D provides model ordinances that address both erosion control and post-construction storm water management. The post-construction ordinance has requirements parallel to DNR's NR 151 and is based on an ordinance that has met past DNR approval for compliance with MS4 permit ordinance enactment requirements. In addition to the NR 151 requirements this ordinance requires peak flow controls for the larger storms up to and including the 100-year event.

The ordinance still must be formatted to conform to Sister Bay's ordinance style. Generic references to "municipality" should be replaced by "Sister Bay" (if appropriate), and local authorities used for review, appeals, and recommendation such as a Board of Public Works should be inserted where appropriate in place of the language used in this model ordinance. The final version should be reviewed by the Village's legal counsel to assure that the Village recognizes the responsibilities and liabilities that may be encumbered by adoption of these ordinances.

Discussions are currently underway between DNR and Door County regarding how NR 151 Technical Standards for BMPs approved for use by DNR do not consider the unique karst and aquifer conditions for Door County. One example is on site infiltration which is a valued BMP that would help to minimize storm water runoff volume increases not mitigated by lined detention basins. The ordinance should be reviewed for revisions that will be coordinated with the outcome of these talks.

13.5.1.1 Why Ordinance is Needed

Currently, state storm water rules do not provide water quality protection for small sites and provide no protection from increased flooding from major storms. The state currently regulates both construction period and post-construction runoff from development that involves construction sites disturbing 1 acre or more with the Wisconsin Pollution Discharge Elimination System (WPDES) permit. Construction site erosion control of all one and two family dwellings is regulated by Commerce, but not post-construction runoff unless the 1 acre threshold is met. There are loopholes for land disturbing activity development for multifamily and nonresidential construction sites under one acre where no state requirements are in force, and for all construction under one acre for post-construction performance standards. Regardless of disturbed area size, only the discharge rates for the two year storm are addressed.

13.5.1.2 Recommendations

To close these loopholes the Village of Sister Bay will need to enact storm water and construction site erosion control ordinances. A Village ordinance should address the sites which are less than one acre and require controls to prevent flooding or runoff increases from major events. In lieu of individual lot specific practices, a regional site can be selected if all parties can agree, but the primary responsibility should remain with the developer, be it private or commercial.

13.6 BMP STRATEGIES - USE OF EXISTING FEATURES

13.6.1 Roadside Ditches as BMPs

Existing roadside swales are already providing some level of treatment. BMPs are most effective when they treat runoff at the source. Roadside ditches as swales treat road runoff and provide some pretreatment prior to discharge to NCDs. New development should be required to treat runoff prior to discharge from the site. The treatment should occur as close to the area of origin as possible. Ideally, roadside swales should be reserved for treatment of the roadway runoff. When within a sub-watershed discharging to a NCD, the discharge from developments should be designed to bypass the swale system of existing roads and route the treated runoff at the undeveloped discharge rate to the NCDs.

13.6.2 Ground Water Protection Concerns Natural Closed Depressions as BMPs

Presently, runoff water carrying surface contact pollutants drains to NCDs. NCDs are natural traps for the surface water contaminants and divert runoff from surface water outlets. As noted in the previous chapter, it is recommended that the existing storage areas or natural closed depressions within the planning boundary be preserved. When the characteristics are favorable, it is beneficial for the aquifer to be replenished or recharged from these sources where the runoff has been captured and cleaned via vegetative uptake and soil adsorption. But aquifer contamination in these natural closed depressions is a very real concern.

If thin, chemically inert soils with high porosity overlie fractured bedrock or karst conditions, there is a direct route for contamination of the aquifer. In northern Door County, this is generally the case. Figure 13-1 illustrates the location of the Village's wellhead protection management zones. The wellhead protection zone represents the underground area supplying water to municipal wells within a given time span. It is the area of most vulnerable to contamination from surface water pollutants. Given the hydro geologic characteristics of the area, that is the ground water pathway through voids, all areas within the study area should provide protection whenever feasible.

The working assumption by professionals concerned with ground water contamination in northern Door County is that shallow fractured bedrock with karst features is prevalent throughout the study area. Although karst features sometimes provide evidence of their existence on the surface, it is not always the case. Short of scraping away the entire overburden to inspect the bedrock surface, it is not feasible to determine their existence. The NCDs themselves are a strong indicator of highly fractured bedrock and karst features.

Typical storm water BMPs are not intended to provide the needed ground water protection for Sister Bay's subsurface conditions. The storm water rules are primarily a surface water protection mechanism that includes some protections for ground water by exempting the infiltration requirements of NR 151.12 and prohibiting infiltration practices in areas with the conditions found in the Sister Bay planning area. The primary focus of the rules and BMPs is the reduction of Total Suspended Solids (TSS). Many of the pollutants of concern are related to TSS and a reduction in TSS provides a similar decrease in other pollutants. However, pollutants of concern to ground water are not always reduced by mechanisms that reduce TSS.

There are two basic alternatives to reduce aquifer contamination: treatment or diversion. One alternative is to line the bottom of the NCDs with impervious materials to minimize infiltration. Since the infiltration outlet no longer exists, it will be necessary to provide an alternative outlet. A storm system to convey excess runoff to Green Bay would be necessary, but to protect Green Bay from the new storm water pollutant load it would still be necessary to provide treatment prior to discharge. The other alternative is to treat water sufficiently to allow infiltration.

The natural closed depressions were shaped under undeveloped land use runoff conditions. The practical strategy for storm water conveyance is to follow the existing landscape and reconnect storm water with the natural drainage system whenever possible. The NCDs are a final destination or receiver for much of the natural drainage system. The goal of the storm water management system is to mitigate human changes to the water cycle and restore as much of the natural system as possible. It is desirable and most efficient to utilize the NCDs to the extent possible. The alternative is to convey the majority of the runoff to Green Bay. This does not eliminate the step of reducing runoff water pollutants with BMPs. Due to shallow bedrock, construction costs for a storm system would be prohibitive when compared to the NCD outlet alternative.

Ideally, runoff would be treated prior to discharge to the NCD. Presently, there are developed and varying levels of undeveloped areas contributing to NCDs. As undeveloped areas are developed treatment will occur by regulatory requirement and by the proposed ordinance. The currently developed area runoff remains untreated except to the level of treatment provided by swales.

The problem that presents itself is how to capture and treat untreated runoff. Small events have been shown to carry the bulk of the annual pollutant load. (The caveat to this rule is that uncontrolled soil erosion from construction sites and bare agricultural fields is heavier during larger events). A standard water quality treatment strategy is to capture and treat the small storm runoff event. This is typically done by introducing a diversion structure within the conveyance system. Small discharges are diverted to the BMP for treatment, while larger discharge rates bypass the diversion and are conveyed to the outfall. The small (water quality) storm is selected as the design storm.

There are few if any concentrated flow conveyances in the landscape in which to install the design storm diversion. Since the landscape does not generally focus runoff into a distinct channel, the diversion strategy is not a practical means of isolating the small event for treatment. The NCDs represent the most logical and feasible location to treat the existing development runoff.

13.6.3 Recommended BMP

A unique BMP is needed to address the unique conditions of the Sister Bay area. Two mechanisms that will reduce pollutants of concern are wet retention and soil adsorption. Figure 13-2, “Improved Natural Closed Depression” is a schematic of the proposed BMP. The runoff events at or below the water quality storm are the primary source of storm water pollutants. SEH proposes an alternative strategy to “isolate” the small storm for treatment.

SEH proposes to utilize the NCDs as wet retention ponds as well as flood control practices. The bottom of the basin will be where the initial runoff accumulates. Ground water contamination is reduced from the current condition by lining the bottom of the natural closed depressions to the level equivalent to the volume of the water quality storm.

Wet retention ponds normally use a surface water outlet. The existing NCDs can be viewed as retention ponds that outlet through the soil/bedrock pores and voids. To capture the runoff long enough to afford treatment the bottom is lined. Then only the flooded area above the initial runoff volume is allowed to infiltrate. While passing through the soils, the second type of treatment known as soil adsorption will occur in soils with sufficient fines. This represents a secondary treatment system. Due to financial constraints only the liner soil is imported if native soils are unsuitable. The upper soils remain as undisturbed as possible. The secondary treatment option occurs only if the proper soil type is available, but is an extra step.

The water level in the NCD at the onset of the storm will dictate the behavior of the BMP. When the period between runoff events is prolonged, the NCD will be "dry". Any event less than the water quality storm, the event with the majority of the annual pollutant loading, will be captured within the lined area which in effect achieves100 percent capture.

When periodic runoff events are more frequent water is present. A similar condition occurs when the storm exceeds the design storm capacity elevation. The Improved NCD will function as a wet retention pond. Although the runoff event will flood the NCD to a level above the liner allowing runoff to infiltrate towards the ground water, infiltration represents a very small outlet, prolonging detention and leading to a cleaner discharge. Since the H&H analysis assumed no infiltration whatsoever (except as noted in Chapter 12 in a few isolated watersheds), the flood protection elevations of the proposed Storm Water Storage Zones are still conservative.

Surface discharge may not occur if the storm is small enough. Surface outlets are modified if necessary such that the elevation of the discharge invert is at the design storm storage elevation. Thus the first flush is either captured totally or treated, and the excess runoff from larger events carrying a minority of the annual pollutant load is allowed to escape to either the existing surface conveyance or through the soil to replenish the ground water.

13.6.4 When to Use an Improved Natural Closed Depression

This practice does not promote additional infiltration. Instead, infiltration is reduced and the practice allows some level of treatment before infiltration occurs. Careful selection of liner materials and type should be investigated. Locally available silts have shown better performance than clays. The liner should be constructed with the appropriate soils or in accordance with NRCS Field Office Technical Guide Standards including Clay, Geosynthetic, Polyethylene Geomembrane Liners.

When an opportunity presents to observe fractures and karst, they should be sealed. A geotextile should be installed below the liner, directly over the bedrock when possible, to prevent the migration of soil though voids.

Door County is currently in the process of working with DNR to provide more specific guidance on the use of BMPs to address the shallow bedrock and karst features conditions occurring there. SEH has forwarded information on this proposed BMP to both Door County and Wisconsin DNR and incorporated their comments into this discussion. DNR reports the use of a similar concept in other parts of the state.

Each NCD has unique characteristics that need to be established for the parameters of:

1. Storage volume
2. Outlet size, elevation
3. Soil profile
4. Bedrock depth
5. Seasonal high ground water
6. contributing area size and land use

These parameters are necessary to analyze and /or design both water quantity and quality functions of the NCD. A soil boring or excavation pit to determine: soil profile information for potential natural attenuation and liner potential, bedrock depth, and seasonal high ground water elevation. The installation of a monitoring well with regular testing of ground water or interflow from monitoring well samples should establish if pollutants are migrating into the aquifer from runoff. The results of the testing will demonstrate which sites are contributing the most pollutants and provide a means of prioritizing implementation of ground water protection BMPs.

Each site needs to checked for the presence of wetlands prior to moving forward with design. The basin's topography should be established through a detailed site survey providing greater accuracy than the 2 foot contour mapping used for this study. This includes the presence of existing development, basement elevations, and potential surface outlets.

Current outlet conditions for these Natural Closed Depressions need to be investigated. There are three basic scenarios which may exist. The first is natural drainage through the bottom of the depression into the ground water aquifer. This is the anticipated method of drainage for the majority of NCDs within Sister Bay. This outlet conditions would be treated by the modified BMP as described above.

The second scenario may occur in NCDs near roads, where ditches and culverts may serve as outlets that are below the storage elevation of the 2 year storm. This outlet should be modified to force the lined NCD to act as a wet detention pond, releasing it at a reduced rate to promote sedimentation or deposition of TSS..

The third scenario involves NCDs which have a higher outlet, above the storage elevation of the 2 year storm. The lined bottom of the NCD treats the small storm first flush as described above, but leaves a route of escape for the larger, cleaner flows. This allows prevention of erosion of the NCD and liner by releasing excess water.

Like all BMPs, an Operation and Maintenance Plan should accompany the design. Use of the monitoring wells can provide evidence of changes in the function of the system and alert those responsible to the need for additional steps.

The final design of this BMP should incorporate the flood storage elements for the Storm Water Storage Zones as discussed in Chapter 12. The concerns expressed in Chapter 12 regarding the authority to implement both the flood control and water quality protection aspects apply to these recommendations as well. The responsible party must be identified, who may not be the developer. An easement or other legal instrument is necessary for the responsible party to gain access for investigation, construction, monitoring, and maintenance.

Surface water carrying pollutants is presently draining into NCDs and reaching the ground water through the subsurface drainage voids. The proposed BMP is not intended as the primary ground water protection feature to guarantee the prevention of ground water contamination from storm water runoff. Such protection is beyond the scope of this study. It does provide treatment counted by WI-DNR under NR 151.13 while improving the present level of groundwater contamination.

13.7 WATER QUALITY CAPITAL IMPROVEMENTS

The first step is to secure easements and property necessary to accomplish the work in order to minimize the potential for development encroachment on areas that require structural BMPs beyond the Improved NCD. It is believed that since the area of the NCD improvement lies within the Storm Water Storage Zones, this work can be done by easement vs. purchase if the authority to zone in this manner exists.

is the 2005 digital ortho photos available from Door County. Development taking place after that time must meet the new rules for post-construction storm water; typically to reduce TSS by 80 percent compared to loading with no-controls. In effect, the scope of the water quality study is limited to the currently developed area, since future development is required to meet other goals and is explicitly excluded by NR 151 language. SEH's analysis with SLAMM however does include the loadings generated by future development, but assumes that the future development will meet the 80 percent reduction rule of the law.

Two of Sister Bay's stated primary concerns are to reduce and prevent beach closings and to prevent ground water contamination. This analysis and recommendations are focused on surface water pollutant loadings using SLAMM as outlined in the requested scope of services. SEH has made recommendations that provide some level of protection for ground water and will reduce pollutant loadings to Green Bay. This in turn will reduce pollutants at beaches.

13.3 STORM WATER QUALITY STUDY METHODOLOGY

The water quality analysis uses information developed as described in Chapter 12 to input to the Source Loading and Management Model (SLAMM). SLAMM version 9.1 evaluates the TSS and Phosphorus loading generated by the developed condition. SLAMM is strictly a surface water model for developed areas only which evaluates pollutant loadings. It is intended for developed areas only and is not intended for agricultural or natural undeveloped conditions. It is not a hydrologic model.

NR 151 compares annual pollutant loading of developed areas for the "no-controls" condition to the "with controls" condition. This is different from the H&H comparison of "undeveloped" compared to "developed" since water quality considers only the developed condition. The no- controls pollutant loading establishes the baseline of comparison. No-controls is defined as when no Best Management Practices (BMPs) are employed in the model to mitigate runoff pollutant loading. The drainage system is assumed to be curb and gutter in good condition regardless of actual conditions. The drainage system control for Sister Bay supplies some level of TSS/P reduction through the shallow roadside ditches that function as swales, a recognized BMP control in SLAMM.

13.4 SURFACE WATER QUALITY ANALYSIS

13.4.1 Water Quality Analysis

SLAMM was used to create a model that quantifies pollutant loadings from outfalls identified during the system inventory. Pollutant loadings included the amount of TSS and Phosphorus in the water for three stages. These three stages include (1) baseline (without pollution controls); (2) drainage controls; and (3) recommended BMPs to obtain the target amount of pollutant removal.

The analysis will present the pollutant load:

1. coming off the landscape with no controls, regardless of destination for the existing conditions;
2. coming off the landscape with no controls regardless of destination for the future condition;
3. reaching the surface waters with no controls for the existing condition;
4. reaching the surface waters with drainage control for the existing condition.

TABLE 13-1

RECOMMENDED STORM WATER QUALITY IMPROVEMENTS
STORM WATER MANAGEMENT PLAN
SISTER BAY, WISCONSIN

Watershed No.	Wellhead Protection Management Zone		Area (acres)	Estimated NCD Improvement Cost	
	Zone 1	Zone 2			
Priority Watersheds					
3000	**Yes**	No	83	$	488,400
3102	**Yes**	No	119.5	$	703,200
3105	**Yes**	No	n/a	$	2,942,400
4401	**Yes**	No	58.8	$	346,000
4402	No	Yes	117	$	688,500
Priority Watershed Subtotal				**$**	**5,168,500**
Secondary Watersheds					
300	No	No	10	$	58,800
500	No	No	16	$	94,200
800	No	No	40	$	235,400
900	No	No	192	$	1,129,900
1002	No	No	203	$	1,194,600
1300	No	No	37.3	$	219,500
1600	No	No	70	$	411,900
1900	No	No	65	$	382,500
2002	No	No	52.3	$	307,800
2302	No	No	21	$	123,600
2500	No	No	11.3	$	66,500
3101	No	No	156	$	918,000
3200	No	No	29	$	170,700
3603	No	No	131.8	$	775,600
5200	No	No	11.4	$	67,100
6301	No	No	13.9	$	81,800
Secondary Watershed Subtotal				**$**	**6,237,900**
Overall Planning Area Total				**$**	**11,406,400**

Note

1. Estimates include engineering and contingency costs.

X:\S\SISTB\050200_UTILITIES\Project\Sister Bay copy\Report\Chapter 13\[Table 13-1.xls]Table 13-1

Inspect ditches for signs of erosion and excessive sedimentation. Repair as needed.

Minimize use of salt for de-icing and sweep roadways to recapture and dispose of sanding.

Establish a leaf collection program for the fall.

Sweep roadways in the spring after local trees shed blossoms to prevent what has found to be significant sources of phosphorus.

Information and Education. Issue periodic bulletins of storm water information and education materials available from WI-DNR and the UW-Extension. Issue these with building permits and make available in a rack in the Village Hall.

CHAPTER 14

RECOMMENDED CAPITAL IMPROVEMENTS PLAN

This chapter summarizes the recommended infrastructure system improvements and presents a proposed capital improvements program. The recommended Capital Improvements Plan prioritizes system improvements and provides a schedule for the timing of construction. Budget cost estimates for each improvement are also summarized.

14.1 RECOMMENDED WATER SYSTEM CAPITAL IMPROVEMENTS

14.1.1 Water Storage

It is recommended that the Water Utility construct a new 0.15 MG water tower. The recommended tower overflow elevation should match the overflow elevation of the Jungwirth Tower. The recommended location for the new water tower is on the Village-owned wastewater treatment plant site.

14.1.2 Combining Pressure Zones

It is recommended that the Utility consolidate the operation of the existing dual pressure zone system under one combined pressure zone. The existing PRV Stations should be decommissioned, and any isolation valves between the existing pressure zones be opened. The Hwy 57 Standpipe will then function as a ground reservoir, with standpipe storage being pumped into the system by the Sister Bay Booster Station. The Well 1 pump equipment will need to be modified to pump against the higher head conditions of the combined pressure zone.

Because of the higher pressures that will be generated in the lowest lying portions of the water distribution system immediately adjacent to Green Bay, the Utility should consider the installation of individual PRVs on water services in these affected areas. It is also recommended that the consolidation of the two pressure zones into a single zone be planned to coincide with the implementation of the new water tower recommended above.

14.1.3 Water Service to Outlying Planning Areas

The recommended combined pressure zone system will adequately serve the majority of the future water service area with pressures and fire flows. The only significant area that cannot be adequately served is the area of higher topographic elevation south and west of Country Lane in the southwestern corner of the planning area. The Water Utility will need to implement a new high level pressure zone to adequately serve future customers in this area.

14.1.4 Distribution System

Figure 7-7 illustrated the recommended Sister Bay Water Utility Year 2025 Master Plan. The figure illustrates recommended improvements to the existing distribution system and the recommended transmission mains required to serve the future service area. The improvements have been recommended to strengthen and expand the existing transmission main network, and support system expansion into future service areas.

14.2 RECOMMENDED SANITARY SEWER CAPITAL IMPROVEMENTS

14.2.1 Improvements to Address Existing Needs

The improvements to address existing sanitary sewer deficiencies were shown in Figure 10-1 and summarized in Table 10-1. These improvements address three different types of existing sewer system deficiencies: potential future capacity restrictions, pipe settlements and sump manholes.

To address the potential future capacity restrictions deficiency, it is recommended that a sanitary sewer flow diversion upstream of the problem area be created by redirecting some of the wastewater flow around the affected low flow capacity area. It is recommended that this sewer flow diversion be constructed south of Maple Drive and west of Claflin Street.

The sewer pipe segment deficiency identified in Chapter 10 will be difficult to repair without removing and replacing the settled sections of pipe. Due to the potential for further settlement, it is recommended that the entire segment from manhole to manhole be removed and replaced. This will allow inspection of the trench bottom prior to new pipe installation, to determine if the trench bottom needs additional treatment prior to installation of a new sewer pipe.

Several sanitary sewer system manholes were constructed without poured inverts, and currently act as sump manholes. During periods of low flow, wastewater solids drop out of suspension into these sumps, and the sumps need to be regularly cleaned. This is an unnecessary recurring system maintenance activity that can be eliminated with proper manhole construction. It is recommended that the identified sump manholes be modified with poured concrete inverts. The recommended improvement will require a contractor to temporarily bypass pump around the affected manholes, properly clean the sumps, pour new concrete inverts, and allow the concrete inverts to properly cure prior to removing the temporary bypass.

14.2.2 Improvements to Serve Future Growth

The recommended approach for serving future growth is a combination of constructing new facilities in the future development areas as well as building a new trunk sewer down Bay Shore Drive to serve future development to the south. As the existing trunk sewer system is close to capacity in critical downtown Village areas, the new trunk sewer in Bay Shore Drive will provide additional capacity in the downtown area, while at the same time providing a means to serve future areas to the south.

Sanitary sewer service to the future expansion area was broken down into drainage basins or sanitary sewer "regions" for planning the implementation and sequencing of improvements. There are six individual sewer service regions identified in the northern portion of the future sewer service expansion area (Regions A – F), and four regions identified in the southern portion of the expansion area (Regions G – J). Recommended trunk sewer facility improvements to serve expansion areas were illustrated in Figures 10-2 and 10-3.

14.3 RECOMMENDED STORM WATER CAPITAL IMPROVEMENTS

It is recommended that the necessary easements and/or property described in Chapter 12 are secured to accomplish the work to minimize potential for development encroachment on areas that require structural BMPs beyond the Improved NCD. As it is believed that the area of the NCD improvement lies within the Storm Water Storage Zones, this work can be done by easement vs. purchase if the authority to zone in this manner exists. The highest priority recommended Improved Natural Closed Depressions improvement projects are discussed further below.

14.3.1 Watersheds #2700 and #2900

The highest priority project should be all associated work that affects the recommended storm water lift station pump replacement in NCD #2700. Recommended improvements include proposed piping, culverts, replacement of pump and storm sewer along Scandia Road from Watershed #2900.

14.3.2 Watershed #3100

The second priority is the recommended improvements associated with the proposed detention basin within Watershed #3100 to address flooding concerns adjacent to Woodcrest Road. It has been reported that the flooding currently experienced is minor.

14.3.3 Watershed #6300

A new storm sewer is recommended for North Bay Shore Drive between and Sunset and E. Mill Roads, and the project should be incorporated with the proposed WDOT improvements planned for the STH 42 corridor unless flooding becomes chronic and causes extensive property damage.

14.3.4 Watershed #2800

A new storm sewer along Waters End Road and channel lining is recommended and can be incorporated into other work when this project can be accommodated into the Town of Liberty Grove's budget.

14.3.5 Watershed #2600

Improvements recommended for Watershed #2600 will replace undersized storm sewer but was not listed as a flood prone area. Continued monitoring should determine this project's priority.

14.3.6 Watershed #3600

While recommended improvements in this area are important for reducing flooding in the golf course/condominium area, much of the recommended corrective work can be accomplished within the grounds by the responsible party. The recommended detention pond improvement upslope may not prove to be necessary if flood proofing is determined to be feasible.

14.3.7 Watershed #3900

It is recommended that preliminary engineering work should proceed on the potential detention pond near East Larson to confirm that the pond would be effective in this location and to allow property purchase to proceed before additional development takes place. If flooding near Larson is of minor concern, the remaining upslope improvements can be delayed. The property at Gateway Drive is already under the control of the Village, and construction can be coordinated with the STH 42 work by WDOT to save cost, but final construction of improvements can wait for funding.

14.3.8 Water Quality Capital Improvements

The proposed capital improvements to address storm water quality issues are primarily recommended to provide protection for the Village's underground drinking water supply. The current level of storm water runoff treatment, primarily due to the numerous NCDs, is at the approximate level required by NR 216 permit requirements. The discussion in Chapter 13 prioritizes the work by location with respect to the Wellhead Protection Zones. As the water service is extended out to areas beyond the wellhead protection zones as they now exist, the need to protect private wells replaced by a municipal water system decreases. However, continued growth may exceed projections, or current wells may need to be abandoned. New well sites will need protection zones. The protection of the area's groundwater aquifer should continue to be pursued to the extent feasible.

The locations recommended for Improved NCDs are noted in the figures for each watershed in Chapter 12.

14.4 COMPREHENSIVE UTILITIES MASTER PLAN

The proposed Sister Bay Comprehensive Utilities Master Plan is illustrated in Figure 14-1. The proposed Master Improvements Plan has been formulated based on all the information presented in this study. All the improvements have been developed and prioritized based on deficiencies identified in the existing infrastructure systems, and the needs of the Village's future service planning area. Table 14-1 summarizes the recommended capital improvements plan for the Comprehensive Utilities Planning area over the 20-year planning period.

The actual construction cost for the recommended improvements may vary from the costs outlined in this report, depending on the year facilities are constructed, the rate of increase in future construction costs, and unforeseen conditions which could be encountered during design of the improvements.

In establishing priorities for these improvements, it will be necessary to take into consideration the availability of Utility financial resources and local Village needs to assure that the recommended improvements are implemented in an orderly, coordinated, and economical fashion.

TABLE 14-1

RECOMMENDED CAPITAL IMPROVEMENTS PLAN

COMPREHENSIVE UTILITIES MASTER PLAN

SISTER BAY, WISCONSIN

SHORT-TERM IMPROVEMENTS (2006-2010)

Water System	**Budget Estimate**
Construct New Water Tower	$840,000
Eliminate Dead End Water Mains in Low Fire Flow Areas	$760,000
Modify Well 1 Pump to operate in Combine Zone System	$55,000
Water System Subtotal	**$1,655,000**

Sanitary Sewer System	**Budget Estimate**
Construct Diversion From MH 39 to MH 193	$50,000
Remove and replace 10 inch pipe from MH 47 to MH 45	$60,000
Pour concrete inverts in manholes to eliminate sumps	$40,000
Construct Trunk Sewer in Bay Shore Drive	$445,000
Upgrade Lift Station No. 1 Capacity	$229,000
Stage 1 of Region H Improvements; Interim Lift Station and Force Main	$549,000
Stage 2 of Region H Improvements; First Phase of Lift Station H and Force Main	$2,000,000
Region E Improvements	$947,000
Region F Improvements	$1,504,000
Region G Improvements	$1,096,000
Sanitary Sewer System Subtotal	**$6,920,000**

Storm Water System - Infrastructure Improvements *(by Watershed Priority)*		**Budget Estimate**
2700	North Spring Road Culverts; New Lift Station Pump; Subwatershed #2702 Retention Basin; Lift Station Storm Sewer Outlet	$1,196,000
2900	New Storm Sewer and Retention Pond	$530,000
3100	Channel Improvements; Subwatershed #3102 Retention Pond; Subwatershed #3105 Outlet Storm Sewer	$2,094,000
6300	New Storm Sewer Improvements	$143,000
2800	Replace Storm Sewer System with Larger Pipe	$100,000
2600	New Storm Sewer along Waters End Road	$125,000
3600	Private Improvements: Floodproofing Structures; Golf Hazard Retention Pond Improvements; Channel and Culvert Replacement and/or Reconstruction	$487,000
3600	Public Improvements: Culvert Replacements; Subwatershed #3601 and #3603 NCD Capacity Expansion	$3,920,000
3900	Gateway Drive Retention Pond; Larson Road Retention Pond; Storm Sewer Capacity Improvements	$232,000
Storm System Infrastructure Subtotal		**$8,827,000**

Storm Water System - Water Quality Improvements *(by Watershed Priority)*		**Budget Estimate**
3000	NCD Improvements	$488,400
3102	NCD Improvements	$703,200
3105	NCD Improvements	$2,942,400
4401	NCD Improvements	$346,000
4402	NCD Improvements	$688,500
Storm System Water Quality Subtotal		**$5,168,500**

Overall Short-Term Improvement Total	**$22,570,500**

TABLE 14-1

RECOMMENDED CAPITAL IMPROVEMENTS PLAN
COMPREHENSIVE UTILITIES MASTER PLAN
SISTER BAY, WISCONSIN

LONG-TERM IMPROVEMENTS (2011-2025)

Water System	**Budget Estimate**
Distribution System Expansion	$12,033,000
Water Supply or Storage	$800,000
Implement New Southwest High Level Pressure Zone	$450,000
Water System Subtotal	**$13,283,000**

Sanitary Sewer System	**Budget Estimate**
Stage 3 of Region H Improvements	$4,829,000
Region J Improvements	$2,591,000
Region I Improvements	$817,000
Upgrade MH106 to MH104 from an 8 inch to 10 inch pipe	$50,000
Region D Improvements	$731,000
Region C Improvements	$3,193,000
Region B Improvements	$429,000
Region A Improvements	$789,000
Sanitary Sewer System Subtotal	**$13,429,000**

	Storm Water System - Infrastructure Improvements	**Budget Estimate**
3000	New Storm Sewer with Underdrain	$100,000
Storm System Infrastructure Subtotal		**$100,000**

	Storm Water System - Water Quality Improvements *(no priority)*	**Budget Estimate**
300	NCD Improvements	$58,800
500	NCD Improvements	$94,200
800	NCD Improvements	$235,400
900	NCD Improvements	$1,129,900
1002	NCD Improvements	$1,194,600
1300	NCD Improvements	$219,500
1600	NCD Improvements	$411,900
1900	NCD Improvements	$382,500
2002	NCD Improvements	$307,800
2302	NCD Improvements	$123,600
2500	NCD Improvements	$66,500
3101	NCD Improvements	$918,000
3200	NCD Improvements	$170,700
3603	NCD Improvements	$775,600
5200	NCD Improvements	$67,100
6301	NCD Improvements	$81,800
Storm System Water Quality Subtotal		**$6,237,900**

Overall Long-Term Improvement Total	**$33,049,900**

Note

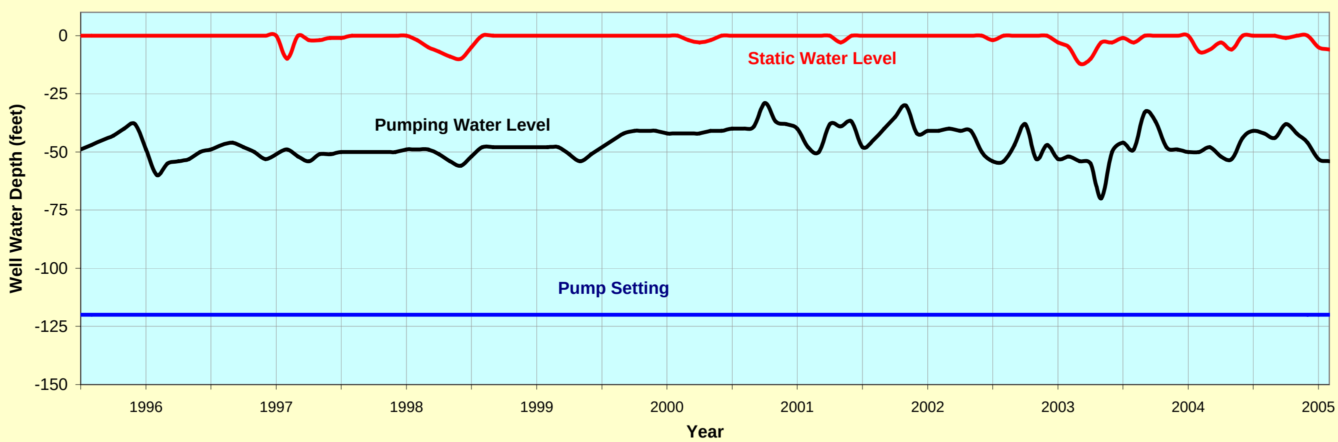
Estimates include engineering and contingency costs.

P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay study\March-April 2008 Report Revisions\Chapters 1-14\Chapter 14\[Table 14-1 Revised 04090

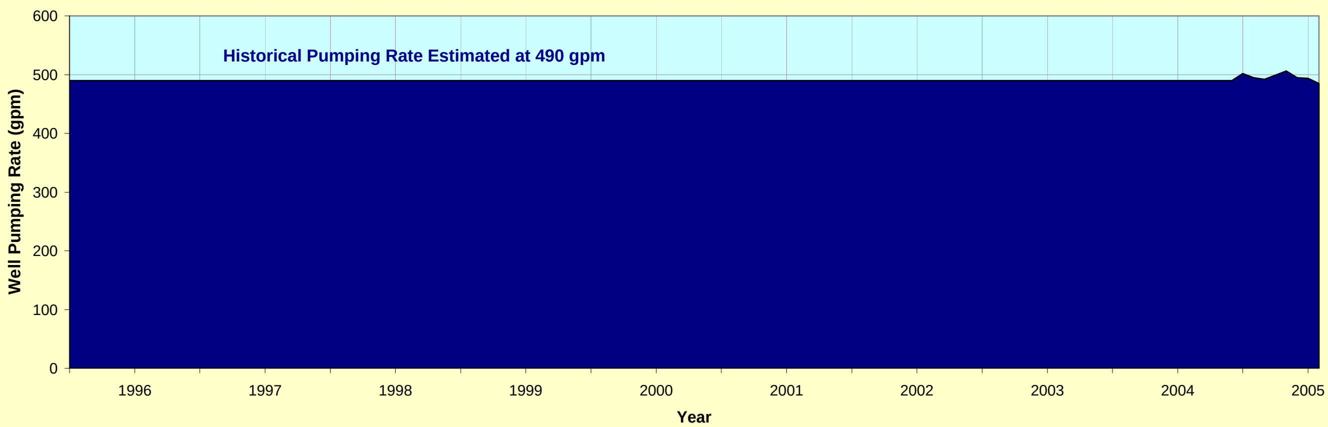
APPENDIX A

WELL PERFORMANCE SUMMARY

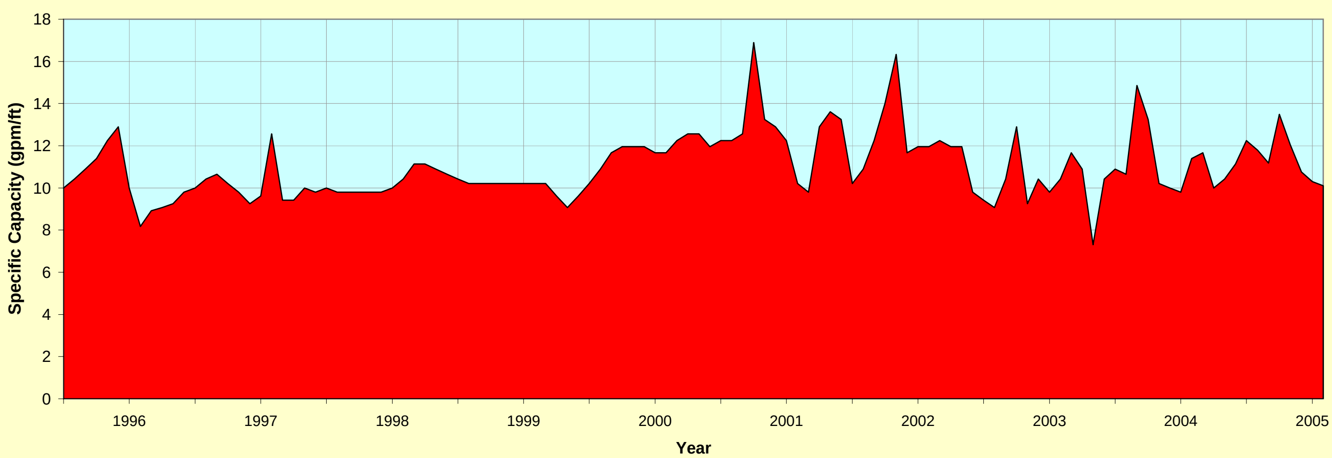
Well 1 Water Levels (1996 - 2005)



Well 1 Pumping Rate (1996-2005)



Well 1 Specific Capacity (1996-2005)



SHORT ELLIOTT HENDRICKSON

SEH

FIGURE A-1

HISTORICAL PERFORMANCE: WELL 1

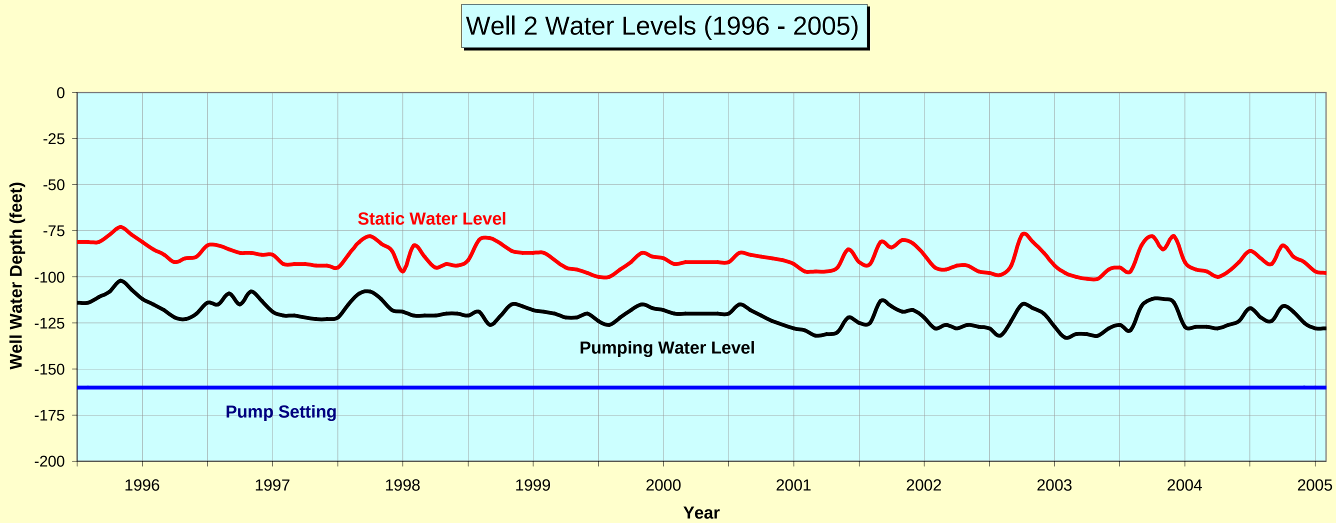
SISTER BAY WATER UTILITY

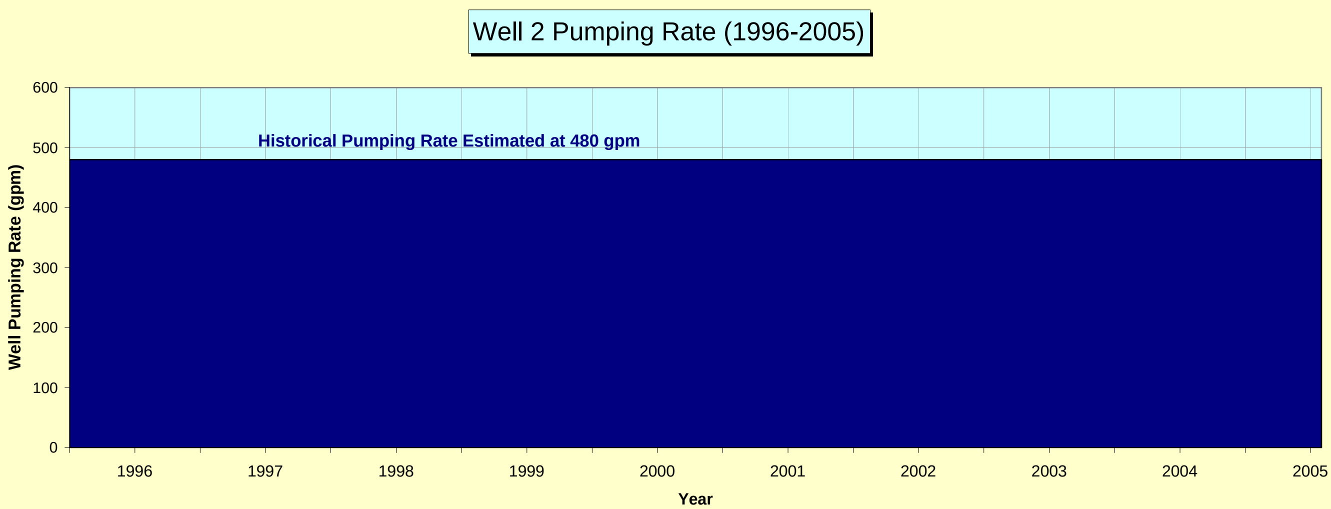
VILLAGE OF SISTER BAY, WISCONSIN

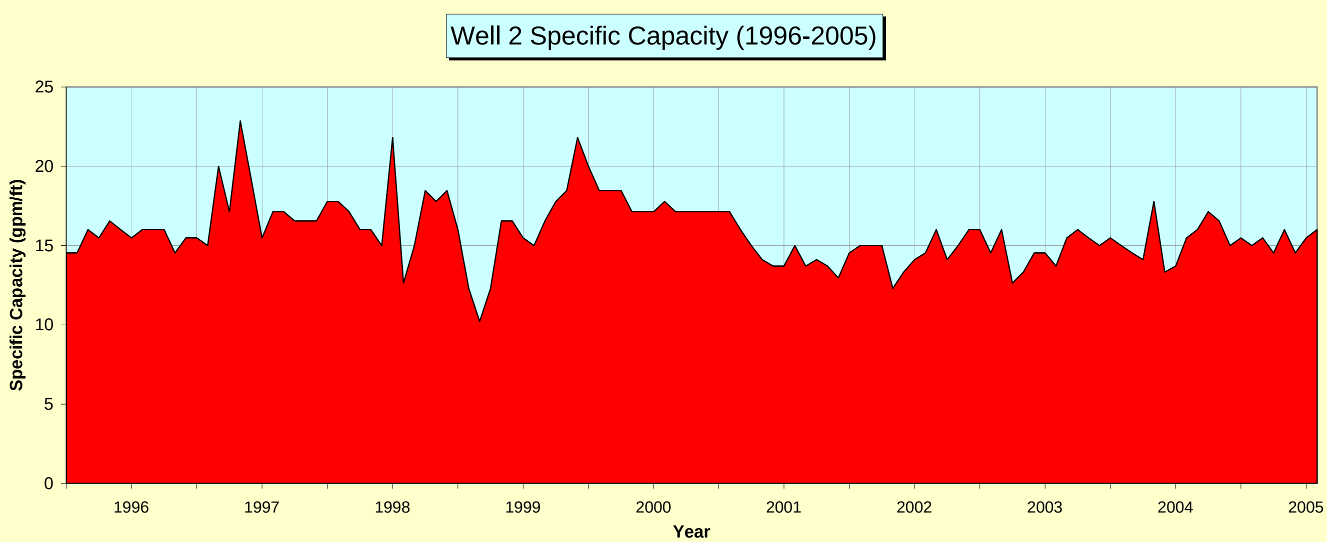
JANUARY 2006

SISTB0502.00

X:\S\SISTB\050200_UTILITIES\Project\Sister Bay copy\Report\Appendix\A\[WELLPERF final.XLS]WELL 1





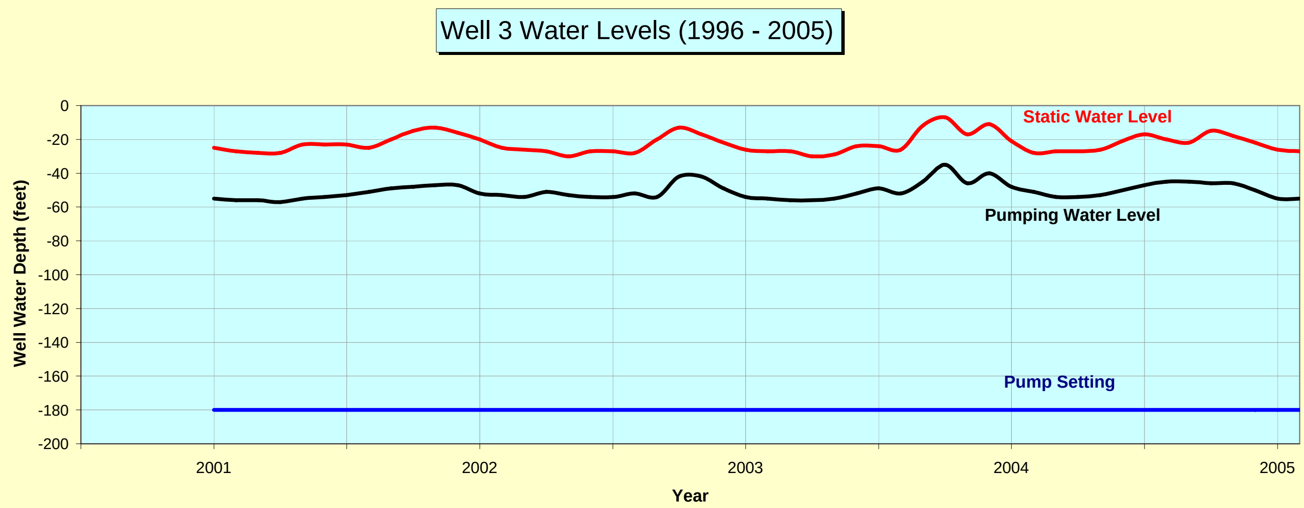


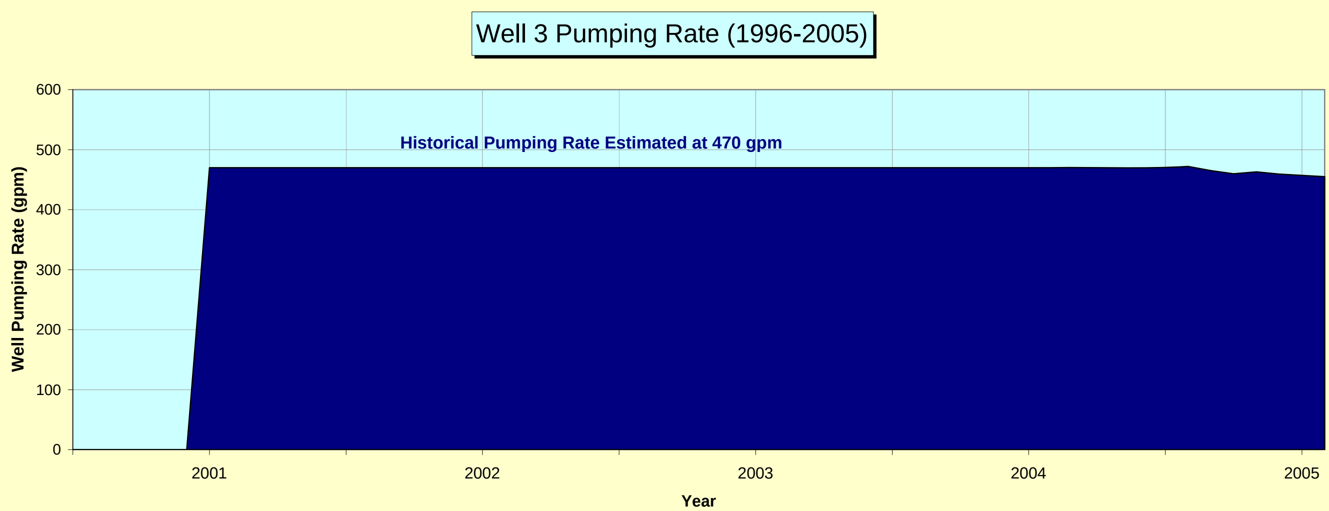
SHORT ELLIOTT
HENDRICKSON

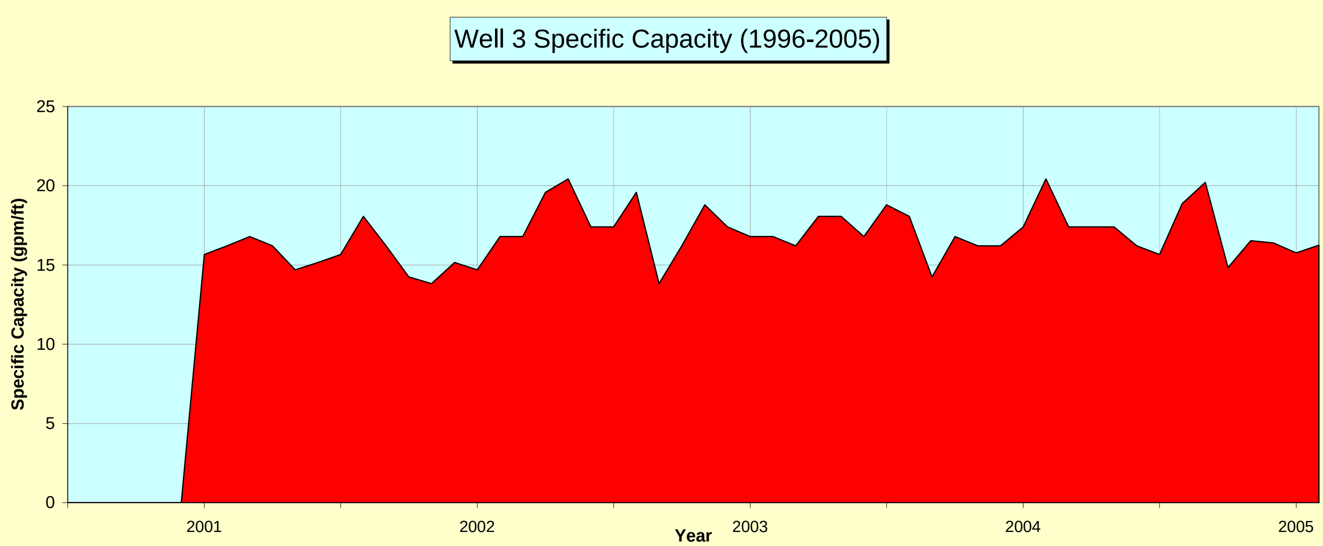
FIGURE A-2
HISTORICAL PERFORMANCE: WELL 2
SISTER BAY WATER UTILITY
VILLAGE OF SISTER BAY, WISCONSIN

JANUARY 2006 SISTB0502.00

X:\S\SISTB\050200_UTILITIES\Project\Sister Bay copy\Report\Appendix\A\[WELLPERF final.XLS]WELL 2







SHORT ELLIOTT
HENDRICKSON

SEH

FIGURE A-3

HISTORICAL PERFORMANCE: WELL 3

SISTER BAY WATER UTILITY

VILLAGE OF SISTER BAY, WISCONSIN

JANUARY 2006

SISTB0502.00

X:\S\SISTB\050200_UTILITIES\Project\Sister Bay copy\Report\Appendix\A\[WELLPERF final.XLS]WELL 3

APPENDIX B

WATER SYSTEM FIELD TESTING SUMMARY

FLOW & PRESSURE TEST

Sister Bay Water Utility
Village of Sister Bay, Wisconsin

Test Number: **F-1**

Date: *September 26, 2005* **Time:** *02:30 PM* **Area of City:** *North*

FLOWING HYDRANT **Pressure Zone:** *HLPZ*

Location: *Last hydrant on North Hillcrest Drive* Hydrant Number: *254*

RESIDUAL HYDRANT

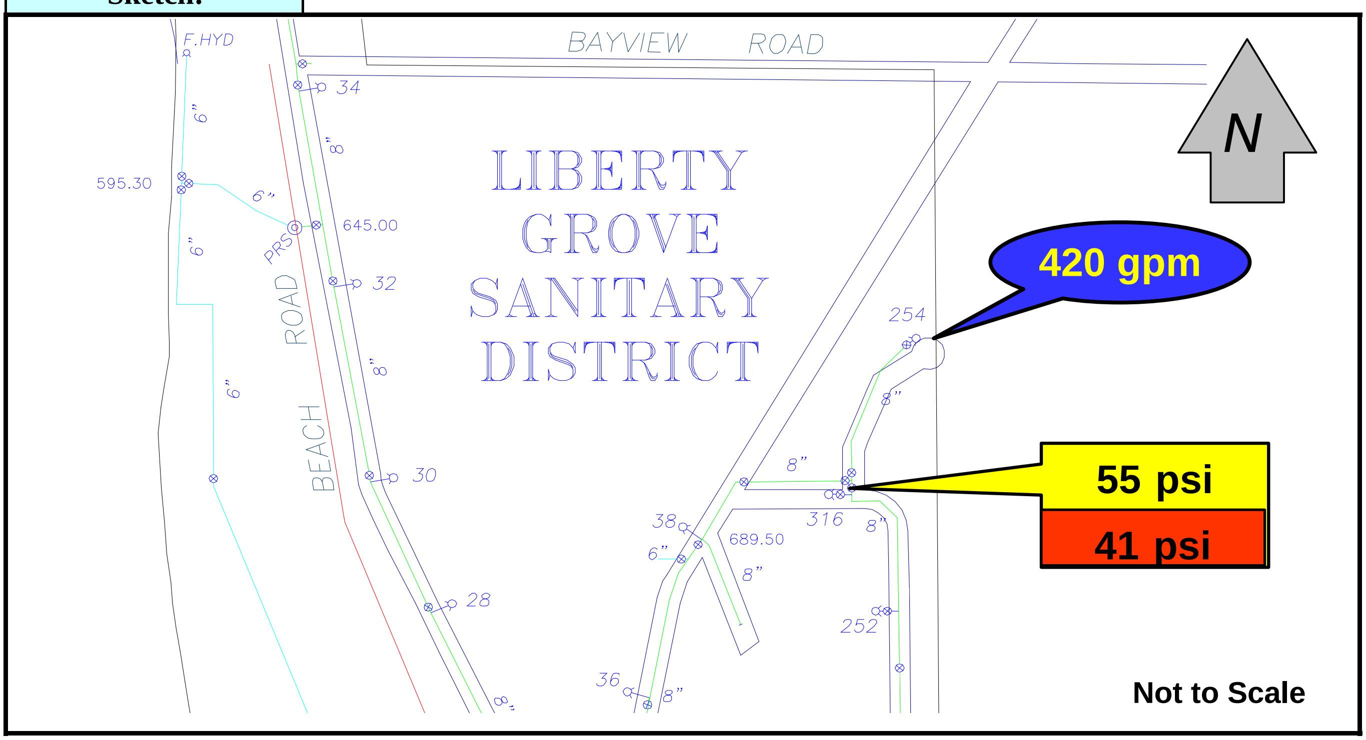
Location: *1st hydrant south of flowing hydrant* Hydrant Number: *316*

Test Hydrant	Flow (gpm)	Static Pressure (psi)	Residual Pressure (psi)	Hydrant No.
Flowing: 2.5 inch Dia.	420			254
Flowing: 2.5 inch Dia.				
Residual Hydrant		55	41	316

Test Nozzle Size **1 1/2** inches

Tower Elevations			Pumps Operating			
Jungwirth Court	21.9	*feet*	Well 1	OFF	Booster 1	OFF
Standpipe	16.3	*feet*	Well 2	OFF	Booster 2	OFF
			Well 3	ON	Booster 3	ON
					Booster 4	Offline

Sketch:



Remarks: *None*

C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[f-x.xls]f-1

FLOW & PRESSURE TEST

Sister Bay Water Utility
Village of Sister Bay, Wisconsin

Test Number: **F-2**

Date: *September 27, 2005* **Time:** *08:45 AM* **Area of City:** *North*

FLOWING HYDRANT — **Pressure Zone:** *HLPZ*

Location: *North end of Beach Road* Hydrant Number: *314*

RESIDUAL HYDRANT

Location: *Beach Road and Bayview Road* Hydrant Number: *34*

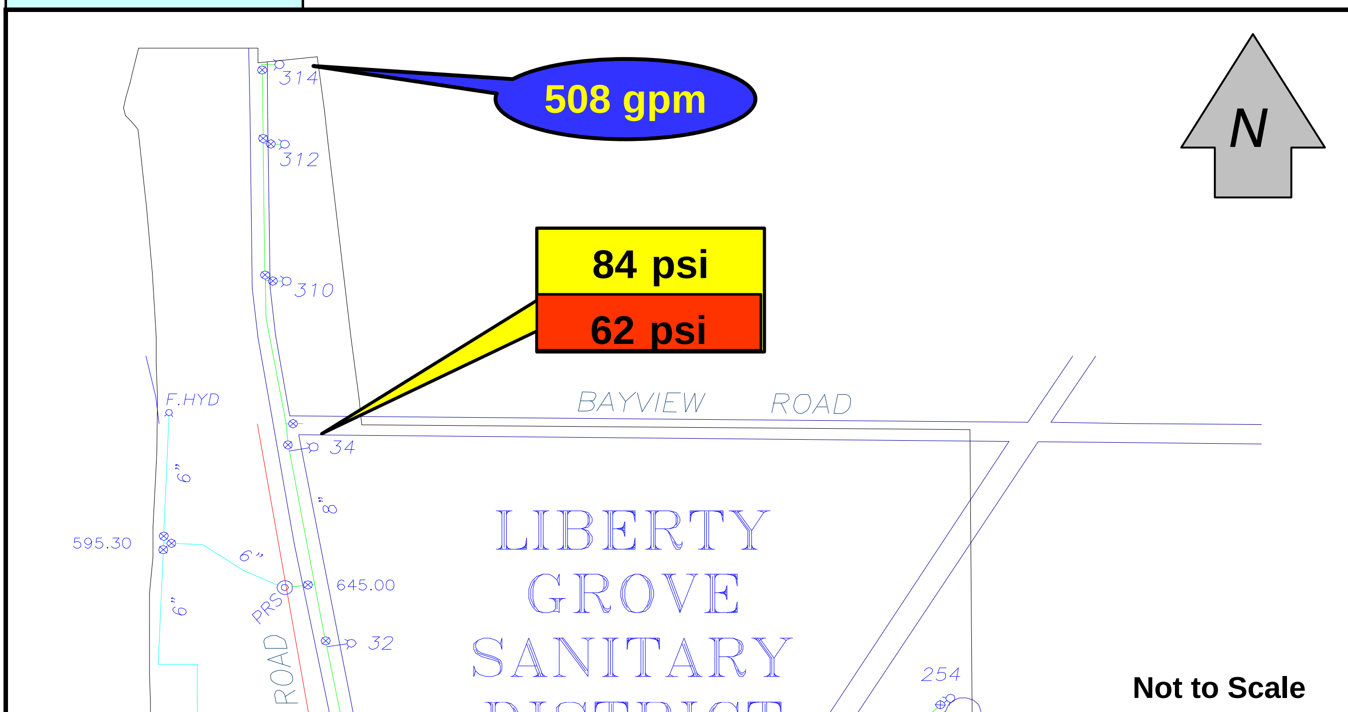
Test Hydrant	Flow (gpm)	Static Pressure (psi)	Residual Pressure (psi)	Hydrant No.
Flowing: 2.5 inch Dia.	508			314
Flowing: 2.5 inch Dia.				
Residual Hydrant		84	62	34

Test Nozzle Size **1 1/2** inches

Tower Elevations		
Jungwirth Court	20.5	feet
Standpipe	13.0	feet

Pumps Operating			
Well 1	ON	Booster 1	OFF
Well 2	OFF	Booster 2	OFF
Well 3	ON	Booster 3	ON
		Booster 4	Offline

Sketch:



Remarks: *None*

C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[f-x.xls]f-2

FLOW & PRESSURE TEST

Sister Bay Water Utility
Village of Sister Bay, Wisconsin

Test Number: **F-3**

Date: *September 27, 2005* **Time:** *09:20 AM* **Area of City:** *North*

FLOWING HYDRANT **Pressure Zone:** *Main*

Location: *Bay Shore Dr. - 3rd Hyd. south of Waters End* Hydrant Number: *18*

RESIDUAL HYDRANT

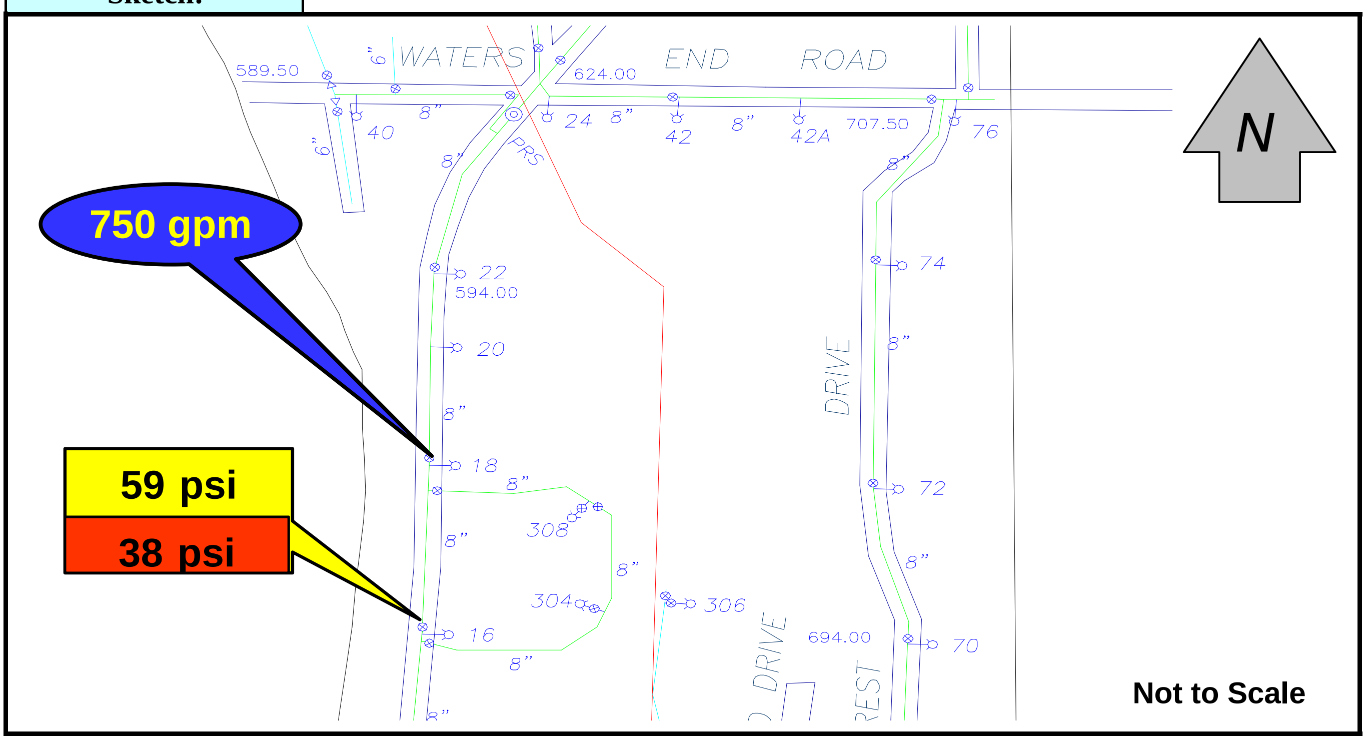
Location: *Bay Shore Dr. - 4th Hyd. south of Waters End* Hydrant Number: *16*

Test Hydrant	Flow (gpm)	Static Pressure (psi)	Residual Pressure (psi)	Hydrant No.
Flowing: 2.5 inch Dia.	**750**			***18***
Flowing: 2.5 inch Dia.				
Residual Hydrant		***59***	***38***	***16***

Test Nozzle Size **2 1/2** inches

Tower Elevations			Pumps Operating			
Jungwirth Court	**22.0**	*feet*	Well 1	**ON**	Booster 1	**OFF**
Standpipe	**15.9**	*feet*	Well 2	**OFF**	Booster 2	**OFF**
			Well 3	**OFF**	Booster 3	**ON**
					Booster 4	**Offline**

Sketch:



Remarks: *None*

C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[f-x.xls]f-3

FLOW & PRESSURE TEST

Sister Bay Water Utility

Village of Sister Bay, Wisconsin

Test Number: **F-4**

Date: *September 27, 2005* **Time:** *09:40 AM* **Area of City:** *East*

FLOWING HYDRANT

Pressure Zone: *HLPZ*

Location: *Trillium Lane east of Birchwood Dr.* Hydrant Number: *96*

RESIDUAL HYDRANT

Location: *Birchwood Drive west of Trillium Ln.* Hydrant Number: *62*

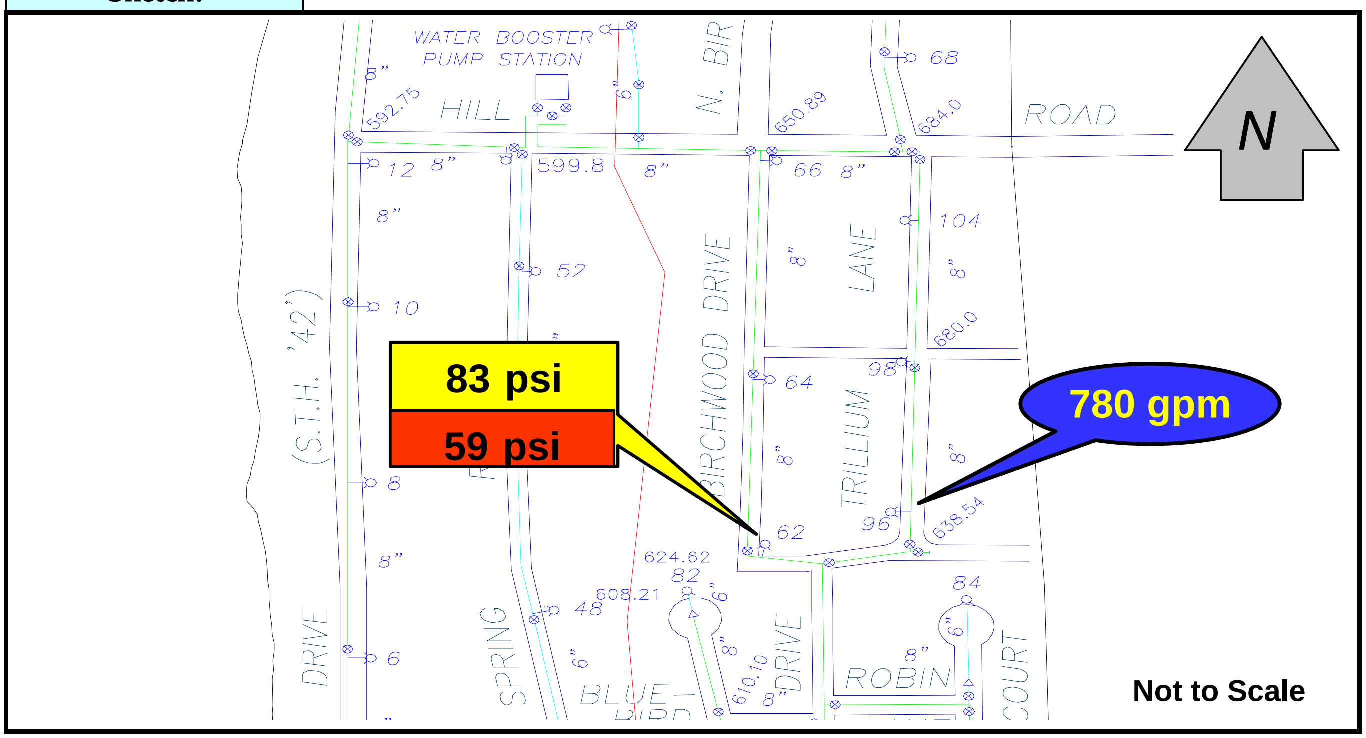
Test Hydrant	Flow (gpm)	Static Pressure (psi)	Residual Pressure (psi)	Hydrant No.
Flowing: 2.5 inch Dia.	**780**			***96***
Flowing: 2.5 inch Dia.				
Residual Hydrant		***83***	***59***	***62***

Test Nozzle Size **2** inches

Tower Elevations		
Jungwirth Court	**21.0**	*feet*
Standpipe	**16.6**	*feet*

Pumps Operating			
Well 1	**OFF**	Booster 1	**OFF**
Well 2	**OFF**	Booster 2	**OFF**
Well 3	**OFF**	Booster 3	**ON**
		Booster 4	**Offline**

Sketch:



Remarks: *None*

C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[f-x.xls]f-4

FLOW & PRESSURE TEST
Sister Bay Water Utility
Village of Sister Bay, Wisconsin

Test Number: F-5

Date: *September 27, 2005* **Time:** *10:00 AM* **Area of City:** *West*

FLOWING HYDRANT **Pressure Zone:** *HLPZ*

Location: *North end of West Little Sister Road* Hydrant Number: *157*

RESIDUAL HYDRANT

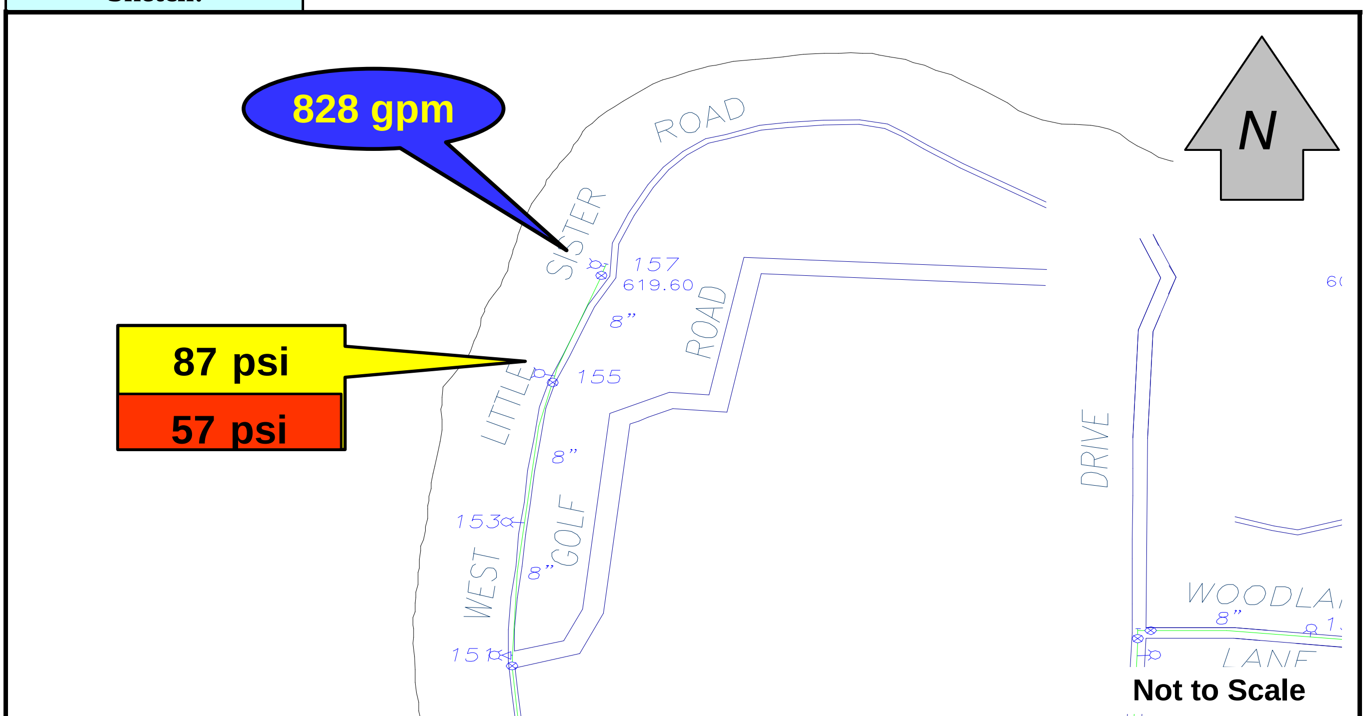
Location: *1st Hydrant south of flowing* Hydrant Number: *155*

Test Hydrant	Flow (gpm)	Static Pressure (psi)	Residual Pressure (psi)	Hydrant No.
Flowing: 2.5 inch Dia.	**828**			***157***
Flowing: 2.5 inch Dia.				
Residual Hydrant		***87***	***57***	***155***

Test Nozzle Size **2** inches

Tower Elevations			Pumps Operating			
Jungwirth Court	**20.0**	*feet*	Well 1	**OFF**	Booster 1	**OFF**
Standpipe	**16.0**	*feet*	Well 2	**OFF**	Booster 2	**OFF**
			Well 3	**OFF**	Booster 3	**ON**
					Booster 4	**Offline**

Sketch:



Remarks: *None*

C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[f-x.xls]f-5

FLOW & PRESSURE TEST

Sister Bay Water Utility

Village of Sister Bay, Wisconsin

Test Number: **F-6**

Date: *September 27, 2005* **Time:** *10:30 AM* **Area of City:** *Southwest*

FLOWING HYDRANT			
		Pressure Zone:	*HLPZ*
Location:	*Bay Shore Drive west of Forest Lane*	Hydrant Number:	*101*

RESIDUAL HYDRANT			
Location:	*Bay Shore Drive and Meadow Lane*	Hydrant Number:	*97*

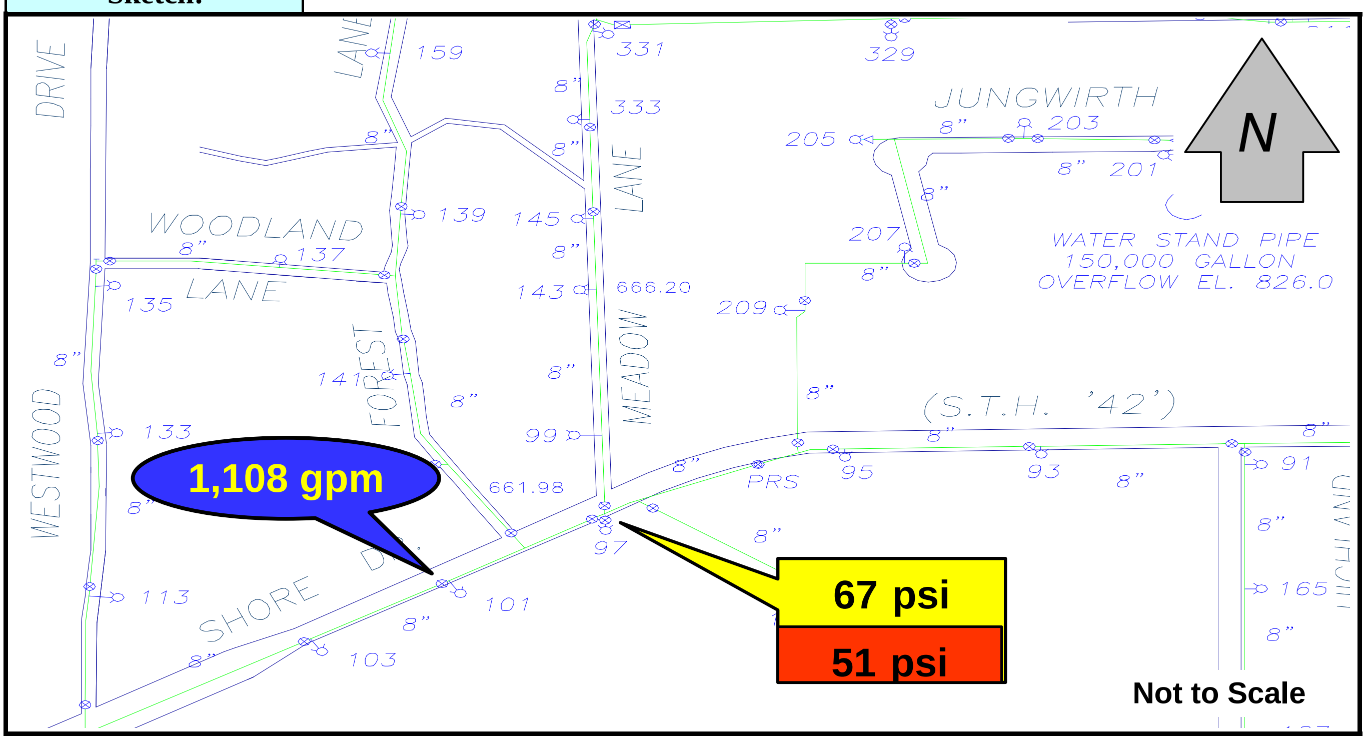
Test Hydrant	Flow (gpm)	Static Pressure (psi)	Residual Pressure (psi)	Hydrant No.
Flowing: 2.5 inch Dia.	**1,108**			***101***
Flowing: 2.5 inch Dia.				
Residual Hydrant		***67***	***51***	***97***

Test Nozzle Size **2 1/2** inches

Tower Elevations		
Jungwirth Court	**18.7**	*feet*
Standpipe	**15.2**	*feet*

Pumps Operating			
Well 1	**OFF**	Booster 1	**OFF**
Well 2	**OFF**	Booster 2	**OFF**
Well 3	**ON**	Booster 3	**ON**
		Booster 4	**Offline**

Sketch:



Remarks: *Well 3 just turned on - could have been off during test*

FLOW & PRESSURE TEST

Sister Bay Water Utility

Village of Sister Bay, Wisconsin

Test Number: **F-7**

Date: *September 27, 2005* **Time:** *11:00 AM* **Area of City:** *West*

FLOWING HYDRANT		**Pressure Zone:**	*HLPZ*
Location:	*West end of Sunnyside Road*	Hydrant Number:	*329*

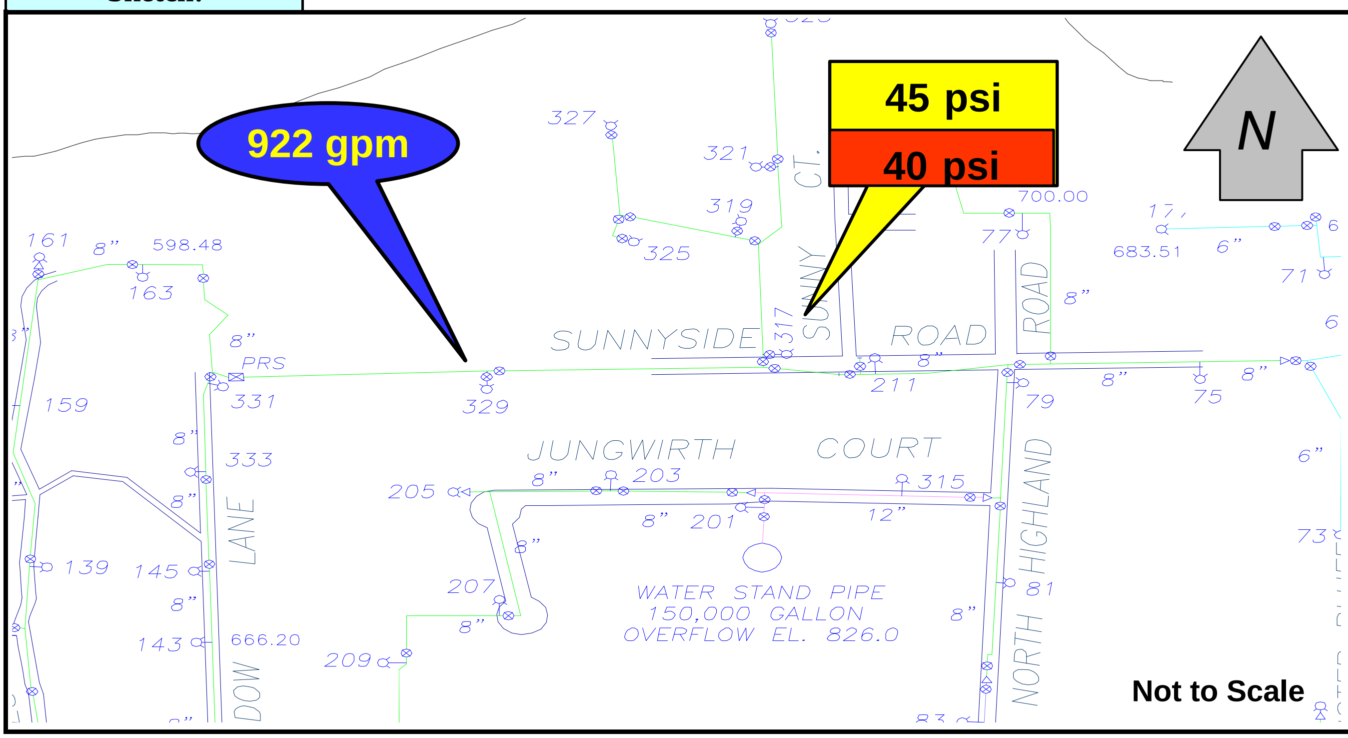
RESIDUAL HYDRANT			
Location:	*Sunnyside Road and Sunnyside Court*	Hydrant Number:	*317*

Test Hydrant	Flow (gpm)	Static Pressure (psi)	Residual Pressure (psi)	Hydrant No.
Flowing: 2.5 inch Dia.	**922**			***329***
Flowing: 2.5 inch Dia.				
Residual Hydrant		***45***	***40***	***317***

Test Nozzle Size **2 1/2** inches

Tower Elevations			Pumps Operating			
Jungwirth Court	**19.9**	*feet*	Well 1	**OFF**	Booster 1	**OFF**
Standpipe	**14.5**	*feet*	Well 2	**OFF**	Booster 2	**OFF**
			Well 3	**ON**	Booster 3	**ON**
					Booster 4	**Offline**

Sketch:



Remarks: *None*

FLOW & PRESSURE TEST

Sister Bay Water Utility
Village of Sister Bay, Wisconsin

Test Number: **F-8**

Date: *September 27, 2005* **Time:** *11:45 AM* **Area of City:** *South*

FLOWING HYDRANT **Pressure Zone:** *HLPZ*

Location: *Cherrywood Lane and Koessl Lane* Hydrant Number: *223*

RESIDUAL HYDRANT

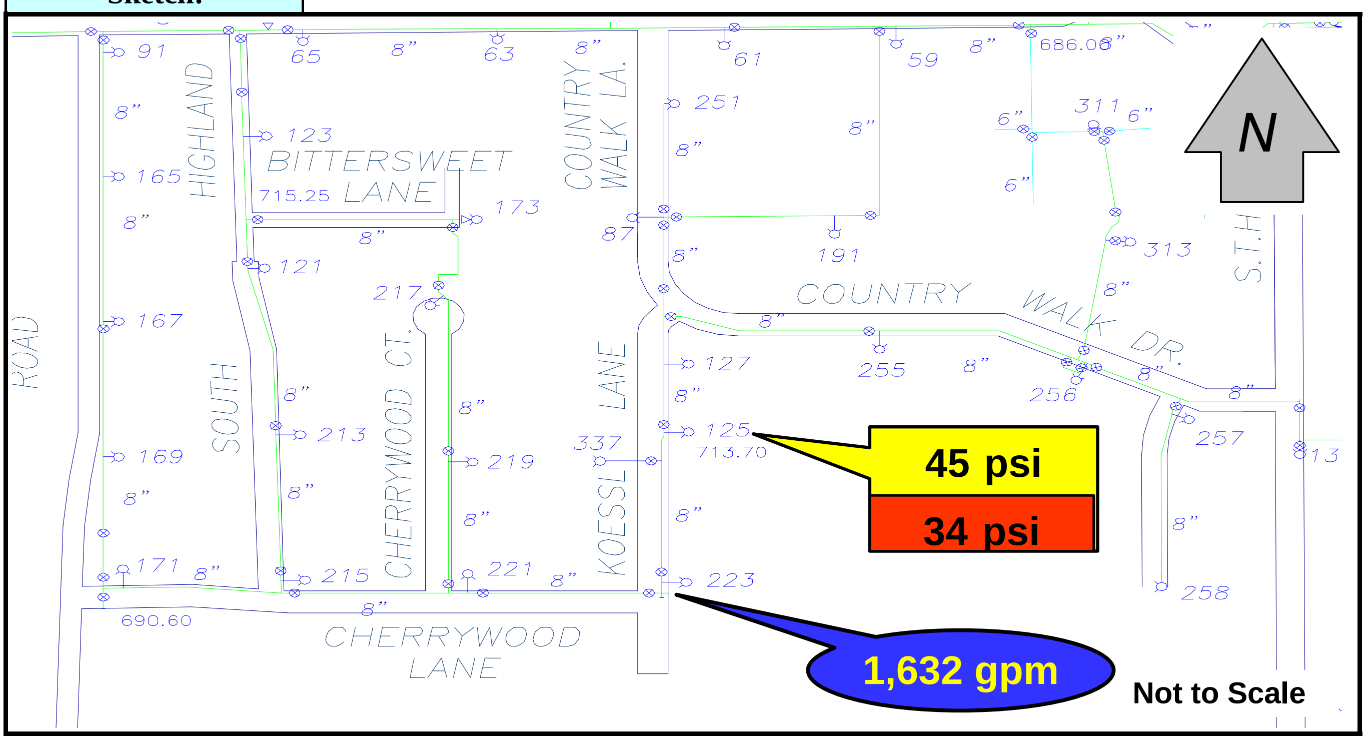
Location: *2nd Hydrant north of flowing* Hydrant Number: *125*

Test Hydrant	Flow (gpm)	Static Pressure (psi)	Residual Pressure (psi)	Hydrant No.
Flowing: 2.5 inch Dia.	**816**			***223***
Flowing: 2.5 inch Dia.	**816**			***223***
Residual Hydrant		***45***	***34***	***125***

Test Nozzle Size **2 1/2** inches

Tower Elevations			Pumps Operating			
Jungwirth Court	**20.5**	*feet*	Well 1	**OFF**	Booster 1	**OFF**
Standpipe	**13.2**	*feet*	Well 2	**OFF**	Booster 2	**OFF**
			Well 3	**OFF**	Booster 3	**ON**
					Booster 4	**Offline**

Sketch:



Remarks: *None*

C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[f-x.xls]f-8

FLOW & PRESSURE TEST
Sister Bay Water Utility
Village of Sister Bay, Wisconsin

Test Number: F-9

Date: *September 27, 2005* **Time:** *01:15 PM* **Area of City:** *East*

Pressure Zone: *HLPZ*

FLOWING HYDRANT

Location: *Last Hydrant east of WWTP* Hydrant Number: *318*

RESIDUAL HYDRANT

Location: *1st Hydrant west of flowing* Hydrant Number: *186*

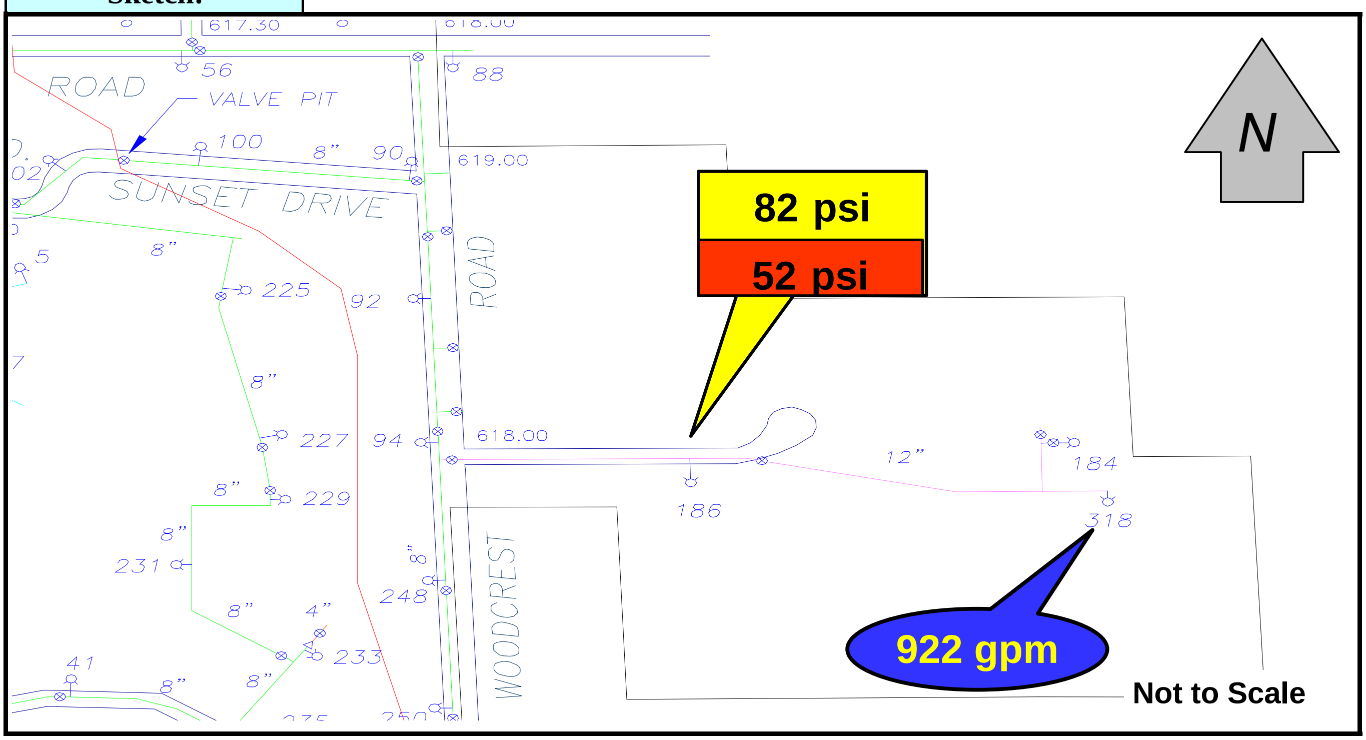
Test Hydrant	Flow (gpm)	Static Pressure (psi)	Residual Pressure (psi)	Hydrant No.
Flowing: 2.5 inch Dia.	922			318
Flowing: 2.5 inch Dia.				
Residual Hydrant		82	52	186

Test Nozzle Size **2 1/2** inches

Tower Elevations		
Jungwirth Court	18.6	*feet*
Standpipe	16.4	*feet*

Pumps Operating			
Well 1	OFF	Booster 1	OFF
Well 2	OFF	Booster 2	OFF
Well 3	OFF	Booster 3	OFF
		Booster 4	ON

Sketch:



Remarks: *None*

C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[f-x.xls]f-9

FLOW & PRESSURE TEST

Sister Bay Water Utility
Village of Sister Bay, Wisconsin

Test Number: **F-10**

Date: *September 27, 2005* **Time:** *01:35 AM* **Area of City:** *Southeast*

FLOWING HYDRANT	
Pressure Zone:	*HLPZ*
Location: *East of Smith Drive*	Hydrant Number: *245*

RESIDUAL HYDRANT	
Location: *2nd Hydrant west of flowing*	Hydrant Number: *241*

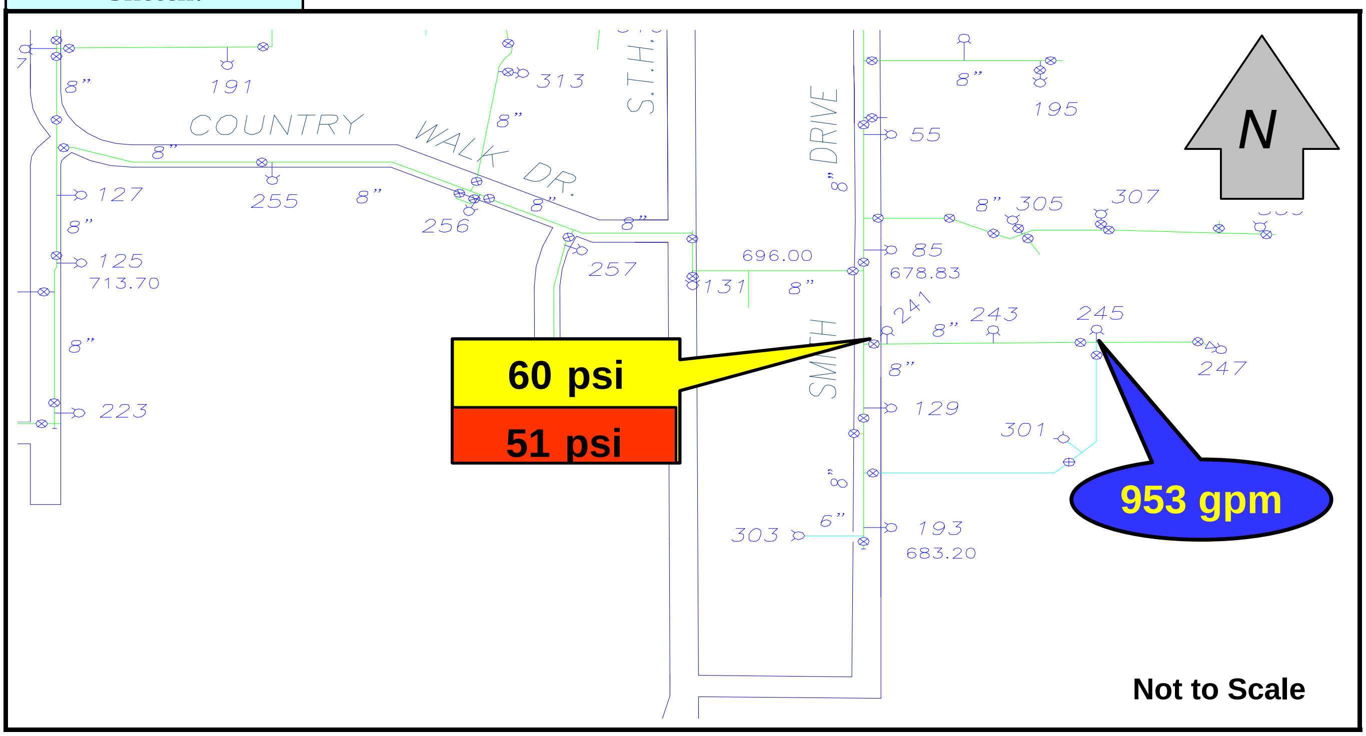
Test Hydrant	Flow (gpm)	Static Pressure (psi)	Residual Pressure (psi)	Hydrant No.
Flowing: 2.5 inch Dia.	**953**			***245***
Flowing: 2.5 inch Dia.				
Residual Hydrant		***60***	***51***	***241***

Test Nozzle Size **2 1/2** inches

Tower Elevations		
Jungwirth Court	**18.3**	*feet*
Standpipe	**15.8**	*feet*

Pumps Operating			
Well 1	**OFF**	Booster 1	**OFF**
Well 2	**OFF**	Booster 2	**OFF**
Well 3	**ON**	Booster 3	**ON**
		Booster 4	**OFF**

Sketch:



Remarks: *None*

FLOW & PRESSURE TEST

Sister Bay Water Utility
Village of Sister Bay, Wisconsin

Test Number: F-11

Date: *September 27, 2005* **Time:** *01:55 PM* **Area of City:** *Central*

FLOWING HYDRANT **Pressure Zone:** *Main*

Location: *Mill Road and South Spring Drive* Hydrant Number: *13*

RESIDUAL HYDRANT

Location: *Mill Road and Park Lane* Hydrant Number: *37*

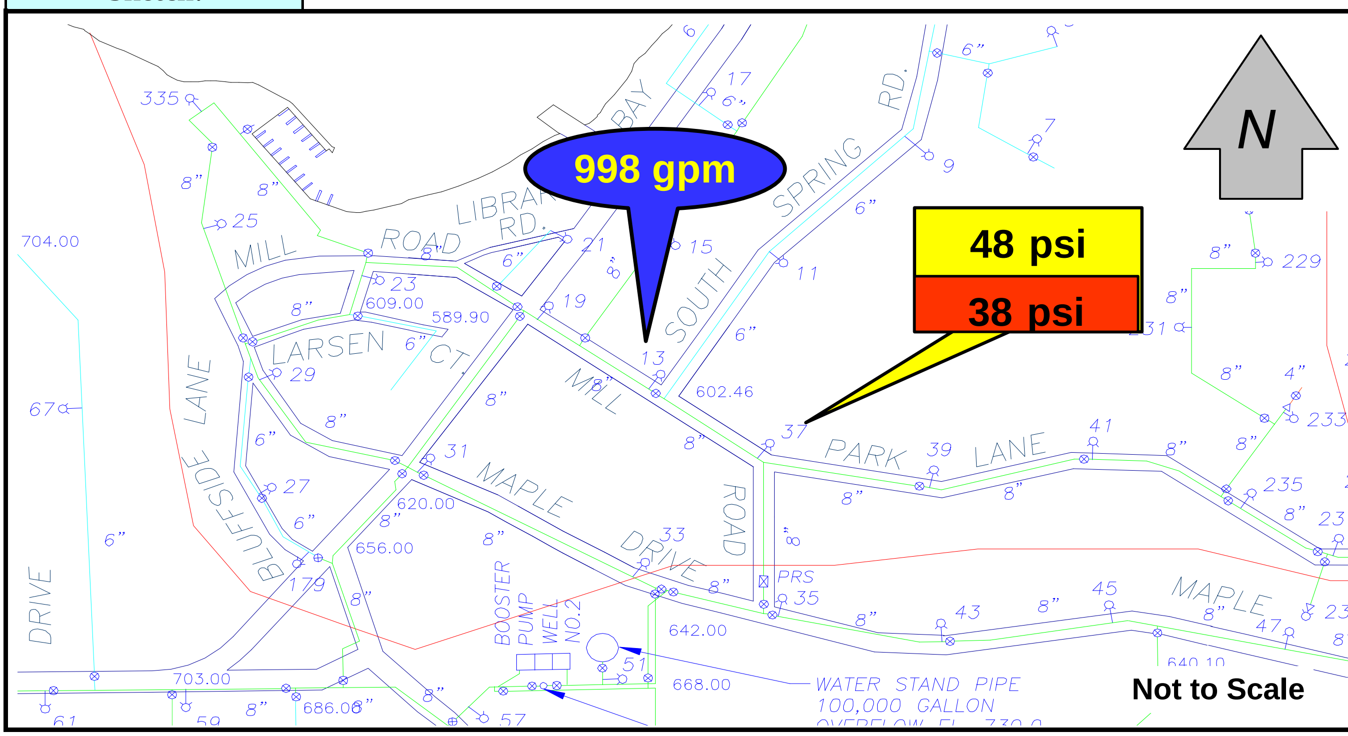
Test Hydrant	Flow (gpm)	Static Pressure (psi)	Residual Pressure (psi)	Hydrant No.
Flowing: 2.5 inch Dia.	998			13
Flowing: 2.5 inch Dia.				
Residual Hydrant		48	38	37

Test Nozzle Size **2 1/2** inches

Tower Elevations		
Jungwirth Court	20.1	*feet*
Standpipe	14.8	*feet*

Pumps Operating			
Well 1	OFF	Booster 1	OFF
Well 2	OFF	Booster 2	OFF
Well 3	ON	Booster 3	ON
		Booster 4	OFF

Sketch:



Remarks: *None*

C:\Documents and Settings\pplanton\My Documents\Sister Bay copy\[f-x.xls]f-11

C-VALUE TEST

Sister Bay Water Utility
Village of Sister Bay, Wisconsin

Test Number: *C-1*

Date: *September 27, 2005* **Time:** *02:20 PM* **Area of City:** *Central*

FLOWING HYDRANT

Location: *Bay Shore Drive Hydrant #10*

RESIDUAL HYDRANT #1

Location: *Bay Shore Drive Hydrant #8*

VALVES CLOSED

Location: *1st valve north of flowing hydrant*

RESIDUAL HYDRANT #2

Location: *Bay Shore Drive and Scandia Road - Hydrant #2*

Hydrant	Pressure (psi)	Flow (gpm)	Static Pressure (psi)	Residual Pressure (psi)	Hydraulic Grade Line (feet)
Flowing: 2.5 inch Dia.	*20*	*830*			
Residual Hydrant #1			*56*	*37*	*43.9*
Residual Hydrant #2			*55*	*53*	*4.6*

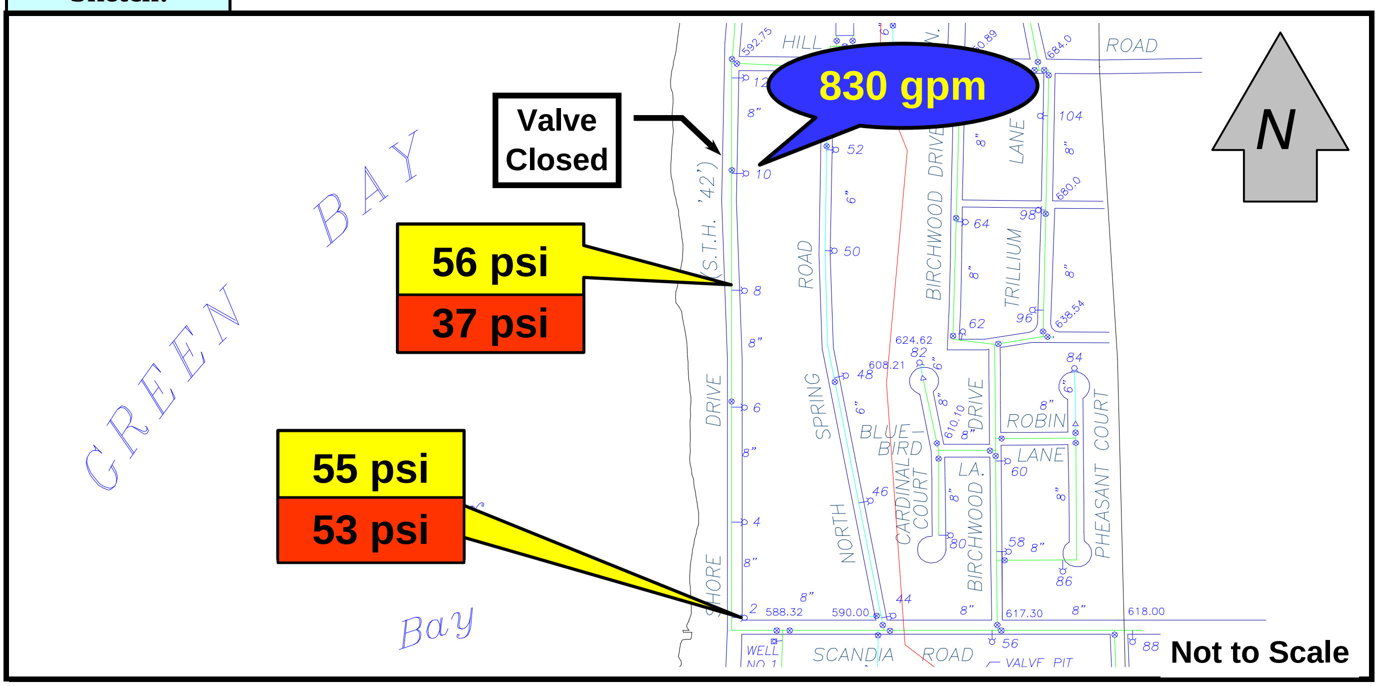
Test Nozzle Size: *2 1/2* inches Number of Barrels: 1 (1 or 2)

Pipe Diameter: *8* inches **Distance Between Residual Hydrants:** *1,556* feet

Pipe Material: *Ductile Iron* **Year Installed:** *1972* **Pipe Age:** *33* years

Calculated C-Value: **91**

Sketch:



Remarks:

X:\S\SISTB\050200_UTILITIES\Project\Water\Field Test\C Value\[c_x final.xls]c-1

C-VALUE TEST

Sister Bay Water Utility
Village of Sister Bay, Wisconsin

Test Number: *C-2*

Date: *September 27, 2005* **Time:** *02:45 PM* **Area of City:** *Central*

FLOWING HYDRANT

Location: *Spring Street Hydrant #52*

RESIDUAL HYDRANT #1

Location: *Spring Street Hydrant #50*

VALVES CLOSED

Location: *1st valve north of flowing hydrant*

RESIDUAL HYDRANT #2

Location: *Spring Street Hydrant #44*

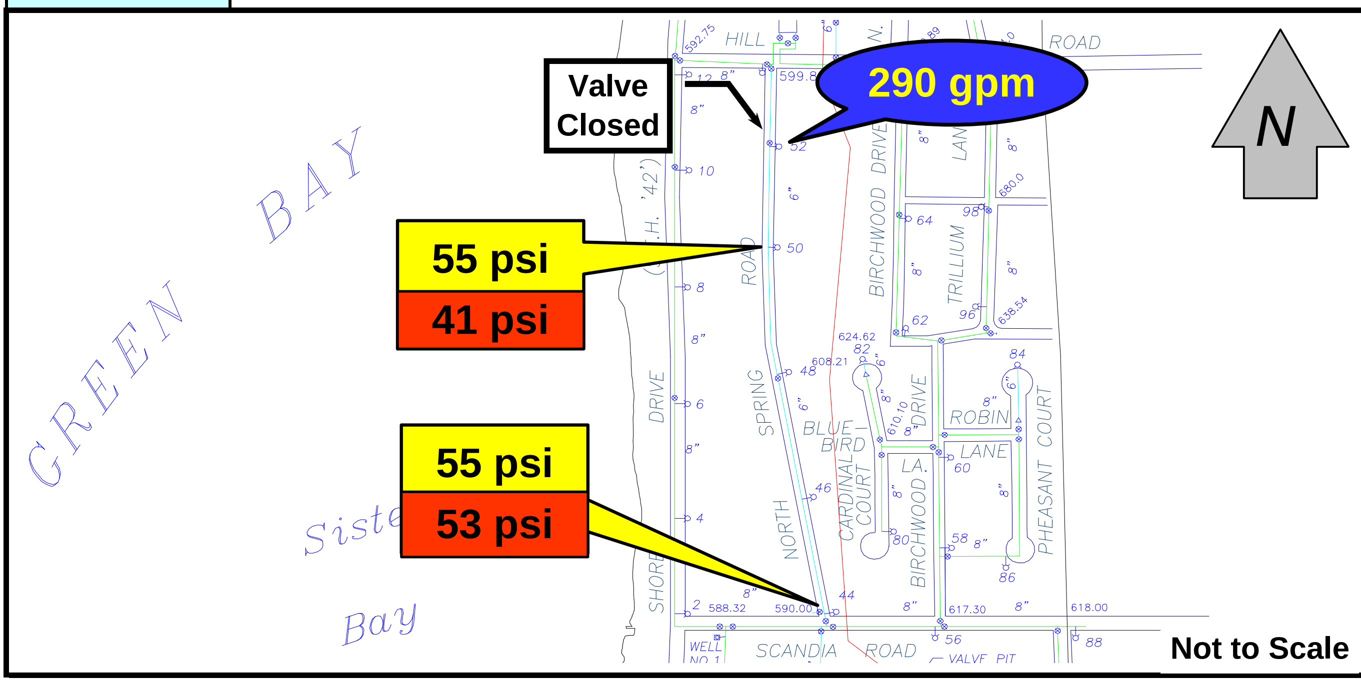
Hydrant	Pressure (psi)	Flow (gpm)	Static Pressure (psi)	Residual Pressure (psi)	Hydraulic Grade Line (feet)
Flowing: 2.5 inch Dia.	*38*	*290*			
Residual Hydrant #1			*55*	*41*	*32.3*
Residual Hydrant #2			*55*	*53*	*4.6*

Test Nozzle Size: *1 1/4* inches Number of Barrels: 1 (1 or 2)

Pipe Diameter: *6* inches **Distance Between Residual Hydrants:** *1,727* feet
Pipe Material: *Ductile Iron* **Year Installed:** *1972* **Pipe Age:** *33* years

Calculated C-Value: **86**

Sketch:



Remarks:

X:\S\SISTB\050200_UTILITIES\Project\Water\Field Test\C Value\[c_x final.xls]c-2

APPENDIX C

H&H ANALYSIS SUMMARY

H&H ANALYSIS DATA SUMMARY
COMPREHENSIVE UTILITY PLANNING AREA
SISTER BAY, WISCONSIN

	Watershed Peak Discharge (cfs)						Natural Closed Depression Flood Information			
	Existing Conditions			Proposed Conditions			Elevations (USGS)			
Storm Event Return Period (yrs)	2	10	100	2	10	100	Existing Low Adjacent Grade of Development	100 Year Ponded Elevation		Escape Route
24 hour Rainfall Depth (inches)	2.4	3.6	4.9	2.4	3.6	4.9		Existing Conditions	Future Developed Conditions	
Watershed Number										
300	4.0	10.7	19.0	5.6	13.9	22.2	710	706.6	706.6	710
400	8.0	17.5	29.7	9.7	19.8	31.4	none	none	none	
500	11.7	24.7	40.1	14.0	27.7	43.5	678	676.2	676.3	678
600	6.1	16.3	29.3	8.5	19.8	33.8	none	none	none	
800	10.7	34.9	67.3	16.7	44.4	79.6	none	660.6	660.6	662
900	14.1	77.7	180.6	26.3	104.3	219.0	718	721.0	721.7	722
1000	16.5	64.0	137.0	28.7	88.4	172.0	644	646.2	646.3	646
1200	10.7	28.5	51.1	15.0	34.8	59.0	none	none	none	
1300	0.4	6.9	22.4	0.8	9.1	26.3		695.5	695.6	698
1400	11.7	30.0	54.0	16.0	36.5	61.5	none	none	none	
1900	2.0	8.0	52.0	8.0	26.0	76.0	none	637.3	637.5	637
2000	1.0	19.0	20.5	9.4	35.0	48.0	642	632.0	634.0	640
Sum 1400-2000	**14.7**	**57.0**	**126.5**	**33.4**	**97.5**	**185.5**				
1500	8.2	30.9	62.6	15.0	42.2	77.8	684	685.0	685.0	688
1600	8.3	40.6	88.3	17.9	58.0	112.4	none	694.0	694.3	696
2200	20.4	41.7	66.4	24.0	46.2	71.4	none	none	none	
2300	3.8	15.5	31.8	16.7	34.8	55.7	632	623.0	625.0	636
2500	0.0	1.5	6.9	1.0	6.5	14.6	590	589.0	589.0	590
2600	15.2	44.4	82.6	21.9	54.7	95.9	none	none	none	
2701	16.2	43.8	80.0	18.8	48.0	85.0	none	none	none	
2702	21.5	59.5	109.0	34.0	78.0	133.0	none	none	none	
2703	31.3	107.0	216.0	45.9	130.0	246.0	583	unknown	unknown	588
Total 2700	**69.0**	**210.3**	**405.0**	**98.7**	**256.0**	**464.0**				
2800	23.6	51.7	85.5	27.9	57.6	92.2		none	none	

	Watershed Peak Discharge (cfs)						Natural Closed Depression Flood Information			
	Existing Conditions			Proposed Conditions			Elevations (USGS)			
Storm Event Return Period (yrs)	2	10	100	2	10	100	Existing Low Adjacent Grade of Development	100 Year Ponded Elevation		Escape Route
24 hour Rainfall Depth (inches)	2.4	3.6	4.9	2.4	3.6	4.9		Existing Conditions	Future Developed Conditions	
Watershed Number										
2900	4.0	17.5	39.5	8.5	27.0	53.4	614			618
3000	1.9	13.4	35.0	6.3	24.5	52.4	630	630.0	631.0	634
3101	28.2	107.0	221.0	47.7	142.0	268.0	600	600.0	600.0	600
3102	1.3	10.6	28.2	9.1	28.0	53.5	none	616.0	616.2	616
3103	0.6	7.6	19.7	11.0	25.7	43.7	none	none	none	
3104	2.7	19.0	49.6	15.6	48.2	93.5	none	none	none	
3105 & 3106	0.0	3.8	27.9	2.0	26.5	84.9	630	628.2	629.0	632
Total 3100	**32.8**	**148.0**	**346.4**	**85.4**	**270.4**	**543.6**				
3200	3.7	16.4	35.1	3.7	16.4	35.1	626	628.4	628.4	630
3300	16.6	50.9	61.2	26.4	66.9	118.0	none	none	none	
3400	Sheet flow to Green Bay						none	none	none	
3500	3.8	9.9	17.7	5.5	12.5	20.9	none	none	none	
3601	46.4	156.0	315.0	77.9	214.0	394.0	648	647.0	647.1	646
3602 & 3603	23.3	75.1	146.4	31.8	89.7	166.0	642	644.6	645.4	648
3602 & 3603	0.4	30.2	83.0	4.6	42.7	91.3				
3604	5.6	18.8	36.7	8.3	23.1	42.3	none	627.8	627.8	628
3605	0.0	0.1	0.8	0.1	2.4	7.8	none	601.0	601.0	600
3615	12.3	24.6	38.8	13.6	26.2	40.6	none	none	none	
3625	0.0	0.4	1.1	0.0	0.4	1.1	none	none	none	
3635	0.3	4.4	11.1	0.9	5.7	13.0	none	none	none	
3645 & 3655	3.4	11.3	22.2	4.3	13.0	24.4	none	none	none	
3665	1.5	4.8	9.1	1.8	5.2	9.6	622			622
3675 & 3685	9.0	25.0	45.7	11.4	28.8	50.7	none	none	none	
Total 3600	**106.0**	**360.6**	**727.6**	**160.2**	**463.7**	**861.7**				
3800	3.8	9.9	17.7	No change						

	Watershed Peak Discharge (cfs)						Natural Closed Depression Flood Information			
	Existing Conditions			Proposed Conditions			Elevations (USGS)			
Storm Event Return Period (yrs)	2	10	100	2	10	100	Existing Low Adjacent Grade of Development	100 Year Ponded Elevation		Escape Route
24 hour Rainfall Depth (inches)	2.4	3.6	4.9	2.4	3.6	4.9		Existing Conditions	Future Developed Conditions	
Watershed Number										
3901	36.9	66.0	98.0	41.6	71.1	103.2	none	none	none	
3902	31.9	62.7	97.9	31.9	62.7	97.9	none	none	none	
3903	9.0	17.2	26.5	9.0	17.2	26.5	none	none	none	
Total 3901-3903	**77.8**	**145.9**	**222.4**	**82.5**	**151.0**	**227.6**				
4001	14.9	32.3	53.0	17.4	35.6	56.9	none	none	none	
4002	23.3	52.7	88.5	28.1	59.4	96.4	674	670.0	670.0	670
4002a			8.9			8.9	583	586.0	586.0	586+
Total 4000	**38.2**	**85.0**	**141.5**	**45.5**	**95.0**	**153.3**				
4401	12.7	35.2	64.6	31.6	62.0	97.0	684	683.0	683.0	682
4402	21.2	65.4	126.4	37.2	91.6	160.0	673	673.4	673.6	674
Total 4400	**33.9**	**100.6**	**191.0**	**68.8**	**153.6**	**257.0**				
5200	0.8	3.3	7.3	4.2	9.0	14.7	none	731.6	732.4	733
5500	60.9	152.0	266.0	88.4	191.0	314.0	none	none	none	
5800	1.7	4.5	8.2	4.5	8.3	12.6	none	none	none	
5900	0.8	1.7	2.7	No change			none	none	none	
6000	14.2	29.2	46.7	17.8	33.7	51.7	none	none	none	
6100	25.8	50.8	79.6	37.2	64.1	93.3	none	none	none	
6301	4.6	12.3	21.4	No change			584	582.4	582.4	586
6302	5.1	8.8	13.9	No change			none	none	none	
Total 6300	**9.7**	**21.1**	**35.3**	**0.0**	**0.0**	**0.0**				
6500	13.2	37.0	69.9	19.8	48.3	84.1	none	none	none	

STORM WATER STORAGE ZONE SUMMARY
COMPREHENSIVE UTILITY PLANNING AREA
SISTER BAY, WISCONSIN

Storm Water Storage Zones		
Watershed No.	Area (acres)	ELEVATION feet NGVD 29
300	1.91	708
500	4.07	678
800	1.95	662
900	18.31	722
1002	5.07	648
1300	2.90	696
1500	6.46	686
1600	12.09	696
1900	1.84	638
2002	4.63	634
2302	2.23	626
2500	0.88	590
2703	19.44	588
3000	6.22	632
3101	17.69	602
3102	4.69	618
3105	19.07	630
3200	3.42	630
3603	18.55	646
4002a	0.72	586
4401	6.04	684
4402	16.23	674
5200	3.83	732
6301	6.14	586

APPENDIX D

MODEL STORM WATER ORDINANCES

AND

ILLICIT DISCHARGE PROGRAM PROPOSAL

AN ORDINANCE TO CREATE CHAPTER 31 OF THE CODE OR ORDINANCS OF SISTER BAY RELATING TO THE CONTROL OF POST–CONSTRUCTION RUNOFF

FOREWORD

The Village Board of Sister Bay finds that land development or redevelopment construction projects and associated increases in impervious cover have altered the hydrologic responses of local watersheds by increasing storm water runoff rates and volumes, flooding, stream channel erosion, and sediment transport and deposition. This storm water runoff contributes to increased quantities of water-borne pollutants. Storm water runoff from development and redevelopment may adversely impact water bodies in the Village. Uncontrolled runoff has significant adverse impacts upon regional water resources and the health, safety, property and general welfare of the community, and diminish the public enjoyment and use of natural resources. Specifically, storm water runoff can:

1. Endanger water resources by reducing water quality;
2. Carry sediment, nutrients, pathogens, organic matter, heavy metals, toxins and other pollutants to lakes, streams and wetlands;
3. Degrade physical stream habitat by increasing stream bank erosion, increasing streambed scour, diminishing groundwater recharge, diminishing stream base flows and increasing stream temperature;
4. Undermine floodplain management efforts by increasing the incidence and levels of flooding;
5. Threaten public health, safety, property and general welfare by increasing major flood peaks and volumes;
6. Threaten public health, safety, property and general welfare by overtaxing storm sewers, drainage ways, and other minor drainage facilities;
7. Contaminate drinking water supplies;
8. Increase the risk of property damage and personal injury.

The purpose of the local regulation is to safeguard persons, protect property, prevent damage to the environment and promote the public welfare by guiding, regulating, and controlling the design, construction, use, and maintenance of any development, redevelopment or other activity which adversely impacts runoff in Sister Bay. The intent of this ordinance is several fold: 1. To require use of best management practices to reduce and control the hydrologic impacts and the amount storm water pollutants produced by land development or redevelopment activities, and 2. To allow Sister Bay to comply with non agricultural runoff performance standards for new development and redevelopment contained in subchapters III and IV of ch. NR 151, Wis. Adm. Code.

The Village Board of Sister Bay does hereby ordain that Chapter 31 of the Code of Ordinances of Sister Bay is created to read as follows:

CHAPTER 31

STORM WATER MANAGEMENT

POST–CONSTRUCTION STORM WATER MANAGEMENT

31.01 AUTHORITY.

(1) This ordinance is adopted by the Village Board under the authority granted by s. 62.234 Wis. Stats. This ordinance supersedes all provisions of an ordinance previously enacted under s. 62.23 Wis. Stats., that relate to storm water management regulations. Except as otherwise specified in s. 62.234 Wis. Stats., s. 62.23, Wis. Stats., applies to this ordinance and to any amendments to this ordinance.

(2) The provisions of this ordinance are deemed not to limit any other lawful regulatory powers of the same governing body.

(3) The Village Board hereby designates the Public Works Department to administer and enforce the provisions of this ordinance.

(4) The requirements of this ordinance do not pre– empt more stringent storm water management requirements that may be imposed by any of the following:

(a) Wisconsin Department of Natural Resources administrative rules, permits or approvals including those authorized under ss. 281.16 and 283.33, Wis. Stats.

(b) Targeted non–agricultural performance standards promulgated in rules by the Wisconsin Department of Natural Resources under s. NR 151.004, Wis. Adm. Code.

31.02 FINDINGS OF FACT.

The Village Board finds that uncontrolled, post– construction runoff has a significant impact upon water resources and the health, safety and general welfare of the community and diminishes the public enjoyment and use of natural resources.

31.03 PURPOSE AND INTENT.

(1) PURPOSE. The general purpose of this ordinance is to establish long–term, post– construction runoff management requirements that will diminish the storm water related threats to public health, safety, welfare and the aquatic environment. Specific purposes are to:

(a) Further the maintenance of safe and healthful conditions.

(b) Prevent and control the adverse effects of storm water; prevent and control soil erosion; prevent and control water pollution; protect spawning grounds, fish and aquatic life; control building sites, placement of structures and land uses; preserve ground cover and scenic beauty; and promote sound economic growth.

(c) Prevent exceedance of the safe capacity of existing drainage facilities and receiving water bodies; prevent undue channel erosion; control increases in the scouring and transportation of particulate matter; and prevent conditions that endanger downstream property.

(2) INTENT. It is the intent of the Village Board that this ordinance regulates post–construction storm water discharges to waters of the state. This ordinance may be applied on a site–by–site basis. The Village Board recognizes, however, that the preferred method of achieving the storm water performance standards set forth in this ordinance is through the preparation and implementation of comprehensive, systems–level storm water management plans that cover hydrologic units, such as watersheds, on a municipal and regional scale. Such plans may prescribe regional storm water devices, practices or systems, any of which may be designed to treat runoff from more than one site prior to discharge to waters of the state. Where such plans are in conformance with the performance standards developed under s. 281.16, Wis. Stats., for regional storm water management measures and have been approved by the Village Board, it

is the intent of this ordinance that the approved plan be used to identify post–construction management measures acceptable for the community.

31.04 APPLICABILITY AND JURISDICTION.

(1) APPLICABILITY.

(a) Where not otherwise limited by law, this ordinance applies after final stabilization to a site of land disturbing construction activity meeting any of the criteria in this paragraph, unless the site is otherwise exempt under Chapter 31.04 (b).

1. A post construction or redevelopment site that had any land disturbing construction activity.

(b) A site that meets any of the criteria in this paragraph is exempt from the requirements of this ordinance.

1. A redevelopment post–construction site with no increase in impervious area.
2. A post–construction site with less than 10% connected imperviousness based on complete development of the post–construction site, provided the cumulative area of all parking lots and rooftops is less than one acre.
3. Nonpoint discharges from agricultural facilities and practices.
4. Nonpoint discharges from silviculture activities.
5. Routine maintenance for project sites under 5 acres of land disturbance if performed to maintain the original line and grade, hydraulic capacity or original purpose of the facility.
6. Underground utility construction such as water, sewer and fiberoptic lines. This exemption does not apply to the construction of any above ground structures associated with utility construction.

(c) Notwithstanding the applicability requirements in Chapter 31.04(1) (a), this ordinance applies to post– construction sites of any size that, in the opinion of the Department of Public Works, is likely to result in runoff that exceeds the safe capacity of the existing drainage facilities or receiving body of water, that causes undue channel erosion, that increases water pollution by scouring or the transportation of particulate matter or that endangers property or public safety.

(2) JURISDICTION.

This ordinance applies to post construction sites within the corporate limits and jurisdiction of Sister Bay, as well as the extraterritorial division of land subject to an ordinance enacted pursuant to s. 236.45(2) and (3) Wis. Stats.

31.05 DEFINITIONS.

(1) “Administering authority” means the Department of Public Works.

(2) “Agricultural facilities and practices” has the meaning given in s. 281.16, Wis. Stats.

(3) “Average annual rainfall” means a calendar year of precipitation, excluding snow, which is considered typical.

(4) “Best management practice” or “BMP” means structural or non–structural measures, practices, techniques or devices employed to avoid or minimize sediment or pollutants carried in runoff to waters of the state.

(5) “Business day” means a day the office of the Department of Public Works is routinely and customarily open for business.

(6) “Cease and desist order” means a court–issued order to halt land disturbing construction activity that is being conducted without the required permit.

(7) “Combined sewer system” means a system for conveying both sanitary sewage and storm water runoff.

(8) “Connected imperviousness” means an impervious surface that is directly connected to a separate storm sewer or water of the state via an impervious flow path.

(9) “Design storm” means a hypothetical discrete rainstorm characterized by a specific duration, temporal distribution, rainfall intensity, return frequency, and total depth of rainfall.
(10) “Development” means residential, commercial, industrial or institutional land uses and associated roads.
(11) “Division of land” means the creation from one parcel of two or more parcels or building sites where such creation occurs at one time or through the successive partition within a 5 year period.
(12) “Effective infiltration area” means the area of the infiltration system that is used to infiltrate runoff and does not include the area used for site access, berms or pretreatment.
(13) “Erosion” means the process by which the land’s surface is worn away by the action of wind, water, ice or gravity.
(14) “Exceptional resource waters” means waters listed in s. NR 102.11, Wis. Adm. Code.
(15) “Extraterritorial” means the unincorporated area within 3 miles of Sister Bay corporate limits.
(16) ”Final stabilization” means that all land disturbing construction activities at the construction site have been completed and that a uniform, perennial, vegetative cover has been established, with a density of at least 70% of the cover, for the unpaved areas and areas not covered by permanent structures, or employment of equivalent permanent stabilization measures.
(17) “Financial guarantee” means a performance bond, maintenance bond, surety bond, irrevocable letter of credit, or similar guarantees submitted to the Department of Public Works by the responsible party to assure that requirements of the ordinance are carried out in compliance with the storm water management plan.
(18) “Governing body” means the Village council.
(19) “Illegal discharge” means any direct or indirect non-storm water discharge to the storm water system.
(20) “Illicit connections” means any drain or conveyance, whether on the surface or subsurface, which allows an illegal discharge to enter the storm water system including but not limited to any conveyances which allow any non-storm water discharges including sewage, process wastewater and wash water to enter the storm water system. Illicit connections also includes: 1. Connections to the storm water system from indoor drains and sinks regardless of whether the connection was previously allowed or permitted, and 2. Any drain or conveyance connected from a commercial, industrial or institutional land use to the storm water system which has not been documented in plans, maps or equivalent records and approved by the Village or Wisconsin Department of Natural Resources.
(21) “Impervious surface” means an area that releases as runoff all or a large portion of the precipitation that falls on it, except for frozen soil. Rooftops, sidewalks, driveways, parking lots and streets are examples of areas that typically are impervious.
(22) “In–fill area” means an undeveloped area of land located within existing development.
(23) “Infiltration” means the entry of precipitation or runoff into or through the soil.
(24) “Infiltration system” means a device or practice such as a basin, trench, rain garden or swale designed specifically to encourage infiltration, but does not include natural infiltration in pervious surfaces such as lawns, redirecting of rooftop downspouts onto lawns or minimal infiltration from practices, such as swales or road side channels designed for conveyance and pollutant removal only.

(25) “Karst feature” means an area or surficial geologic feature subject to bedrock dissolution so that it is likely to provide a conduit to groundwater, and may include caves, enlarged fractures, mine features, exposed bedrock surfaces, sinkholes, springs, seeps or swallets.
(26) “Land disturbing construction activity” means any man–made alteration of the land surface resulting in a change in the topography or existing vegetative or non–vegetative soil cover, that may result in runoff and lead to an increase in soil erosion and movement of sediment into

waters of the state. Land disturbing construction activity includes clearing and grubbing, demolition, excavating, pit trench dewatering, filling and grading activities.

(27) "Maintenance agreement" means a legal document that provides for long–term maintenance of storm water management practices.

(28) "MEP" or "maximum extent practicable" means a level of implementing best management practices in order to achieve a performance standard specified in this ordinance which takes into account the best available technology, cost effectiveness and other competing issues such as human safety and welfare, endangered and threatened resources, historic properties and geographic features. MEP allows flexibility in the way to meet the performance standards and may vary based on the performance standard and site conditions.

(29) "New development" means development resulting from the conversion of previously undeveloped land or agricultural land uses.

(30) "Non-storm water discharge" means any discharge to the storm drain system this is not composed entirely of storm water.

(31) "Off–site" means located outside the property boundary described in the permit application.

(32) "On–site" means located within the property boundary described in the permit application.

(33) "Ordinary high–water mark" has the meaning given in s. NR 115.03(6), Wis. Adm. Code.

(34) "Outstanding resource waters" means waters listed in s. NR 102.10, Wis. Adm. Code.

(35) "Percent fines" means the percentage of a given sample of soil, which passes through a #200 sieve.

(36) "Performance standard" means a narrative or measurable number specifying the minimum acceptable outcome for a facility or practice.

(37) "Permit" means a written authorization made by the Department of Public Works to the applicant to conduct land disturbing construction activity or to discharge post–construction runoff to waters of the state.

(38) "Permit administration fee" means a sum of money paid to the Department of Public Works by the permit applicant for the purpose of recouping the expenses incurred by the authority in administering the permit.

(39) "Pervious surface" means an area that releases as runoff a relatively small portion of the precipitation that falls on it. Lawns, gardens, parks, forests or other similar vegetated areas are examples of surfaces that typically are pervious.

(40) "Pollutant" has the meaning given in s. 283.01(13), Wis. Stats.

(41) "Pollution" has the meaning given in s. 281.01(10), Wis. Stats.

(42) "Post–construction site" means a construction site following the completion of land disturbing construction activity and final site stabilization.

(43) "Pre–development condition" means the extent and distribution of land cover types present before the initiation of land disturbing construction activity, assuming that all land uses prior to development activity are managed in an environmentally sound manner.

(44) "Preventive action limit" has the meaning given in s. NR 140.05(17), Wis. Adm. Code.

(45) "Redevelopment " means areas where development is replacing older development.

(46) "Responsible party" means any entity holding fee title to the property or other person contracted or obligated by other agreement to implement and maintain post–construction storm water BMPs.

(47) "Runoff" means storm water or precipitation including rain, snow or ice melt or similar water that moves on the land surface via sheet or channelized flow.

(48) "Separate storm sewer" means a conveyance or system of conveyances including roads with drainage systems, streets, catch basins, curbs, gutters, ditches, constructed channels or storm drains, which meets all of the following criteria:

(a) Is designed or used for collecting water or conveying runoff.

(b) Is not part of a combined sewer system.
(c) Is not draining to a storm water treatment device or system.
(d) Discharges directly or indirectly to waters of the state.

(49) "Site" means the entire area included in the legal description of the land on which the land disturbing construction activity occurred.

(50) "Stop work order" means an order issued by the Department of Public Works which requires that all construction activity on the site be stopped.

(51) "Storm water management plan" means a comprehensive plan designed to reduce the discharge of pollutants in storm water and to control storm water discharge rates and volumes after the site has under gone final stabilization following completion of the construction activity.

(52) "Storm water management system plan" is a comprehensive plan designed to reduce the discharge of runoff and pollutants from hydrologic units on a regional or municipal scale.

(53) "Storm water system" means publicly owned facilities by which storm water is collected, conveyed or treated, including but not limited to any roads with drainage systems, municipal streets, gutters, curbs, inlets, piped storm drains, retention and detention ponds, infiltration facilities, natural and human-made or altered water channels, and other drainage structures.

(54) "Technical standard" means a document that specifies design, predicted performance and operation and maintenance specifications for a material, device or method.

(55) "Top of the channel" means an edge, or point on the landscape, landward from the ordinary high– water mark of a surface water of the state, where the slope of the land begins to be less than 12% continually for at least 50 feet. If the slope of the land is 12% or less continually for the initial 50 feet, landward from the ordinary high–water mark, the top of the channel is the ordinary high–water mark.

(56) "TR–55" means the United States Department of Agriculture, Natural Resources Conservation Service (previously Soil Conservation Service), Urban Hydrology for Small Watersheds, Second Edition, Technical Release 55, June 1986.

(57) "Type II distribution" means a rainfall type curve as established in the "United States Department of Agriculture, Soil Conservation Service, Technical Paper 149, published 1973". The Type II curve is applicable to all of Wisconsin and represents the most intense storm pattern.

(58) "Waters of the state" has the meaning given in s. 281.01 (18), Wis. Stats.

31.06 STORM WATER TECHNICAL REVIEW COMMITTEE.

(1) Sister Bay Storm Water Technical Review Committee shall consist of:
(a) Voting Members
1. The Village Engineer/Public Works Director, acting as committee chair.
2. An appointed Alderperson
3. The Village Inspector
(b) Advisory Committee:
1. The Door County Liaison as appointed by the County Board Chairperson.
2. A local representative from the Department of Natural Resources with expertise in storm water management.
3. One member as appointed by the Board of Public Works, who has at least one of the following qualifications:
a. A professional engineer or registered hydrologist with a background in storm water management.

4. Three members at large who are citizens of Village to be appointed by the Mayor for a three-year term to represent the diverse interest groups of the Village.
5. Street Manager

(2) The purposes of Sister Bay Storm Water Technical Review Committee are the following:

(a) Provide objective and scientific technical review of storm water management or construction site erosion control issues or permits to be issued by the Village and, when requested by the Board of Public Works, provide recommendations to grant or deny storm water management or construction site erosion control permits.

(b) Provide public involvement and education opportunities for Citizens of the Village.

(3) Sister Bay Storm Water Technical Review Committee may retain a consultant to assist in the review of storm water management, storm water permit or construction site erosion control issues. Any cost incurred as part of the permit application review for storm water management or construction site erosion control permits shall be reimbursed by the applicant.

31.07 TECHNICAL STANDARDS.

The following methods shall be used in designing the water quality, peak flow shaving and infiltration components of storm water practices needed to meet the water quality standards of this ordinance:

(1) Technical standards identified, developed or disseminated by the Wisconsin Department of Natural Resources under subchapter V of chapter NR 151, Wis. Adm. Code.

(2) Where technical standards have not been identified or developed by the Wisconsin Department of Natural Resources, other technical standards may be used provided that the methods have been approved by the Department of Public Works.

(3) In this ordinance, the following year and location has been selected as the average annual rainfall: Minneapolis, 1959 (Mar. 13–Nov. 4).

31.08 PERFORMANCE STANDARDS.

(1) RESPONSIBLE PARTY. The responsible party shall implement a post–construction storm water management plan that incorporates the requirements of this section.

(2) PLAN. A written storm water management plan in accordance with Chapter 31.10 shall be developed and implemented for each post–construction site.

(3) REQUIREMENTS. The plan required under Chapter 31.08 (2) shall include the following:

(a) TOTAL SUSPENDED SOLIDS. BMPs shall be designed, installed and maintained to control total suspended solids carried in runoff from the post– construction site as follows:

1. For new development, by design, reduce to the maximum extent practicable, the total suspended solids load by 80%, based on the average annual rainfall, as compared to no runoff management controls. No person shall be required to exceed an 80% total suspended solids reduction to meet the requirements of this subdivision.
2. For redevelopment, by design, reduce to the maximum extent practicable, the total suspended solids load by 40%, based on the average annual rainfall, as compared to no runoff management controls. No person shall be required to exceed a 40% total suspended solids reduction to meet the requirements of this subdivision.
3. For in–fill development under 5 acres that occurs within 10 years after October 1, 2002, by design, reduce to the maximum extent practicable, the total suspended solids load by 40%, based on an average annual rainfall, as compared to no runoff management controls. No person shall be required to exceed a 40% total suspended solids reduction to meet the requirements of this subdivision.

4. For in–fill development that occurs 10 or more years after October 1, 2002, by design, reduce to the maximum extent practicable, the total suspended solids load by 80%, based on an average annual rainfall, as compared to no runoff management controls. No person shall be required to exceed an 80% total suspended solids reduction to meet the requirements of this subdivision.
5. Notwithstanding Chapter 31.08(3)(a) 1. to 4., if the design cannot achieve the applicable total suspended solids reduction specified, the storm water management plan shall include a written and site–specific explanation why that level of reduction is not attained and the total suspended solids load shall be reduced to the maximum extent practicable.

(b) PEAK DISCHARGE.

1. By design, BMPs shall be employed to maintain or reduce the peak runoff discharge rates, to the maximum extent practicable, as compared to pre– development conditional the 2–year, 10-year, 25-year, and 100-year recurrence interval, 24–hour duration design storms applicable to the post–construction site. Pre–development conditions shall assume "good hydrologic conditions" for appropriate land covers as identified in TR–55 or an equivalent methodology. The meaning of "hydrologic soil group" and "runoff curve number" are as determined in TR–55. However, when pre–development land cover is cropland, rather than using TR–55 values for cropland, the runoff curve numbers in Table 1 shall be used.

Table 1
Maximum Pre-Development Runoff Curve Numbers for Cropland Areas

Hydrologic Soil Group	A	B	C	D
Runoff Curve Number	56	70	79	83

2. This subsection of the ordinance does not apply to any of the following:
   a. A redevelopment post–construction site with no increase in impervious surface.

(c) INFILTRATION. BMPs shall be designed, installed, and maintained to infiltrate runoff to the maximum extent practicable in accordance with the following, except as provided in Chapter 31.08(3)(c) 7. through 10.

1. Areas of the Village protected by the wellhead protection zoning:
   a. All infiltration practices shall meet the requirements of Chapter XX of the Village Ordinances in areas of the Village designated as Zone 1 or Zone 2 wellhead protection areas.
2. Areas of the Village are not served by storm sewers:
   a. In areas of the Village not served by storm sewers, as determined by the Department of Public Works, all increases in runoff volume exceeding predevelopment conditions must be captured and infiltrated for rain events less than or equal to the 100-year, 24-hour duration design storm.
3. Areas of the Village served by storm sewers:
   a. For residential developments one of the following shall be met:
      (i) Infiltrate sufficient runoff volume so that the post– development infiltration volume shall be at least 90% of the pre–development infiltration volume, based on an average annual rainfall. However, when designing appropriate

infiltration systems to meet this requirement, no more than 1% of the project site is required as an effective infiltration area.

(ii) Infiltrate 25% of the post–development runoff from the 2 year –24-hour design storm with a NRCS type II distribution. Separate curve numbers for pervious and impervious surfaces shall be used to calculate runoff volumes and not composite curve numbers as defined in TR–55. However, when designing appropriate infiltration systems to meet this requirement, no more than 1% of the project site is required as an effective infiltration area.

b. For non–residential development, including commercial, industrial and institutional development, one of the following shall be met:

(i) Infiltrate sufficient runoff volume so that the post– development infiltration volume shall be at least 60% of the pre–development infiltration volume, based on an average annual rainfall. However, when designing appropriate infiltration systems to meet this requirement, no more than 2% of the project site is required as an effective infiltration area.

(ii) Infiltrate 10% of the runoff from the 2 year – 24 hour design storm with a type II distribution. Separate curve numbers for pervious and impervious surfaces shall be used to calculate runoff volumes, and not composite curve numbers as defined in TR–55. However, when designing appropriate infiltration systems to meet this requirement, no more than 2% of the project site is required as an effective infiltration area.

c. Pre–development condition shall be the same as in Chapter 31.08(3) (c)3.b.

4. Before infiltrating runoff, pretreatment shall be required. The pretreatment shall be designed to protect the infiltration system from clogging prior to scheduled maintenance and to protect groundwater quality in accordance with Chapter 31.08(3)(c) 10. Pretreatment options may include, but are not limited to, oil/grease separation, sedimentation, biofiltration, filtration, swales or filter strips and must be designed and constructed to remove contaminants associated with the proposed land use to the maximum extent practicable.

5. The sequence of construction at a site where infiltration practices will be constructed shall allow surface soils, disturbed during construction, to be stabilized with vegetation prior to final grading or construction of infiltration areas.

6. Owners and operators of a construction activity which results in eroded soil clogging or other damage to an infiltration area shall restore the infiltration area to the condition existing prior to the damage.

7. Exclusions. The runoff from the following areas are prohibited from meeting the requirements of this paragraph:

a. Areas associated with tier 1 industrial facilities identified in s. NR 216.21(2)(a), Wis. Adm. Code, including storage, loading, rooftop and parking.

b. Storage and loading areas of tier 2 industrial facilities identified in s. NR 216.21(2)(b), Wis. Adm. Code. Runoff from tier 2 parking and rooftop areas may be infiltrated.

c. Fueling and vehicle maintenance areas.

d. Infiltration areas with less than 3 feet separation distance from the bottom of the infiltration system to the elevation of seasonal high groundwater or the top of bedrock unless pretreatment is provided such that the requirements of Chapter 31.08(3)(c) 10 are met. Infiltration rate computations must include the effects of groundwater mounding which may occur under these conditions. Computations related to pretreatment pollutant removal rates and groundwater mounding must be provided to the Department of Public Works for review and approval. This subd. 7. d. does not prohibit infiltration of untreated roof runoff.

e. Areas with runoff from industrial, commercial and institutional parking lots and roads and residential arterial roads with less than 5 feet separation distance from the bottom of the infiltration system to the elevation of seasonal high groundwater or the top of bedrock unless pretreatment is provided such that the requirements of Chapter 31.08(3)(c) 10 are met. Infiltration rate computations must include the effects of groundwater mounding which may occur under these conditions. Computations related to pretreatment pollutant removal rates and groundwater mounding must be provided to the Department of Public Works for review and approval

f. Areas within 400 feet of a community water system well as specified in s. NR 811.16(4), Wis. Adm. Code, or within 100 feet of a private well as specified in s. NR 812.08(4), Wis. Adm. Code, for runoff infiltrated from commercial, industrial and institutional land uses or regional devices for residential development.

g. Areas where contaminants of concern, as defined in s. NR 720.03(2), Wis. Adm. Code are present in the soil through which infiltration will occur.

8. Exemptions. The following are not required to meet the requirements of this paragraph:
   a. Areas where the infiltration rate of the infiltrating soil is less than 0.6 inches/hour measured at the site.
   b. Parking areas and access roads less than 5,000 square feet for commercial and industrial development.
   c. Redevelopment post–construction sites with no increase in impervious surface.
   d. Infiltration areas during periods when the soil on the site is frozen.
9. Where alternate uses of runoff are employed, such as for toilet flushing, laundry or irrigation, such alternate use shall be given equal credit toward the infiltration volume required by this paragraph.
10. a. Infiltration systems designed in accordance with this paragraph shall, to the extent technically and economically feasible, minimize the level of pollutants infiltrating to groundwater and shall maintain compliance with the preventive action limit at a point of standards application in accordance with ch. NR 140, Wis. Adm. Code. However, if site-specific information indicates that compliance with a preventive action limit is not achievable, the infiltration BMP may not be installed or shall be modified to prevent infiltration to the maximum extent practicable.
    b. Notwithstanding Chapter 3108(3)(c) 10.a., the discharge from BMPs shall remain below the enforcement standard at the point of standards application.

(d) PROTECTIVE AREAS.

1. "Protective area" means an area of land that commences at the top of the channel of lakes, streams and rivers, or at the delineated boundary of wetlands, and that is the greatest of the following widths, as measured horizontally from the top of the channel or delineated wetland boundary to the closest impervious surface. However, in this paragraph, "protective area" does not include any area of land adjacent to any stream enclosed within a pipe or culvert, such that runoff cannot enter the enclosure at this location.

a. For outstanding resource waters and exceptional resource waters, and for wetlands in areas of special natural resource interest as specified in s. NR 103.04, 75 feet.
b. For perennial and intermittent streams identified on a United States geological survey 7.5–minute series topographic map, or a county soil survey map, whichever is more current, 50 feet.
c. For lakes, 50 feet.
d. For highly susceptible wetlands, 50 feet. Highly susceptible wetlands include the following types: fens, sedge meadows, bogs, low prairies, conifer swamps, shrub swamps, other forested wetlands, fresh wet meadows, shallow marshes, deep marshes and seasonally flooded basins. Wetland boundary delineations shall be made in accordance with s. NR 103.08(1m). This paragraph does not apply to wetlands that have been completely filled in accordance with all applicable state and federal regulations. The protective area for wetlands that have been partially filled in accordance with all applicable state and federal regulations shall be measured from the wetland boundary delineation after fill has been placed.
e. For less susceptible wetlands, 10 percent of the average wetland width, but no less than 10 feet nor more than 30 feet. Less susceptible wetlands include degraded wetlands dominated by invasive species such as reed canary grass.
f. In Chapter 31.08(3)(d) 1.a., d. and e., determinations of the extent of the protective area adjacent to wetlands shall be made on the basis of the sensitivity and runoff susceptibility of the wetland in accordance with the standards and criteria in s. NR 103.03.
g. For concentrated flow channels with drainage areas greater than 130 acres, 10 feet.

2. This paragraph applies to post–construction sites located within a protective area, except those areas exempted pursuant to Chapter 31.08(3)(d) 4.
3. The following requirements shall be met:
   a. Impervious surfaces shall be kept out of the protective area to the maximum extent practicable. The storm water management plan shall contain a written site–specific explanation for any parts of the protective area that are disturbed during construction.
   b. Where land disturbing construction activity occurs within a protective area, and where no impervious surface is present, adequate sod or self–sustaining vegetative cover of 70% or greater shall be established and maintained. The adequate sod or self– sustaining vegetative cover shall be sufficient to provide for bank stability, maintenance of fish habitat and filtering of pollutants from upslope overland flow areas under sheet flow conditions. Non–vegetative materials, such as rock riprap, may be employed on the bank as necessary to prevent erosion, such as on steep slopes or where high velocity flows occur.
   c. Best management practices such as filter strips, swales, or wet detention basins, that are designed to control pollutants from non–point sources may be located in the protective area, subject to the proposed BMPs being permitted by the Wisconsin Department of Natural Resources and approved by the Department of Public Works.
4. This paragraph does not apply to:
   a. Redevelopment post–construction sites with no increase in impervious surface.

b. Structures that cross or access surface waters such as boat landings, bridges and culverts.
c. Structures constructed in accordance with s. 59.692(1v), Wis. Stats.
d. Post–construction sites from which runoff does not enter the surface water, except to the extent that vegetative ground cover is necessary to maintain bank stability.

(e) FUELING AND VEHICLE MAINTENANCE AREAS. Fueling and vehicle maintenance areas shall, to the maximum extent practicable, have BMPs designed, installed and maintained to reduce petroleum based compounds within runoff, such that the runoff that enters waters of the state contains no visible petroleum sheen.

(f) SWALE TREATMENT FOR TRANSPORTATION FACILITIES.

1. Applicability. Except as provided in Chapter 31.08(3)(f) 2., transportation facilities that use swales for runoff conveyance and pollutant removal meet all of the requirements of this section, if the swales are designed to the maximum extent practicable to do all of the following:
   a. Be vegetated. However, where appropriate, non– vegetative measures may be employed to prevent erosion or provide for runoff treatment, such as rock riprap stabilization or check dams.
   b. Carry runoff through a swale for 200 feet or more in length that is designed with a flow velocity no greater than 1.5 feet per second for the peak flow generated using either a 2–year, 24–hour design storm or a 2–year storm with a duration equal to the time of concentration as appropriate. If a swale of 200 feet in length cannot be designed with a flow velocity of 1.5 feet per second or less, then the flow velocity shall be reduced to the maximum extent practicable.
2. Exemptions. The Department of Public Works may, consistent with water quality standards, require other provisions of this section be met on a transportation facility with an average daily travel of vehicles greater than 2500 and where the initial surface water of the state that the runoff directly enters is any of the following:
   a. An outstanding resource water.
   b. An exceptional resource water.
   c. Waters listed in s. 303(d) of the federal clean water act that are identified as impaired in whole or in part, due to nonpoint source impacts.
   d. Waters where targeted performance standards are developed under s. NR 151.004, Wis. Adm. Code, to meet water quality standards.

(4) GENERAL CONSIDERATIONS FOR ON–SITE AND OFF–SITE STORM WATER MANAGEMENT MEASURES. The following considerations shall be observed in managing runoff:

(a) Natural topography and land cover features such as natural swales, natural depressions, native soil infiltrating capacity, and natural groundwater recharge areas shall be preserved and used, to the extent possible, to meet the requirements of this section.

(b) Emergency overland flow for all storm water facilities shall be provided to prevent exceeding the safe capacity of downstream drainage facilities and prevent endangerment of downstream property or public safety.

(5) LOCATION AND REGIONAL TREATMENT OPTION.

(a) The BMPs may be located on–site or off–site as part of a regional storm water device, practice or system.

(b) Post–construction runoff within a non–navigable surface water that flows into a BMP, such as a wet detention pond, is not required to meet the performance standards of this ordinance. Post–construction BMPs may be located in non–navigable surface waters.

(c) Except as allowed under Chapter 31.08(5) (d), post–construction runoff from new development shall meet the post–construction performance standards prior to entering a navigable surface water.

(d) Post–construction runoff from any development within a navigable surface water that flows into a BMP is not required to meet the performance standards of this ordinance if:

1. The BMP was constructed prior to the effective date of this ordinance and the BMP either received a permit issued under ch. 30, Stats., or the BMP did not require a ch. 30, Wis. Stats., permit; and
2. The BMP is designed to provide runoff treatment from future upland development.

(e) Runoff from existing development, redevelopment and in–fill areas shall meet the post–construction performance standards in accordance with this paragraph.

1. To the maximum extent practicable, BMPs shall be located to treat runoff prior to discharge to navigable surface waters.
2. Post–construction BMPs for such runoff may be located in a navigable surface water if allowable under all other applicable federal, state and local regulations such as ch. NR 103, Wis. Adm. Code and ch. 30, Wis. Stats.

(f) The discharge of runoff from a BMP, such as a wet detention pond, or after a series of such BMPs is subject to this Chapter.

(g) The Department of Public Works may approve off– site management measures provided that all of the following conditions are met:

1. The Department of Public Works determines that the post–construction runoff is covered by a storm water management system plan that is approved by Sister Bay and that contains management requirements consistent with the purpose and intent of this ordinance.
2. The off–site facility meets all of the following conditions:
   a. The facility is in place.
   b. The facility is designed and adequately sized to provide a level of storm water control equal to or greater than that which would be afforded by on–site practices meeting the performance standards of this ordinance.
   c. The facility has a legally obligated entity responsible for its long–term operation and maintenance.

(h) Where a regional treatment option exists such that the Department of Public Works exempts the applicant from all or part of the minimum on–site storm water management requirements, the applicant shall be required to pay a fee in an amount determined in negotiation with the Department of Public Works. In determining the fee for post–construction runoff, the Department of Public Works shall consider an equitable distribution of the cost for land, engineering design, construction, and maintenance of the regional treatment option.

(6) ALTERNATE REQUIREMENTS. The Department of Public Works may establish storm water management requirements more stringent than those set forth in this section if the Department of Public Works determines that an added level of protection is needed to protect sensitive resources.

31.09 PERMITTING REQUIREMENTS, PROCEDURES AND FEES.

(1) PERMIT REQUIRED. No responsible party may undertake a land disturbing construction activity without receiving a post–construction runoff permit from the Department of Public Works prior to commencing the proposed activity.

(2) PERMIT APPLICATION AND FEES. Unless specifically excluded by this ordinance, any responsible party desiring a permit shall submit to the Department of Public Works a permit application made on a form provided by the Department of Public Works for that purpose.

(a) Unless otherwise excepted by this ordinance, a permit application must be accompanied by a storm water management plan, a maintenance and inspection agreement and a non–refundable permit administration fee.

(b) The storm water management plan shall be prepared to meet the requirements of Chapter 31.08 and Chapter 31.10, the maintenance agreement shall be prepared to meet the requirements of Chapter 31.11, the financial guarantee shall meet the requirements of Chapter 31.13, and fees shall be those established by the Village Board as set forth in Chapter 31.14.

(3) REVIEW AND APPROVAL OF PERMIT APPLICATION. The Department of Public Works shall review any permit application that is submitted with a storm water management plan, maintenance agreement, and the required fee. The following approval procedure shall be used:

(a) Within 30 business days of the receipt of a complete permit application, including all items as required by Chapter 31.09 (2), the Department of Public Works shall inform the applicant whether the application, plan and maintenance agreement are approved or disapproved based on the requirements of this ordinance.

(b) If the storm water permit application, plan and maintenance agreement are approved, or if an agreed upon payment of fees in lieu of storm water management practices is made, the Department of Public Works shall issue the permit.

(c) If the storm water permit application, plan or maintenance agreement is disapproved, the Department of Public Works shall detail in writing the reasons for disapproval.

(d) The Department of Public Works may request additional information from the applicant. If additional information is submitted, the Department of Public Works shall have 10 business days from the date the additional information is received to inform the applicant that the plan and maintenance agreement are either approved or disapproved.

(e) Failure by the Department of Public Works to inform the permit applicant of a decision within 30 business days of a required submittal shall be deemed to mean approval of the submittal and the applicant may proceed as if a permit had been issued.

(4) PERMIT REQUIREMENTS. All permits issued under this ordinance shall be subject to the following conditions, and holders of permits issued under this ordinance shall be deemed to have accepted these conditions. The Department of Public Works may suspend or revoke a permit for violation of a permit condition, following written notification of the responsible party. An action by the Department of Public Works to suspend or revoke this permit may be appealed in accordance with Chapter 31.16.

(a) Compliance with this permit does not relieve the responsible party of the responsibility to comply with other applicable federal, state, and local laws and regulations.

(b) The responsible party shall design and install all structural and non–structural storm water management measures in accordance with the approved storm water management plan and this permit.

(c) The responsible party shall notify the Department of Public Works at least 2 business days before commencing any work in conjunction with the storm water management plan, and within 14 business days upon completion of the storm water management practices. If required as a special condition under Chapter 31.09 (5), the responsible party shall make additional notification according to a schedule set forth by the Department of Public Works so that practice installations can be inspected during construction.

(d) Practice installations required as part of this ordinance shall be certified "as built" by a licensed professional engineer. Completed storm water management practices must pass a final inspection by the Department of Public Works or its designee to determine if they are in accordance with the approved storm water management plan and ordinance. The Department of Public Works or its designee shall notify the responsible party in writing of any changes required in such practices to bring them into compliance with the conditions of this permit.

(e) The responsible party shall notify the Department of Public Works of any significant modifications it intends to make to an approved storm water management plan. The Department of Public Works may require that the proposed modifications be submitted to it for approval prior to incorporation into the storm water management plan and execution by the responsible party.

(f) The responsible party shall maintain all storm water management practices in accordance with the storm water management plan until the practices either become the responsibility of Sister Bay, or are transferred to subsequent private owners as specified in the approved maintenance agreement.

(g) The responsible party authorizes the Department of Public Works to perform any work or operations necessary to bring storm water management measures into conformance with the approved storm water management plan, and consents to a special assessment or charge against the property as authorized under subch. VII of ch. 66, Wis. Stats., or to charging such costs against the financial guarantee posted under Chapter 31.13.

(h) If so directed by the Department of Public Works, the responsible party shall repair at the responsible party's own expense all damage to adjoining municipal facilities and drainage ways caused by runoff, where such damage is caused by activities that are not in compliance with the approved storm water management plan.

(i) The responsible party shall permit property access to the Department of Public Works or its designee for the purpose of inspecting the property for compliance with the approved storm water management plan and this permit.

(j) Where site development or redevelopment involves changes in direction, increases in peak rate and/or total volume of runoff from a site, the Department of Public Works may require the responsible party to make appropriate legal arrangements with affected property owners concerning the prevention of endangerment to property or public safety.

(k) The responsible party is subject to the enforcement actions and penalties detailed in Chapter 31.15, if the responsible party fails to comply with the terms of this permit.

(5) PERMIT CONDITIONS. Permits issued under this subsection may include conditions established by Department of Public Works in addition to the requirements needed to meet the performance standards in Chapter 31.08 or a financial guarantee as provided for in Chapter 31.13.

(6) PERMIT DURATION. Permits issued under this section shall be valid from the date of issuance through the date the Department of Public Works notifies the responsible party that all storm water management practices have passed the final inspection required under Chapter 31.09 (4)(d).

31.10 STORM WATER MANAGEMENT PLAN.

(1) PLAN REQUIREMENTS. The storm water management plan required under Chapter 31.09 (2) shall contain at a minimum the following information:

(a) Name, address, and telephone number for the following or their designees: landowner; developer; project engineer for practice design and certification; person(s) responsible for installation of storm water management practices; and person(s) responsible for maintenance of storm water management practices prior to the transfer, if any, of maintenance responsibility to another party.

(b) A proper legal description of the property proposed to be developed, referenced to the U.S. Public Land Survey system or to block and lot numbers within a recorded land subdivision plat.

(c) Pre–development site conditions, including:

1. One or more site maps at a scale of not to exceed 1 inch equals 50 feet. The site maps shall show the following: site location and legal property description; predominant soil types and hydrologic soil groups; existing cover type and condition; topographic contours of the site at a scale not to exceed 2 feet; topography and drainage network including enough of the contiguous properties to show runoff patterns onto, through, and from the site; watercourses that may affect or be affected by runoff from the site; flow path and direction for all storm water conveyance sections; watershed boundaries used in hydrology determinations to show compliance with performance standards; lakes, streams, wetlands, channels, ditches, and other watercourses on and immediately adjacent to the site; limits of the 100 year floodplain; location of wells and wellhead protection areas covering the project area and delineated pursuant to s. NR 811.16, Wis. Adm. Code.
2. Hydrology and pollutant loading computations as needed to show compliance with performance standards. All major assumptions used in developing input parameters shall be clearly stated. The geographic areas used in making the calculations shall be clearly cross–referenced to the required map(s).
3. Infiltration system design information as described in the Wisconsin Department of Natural Resources Infiltration System Site Evaluation Standard.

(d) Post–development site conditions, including:

1. Explanation of the provisions to preserve and use natural topography and land cover features to minimize changes in peak flow runoff rates and volumes to surface waters and wetlands.
2. Explanation of any restrictions on storm water management measures in the development area imposed by wellhead protection plans and ordinances.
3. One or more site maps at a scale of not to exceed 1 inch equals 50 feet showing the following: post–construction pervious areas including vegetative cover type and condition; impervious surfaces including all buildings, structures, and pavement; post–construction topographic contours of the site at a scale not to exceed 2 feet; post–construction drainage network including enough of the contiguous properties to show runoff patterns onto, through, and from the site; locations and dimensions of drainage easements; locations of maintenance easements specified in the maintenance agreement; flow path and direction for all storm water conveyance

   sections; location and type of all storm water management conveyance and treatment practices, including the on–site and off–site tributary drainage area; location and type of conveyance system that will carry runoff from the drainage and treatment practices to the nearest adequate outlet such as a curbed street, storm drain, or natural drainage way; watershed boundaries used in hydrology and pollutant loading calculations and any changes to lakes, streams, wetlands, channels, ditches, and other watercourses on and immediately adjacent to the site.

4. Hydrology and pollutant loading computations as needed to show compliance with performance standards. The computations shall be made for each discharge point in the development, and the geographic areas used in making the calculations shall be clearly cross–referenced to the required map(s).
5. Results of investigations of soils and groundwater required for the placement and design of storm water management measures. Detailed drawings including cross–sections and profiles of all permanent storm water conveyance and treatment practices.

(e) A description and installation schedule for the storm water management practices needed to meet the performance standards in Chapter 31.08.

(f) A maintenance and inspection plan developed for the life of each storm water management practice including the required inspection and maintenance activities, and anticipated maintenance activity schedule.

(g) Cost estimates for the construction, operation, and maintenance of each storm water management practice.

(h) Other information requested in writing by the Department of Public Works to determine compliance of the proposed storm water management measures with the provisions of this ordinance.

(i) All site investigations, plans, designs, computations, and drawings shall be certified by a licensed professional engineer to be prepared in accordance with accepted engineering practice and requirements of this ordinance.

(2) ALTERNATE REQUIREMENTS. The Department of Public Works may prescribe alternative submittal requirements for applicants seeking an exemption to on–site storm water management performance standards under Chapter 31.08 (5).

31.11 MAINTENANCE AND INSPECTION AGREEMENT.

(1) MAINTENANCE AND INSPECTION AGREEMENT REQUIRED. The maintenance and inspection agreement required under Chapter 31.09 (2) for storm water management practices shall be an agreement between the Department of Public Works and the responsible party to provide for maintenance of storm water practices beyond the duration period of this permit. The maintenance and inspection agreement shall be filed with the County Register of Deeds as a property deed restriction so that it is binding upon all subsequent owners of the land served by the storm water management practices.

(2) AGREEMENT PROVISIONS. The maintenance and inspection agreement shall contain the following information and provisions and be consistent with the maintenance and inspection plan required by Chapter 31.10(1)(f):

(a) Identification of the storm water facilities and designation of the drainage area served by the facilities.

(b) A schedule for regular inspection and maintenance of each aspect of the storm water management system consistent with the storm water management plan required under Chapter 31.09 (2).

(c) Identification of the responsible party(s), organization or Village, county, town or village responsible for long term inspection and maintenance of the storm water management practices identified in the storm water management plan required under Chapter 31.09 (2).

(d) Requirement that the responsible party(s), organization, or Village, county, town or village shall maintain storm water management practices in accordance with the schedule included in Chapter 31.11 (2) (b).

(e) Authorization for the Department of Public Works to access the property to conduct inspections of storm water management practices as necessary to ascertain that the practices are being maintained and operated in accordance with the agreement.
(f) A requirement on the Department of Public Works to maintain public records of the results of the site inspections, to inform the responsible party responsible for maintenance of the inspection results, and to specifically indicate any corrective actions required to bring the storm water management practice into proper working condition.
(g) Agreement that the party designated under Chapter 31.11 (2) (c), as responsible for long term maintenance of the storm water management practices, shall be notified by the Department of Public Works of maintenance problems which require correction. The specified corrective actions shall be undertaken within a reasonable time frame as set by the Department of Public Works.
(h) Authorization of the Department of Public Works to perform the corrected actions identified in the inspection report if the responsible party designated under Chapter 31.11 (2) (c) does not make the required corrections in the specified time period. The Department of Public Works shall enter the amount due on the tax rolls and collect the money as a special charge against the property pursuant to subch. VII of ch. 66, Wis. Stats.

31.12 ILLEGAL DISCHARGES AND ILLICIT CONNECTIONS.
(1) DISCHARGES PROHIBITED. No person may discharge, spill or dump pollutants, substances or materials which are not entirely composed of storm water into waters of the state, storm sewers or drainage facilities, or onto driveways, sidewalks, parking lots or other areas that drain into the storm water system.
(2) EXCEPTIONS. The following are exempt from the provisions of this section unless found to have an adverse impact on storm water:
(a) Discharges authorized by a permit issued by the Wisconsin Department of Natural Resources.
(b) Discharges resulting from fire fighting activities, excluding training activities.
(c) Discharges from uncontaminated groundwater, potable water source, roof drains, foundation drains and sump pumps, lawn watering, individual residential car washing, water main and hydrant flushing, and individual residential swimming pools if the water has been dechlorinated.
(3) ILLICIT CONNECTIONS PROHIBITED. The construction, use, maintenance or continued existence of illicit connections to the storm water system is prohibited. The prohibition expressly includes, without limitation, illicit connections made in the past, regardless of whether the connection was permissible under law or practices applicable or prevailing at the time of connection.
(a) VIOLATION. A person is considered to be in violation of this ordinance if the person connects a line conveying sewage or other non storm water discharges not exempted by Chapter 31.12 (2) to the storm water system, or allows such a connection to continue.

(4) Notice of Violation.
Whenever the Department of Public Works finds that a person has violated a prohibition or failed to meet a requirement of this Section, the Department of Public Works may order compliance by written notice of violation to the responsible person. Such notice may require without limitation:

(a) The performance of monitoring, analyses, and reporting;
(b) The elimination of illicit connections or discharges;

(c) That violating discharges, practices, or operations shall cease and desist;
(d) The abatement or remediation of storm water pollution or contamination hazards and the restoration of any affected property; and
(e) Payment of a fine to cover administrative and remediation costs; and
(f) The implementation of source control or treatment BMPs.

If abatement of a violation and/or restoration of affected property is required, the notice shall set forth a deadline within which such remediation or restoration must be completed. Said notice shall further advise that, should the violator fail to remediate or restore within the established deadline, the work will be done by a designated governmental agency or a contractor and the expense thereof shall be charged to the violator.

(5) Appeal of Notice of Violation.
Any person receiving a Notice of Violation may appeal the determination of the authorized enforcement agency. The notice of appeal must be received within 10 business days from the date of the Notice of Violation. Hearing on the appeal before the appropriate authority or his/her designee shall take place within 15 business days from the date of receipt of the notice of appeal. The decision of the municipal authority or their designee shall be final.

(6) Enforcement Measures After Appeal.
If the violation has not been corrected pursuant to the requirements set forth in the Notice of Violation, or, in the event of an appeal, 15 business days within days of the decision of the Department of Public Works upholding the decision of the authorized enforcement agency, then representatives of the Department of Public Works shall enter upon the subject private property and are authorized to take any and all measures necessary to abate the violation and/or restore the property. It shall be unlawful for any person, owner, agent or person in possession of any premises to refuse to allow the Department of Public Works or designated contractor to enter upon the premises for the purposes set forth above.

(7) Cost of Abatement of the Violation.
Within 15 business days after abatement of the violation, the owner of the property will be notified of the cost of abatement, including administrative costs. The property owner may file a written protest objecting to the amount of the assessment within 15 business days. If the amount due is not paid within a timely manner as determined by the decision of the municipal authority or by the expiration of the time in which to file an appeal, the charges shall become a special assessment against the property and shall constitute a lien on the property for the amount of the assessment. Any person violating any of the provisions of this article shall become liable to the Village by reason of such violation.

(8) Violations Deemed A Public Nuisance.
In addition to the enforcement processes and penalties provided, any condition caused or permitted to exist in violation of any of the provisions of this Ordinance is a threat to public health, safety, and welfare, and is declared and deemed a nuisance, and may be summarily abated or restored at the violator's expense, and/or a civil action to abate, enjoin, or otherwise compel the cessation of such nuisance may be taken.

31.13 FINANCIAL GUARANTEE.

(1) ESTABLISHMENT OF THE GUARANTEE. The Department of Public Works may require the submittal of a financial guarantee, the form and type of which shall be acceptable to the Department of Public Works. The financial guarantee shall be in an amount determined by the Department of Public Works to be the estimated cost of construction and the estimated cost of

maintenance of the storm water management practices during the period which the designated party in the maintenance agreement has maintenance responsibility. The financial guarantee shall give the Department of Public Works the authorization to use the funds to complete the storm water management practices if the responsible party defaults or does not properly implement the approved storm water management plan, upon written notice to the responsible party by the administering authority that the requirements of this ordinance have not been met.

(2) CONDITIONS FOR RELEASE. Conditions for the release of the financial guarantee are as follows:

(a) The Department of Public Works shall release the portion of the financial guarantee established under this section, less any costs incurred by the Department of Public Works to complete installation of practices, upon submission of "as built plans" by a licensed professional engineer. The Department of Public Works may make provisions for a partial pro–rata release of the financial guarantee based on the completion of various development stages.

(b) The Department of Public Works shall release the portion of the financial guarantee established under this section to assure maintenance of storm water practices, less any costs incurred by the Department of Public Works, at such time that the responsibility for practice maintenance is passed on to another entity via an approved maintenance agreement.

31.14 FEE SCHEDULE.

The fees referred to in other sections of this ordinance shall be established by the Board of Public Works and may from time to time be modified by resolution. A schedule of the fees established by the Board of Public Works shall be available for review at the Department of Public Works.

31.15 ENFORCEMENT.

(1) Any land disturbing construction activity or post– construction runoff initiated after the effective date of this ordinance by any person, firm, association, or corporation subject to the ordinance provisions shall be deemed a violation unless conducted in accordance with the requirements of this ordinance.

(2) The Department of Public Works shall notify the responsible party by certified mail of any non-complying land disturbing construction activity or post– construction runoff. The notice shall describe the nature of the violation, remedial actions needed, a schedule for remedial action, and additional enforcement action which may be taken.

(3) Upon receipt of written notification from the Department of Public Works under Chapter 31.15 (2), the responsible party shall correct work that does not comply with the storm water management plan or other provisions of this permit. The responsible party shall make corrections as necessary to meet the specifications and schedule set forth by the Department of Public Works in the notice.

(4) If the violations to a permit issued pursuant to this ordinance are likely to result in damage to properties, public facilities, or waters of the state, the Department of Public Works may enter the land and take emergency actions necessary to prevent such damage. The costs incurred by the Department of Public Works plus interest and legal costs shall be billed to the responsible party.

(5) The Department of Public Works is authorized to post a stop work order on all land disturbing construction activity that is in violation of this ordinance, or to request the municipal attorney to obtain a cease and desist order in any court with jurisdiction.

(6) The Department of Public Works may revoke a permit issued under this ordinance for non–compliance with ordinance provisions.

(7) Any permit revocation, stop work order, or cease and desist order shall remain in effect unless retracted by the Department of Public Works or by a court with jurisdiction.
(8) The Department of Public Works is authorized to refer any violation of this ordinance, or of a stop work order or cease and desist order issued pursuant to this ordinance, to the municipal attorney for the commencement of further legal proceedings in any court with jurisdiction.
(9) Any person, firm, association, or corporation who does not comply with the provisions of this ordinance shall be subject to a forfeiture of not less than 250.00 dollars or more than 500.00 dollars per offense, together with the costs of prosecution. Each day that the violation exists shall constitute a separate offense.
(10) Compliance with the provisions of this ordinance may also be enforced by injunction in any court with jurisdiction. It shall not be necessary to prosecute for forfeiture or a cease and desist order before resorting to injunctional proceedings.
(11) When the Department of Public Works determines that the holder of a permit issued pursuant to this ordinance has failed to follow practices set forth in the storm water management plan, or has failed to comply with schedules set forth in said storm water management plan, the Department of Public Works or a party designated by the Department of Public Works may enter upon the land and perform the work or other operations necessary to bring the condition of said lands into conformance with requirements of the approved plan. The Department of Public Works shall keep a detailed accounting of the costs and expenses of performing this work. These costs and expenses shall be deducted from any financial security posted pursuant to Chapter 31.13 of this ordinance. Where such a security has not been established, or where such a security is insufficient to cover these costs, the costs and expenses shall be entered on the tax roll as a special charge against the property and collected with any other taxes levied thereon for the year in which the work is completed.

31.16 APPEALS.

(1) BOARD OF PUBLIC WORKS. The board of public works, created pursuant to Chapter 1.17 of Sister Bay ordinances pursuant to s. 62.23(7)(e), Wis. Stats, shall hear and decide appeals where it is alleged that there is error in any order, decision or determination made by the Department of Public Works in administering this ordinance. The board shall also use the rules, procedures, duties, and powers authorized by statute in hearing and deciding appeals. Upon appeal, the board may authorize variances from the provisions of this ordinance that are not contrary to the public interest, and where owing to special conditions a literal enforcement of the ordinance will result in unnecessary hardship.
(2) WHO MAY APPEAL. Appeals to the board of public works may be taken by any aggrieved person or by an officer, department, board, or bureau of Sister Bay affected by any decision of the Department of Public Works.

31.17 SEVERABILITY.

If any section, clause, provision or portion of this ordinance is judged unconstitutional or invalid by a court of competent jurisdiction, the remainder of the ordinance shall remain in force and not be affected by such judgment.

31.18 EFFECTIVE DATE.

This ordinance shall be in force and effect from and after its adoption and publication. The above and foregoing ordinance was duly adopted by the Village Board of Sister Bay on the ____th day of __________, 200_.

First Reading ______________________________________ ____________________________
Trustee

Second Reading ____________________________________

Adopted______________________________________

Approved _____________________________________
President

Attest ____
Village Clerk

Published ____________________________________

AN ORDINANCE TO CREATE CHAPTER XX OF THE CODE OF ORDINANCES OF SISTER BAY RELATING TO THE CONTROL OF CONSTRUCTION SITE EROSION RESULTING FROM LAND DISTURBING CONSTRUCTION ACTIVITIES

FOREWORD

The Village Board of Sister Bay finds that urbanizing land uses have accelerated the process of soil erosion and sediment deposition within water bodies in the Village. During the construction process, soil is the most vulnerable to erosion by wind and water. Soil erosion from land disturbing and land development activities have significant adverse impacts upon area water resources and the health, safety, property and general welfare of the community, and diminish the public enjoyment and use of natural resources. Specifically, soil erosion can:

1. Endanger water resources by reducing water quality;
2. Cause the siltation of streams, lakes, wetlands or aquatic habitat for fish and other desirable species;
3. Diminish the capacity of water resources to support protected uses and a natural diversity of plant and animal life.
4. Clog existing drainage systems, increasing maintenance problems and costs;
5. Clearing during construction causes the loss of native vegetation necessary for terrestrial and aquatic habitat;

The purpose of the local regulation is to safeguard persons, protect property, prevent damage to the environment and promote the public welfare by guiding, regulating, and controlling the design, construction, use, and maintenance of any development or other activity which disturbs the topsoil or results in the movement of earth on land in the Village. The intent of this ordinance is to require use of best management practices to reduce the amount of sediment and other pollutants resulting from land disturbing construction activities on sites. The intent of this ordinance is also to allow the Village to comply with construction site performance standards for new development and redevelopment contained in subchapters III and IV of ch. NR 151, Wis. Adm. Code.

The Village Board of Sister Bay does hereby ordain that Chapter 30 of the code of ordinances of the Sister Bay is created to read as follows:

CHAPTER 30

CONSTRUCTION SITE EROSION CONTROL ORDINANCE

CONSTRUCTION SITE EROSION

30.01 AUTHORITY.

(1) This ordinance is adopted under the authority granted by s. 62.234, Wis. Stats. Except as otherwise specified in s. 62.234 Wis. Stats., s. 62.23, Wis. Stats., applies to this ordinance and to any amendments to this ordinance.

(2) The provisions of this ordinance are deemed not to limit any other lawful regulatory powers of the same governing body.

(3) The Village Board hereby designates the administering authority for the Village to administer and enforce the provisions of this ordinance to be the Public Works Department for plan review and Village Inspector for construction oversight.

(4) The requirements of this ordinance do not pre–empt more stringent erosion and sediment control or requirements that may be imposed by any of the following:

(a) Wisconsin Department of Natural Resources administrative rules, permits or approvals including those authorized under ss. 281.16 and 283.33, Wis. Stats.

(b) Targeted non–agricultural performance standards promulgated in rules by the Wisconsin Department of Natural Resources under s. NR 151.004, Wis. Adm. Code.

30.02 FINDINGS OF FACT.

The Village Board finds that runoff from land disturbing construction activity carries a significant amount of sediment and other pollutants to the waters of the state in Sister Bay.

30.03 PURPOSE.

It is the purpose of this ordinance to further the maintenance of safe and healthful conditions; prevent and control water pollution; prevent and control soil erosion; protect spawning grounds, fish and aquatic life; control building sites, placement of structures and land uses; preserve ground cover and scenic beauty; and promote sound economic growth, by minimizing the amount of sediment and other pollutants carried by runoff or discharged from land disturbing construction activity to waters of the state in Sister Bay.

30.04 APPLICABILITY AND JURISDICTION.

(1) APPLICABILITY.

(a) This ordinance applies to the following land disturbing construction activities except as provided under Chapter 30.04(1) (b):

1. A construction site, which has land disturbing construction activity.
2. The disturbed area has a slope of twelve percent (12%) or greater regardless of the size of the site.
3. Filling of one or more acres of land.

(b) This ordinance does not apply to the following:

1. A construction project that is exempted by federal statutes or regulations from the requirement to have a national pollutant discharge elimination system permit issued under chapter 40, Code of Federal Regulations, part 122, for land disturbing construction activity.
2. Nonpoint discharges from agricultural facilities and practices.
3. Nonpoint discharges from silviculture activities.
4. Routine maintenance for project sites under 1 acres of land disturbance if performed to maintain the original line and grade, hydraulic capacity or original purpose of the facility, unless land slopes are twelve percent (12%) or greater.

(c) Notwithstanding the applicability requirements in paragraph (a), this ordinance applies to construction or fill sites of any size that, in the opinion of the Public Works Department, are likely to result in runoff that exceeds the safe capacity of the existing drainage facilities or receiving body of water, that causes undue channel erosion, that increases water pollution by scouring or the transportation of particulate matter or that endangers property or public safety.

(2) JURISDICTION.
This ordinance applies to land disturbing construction activities on lands within the corporate limits and jurisdiction of Sister Bay as well as all lands located within the extraterritorial plat approval jurisdiction of Sister Bay.

(3) EXCLUSIONS AND CLARIFICATIONS.

(a) The following activities are exempt from the construction site erosion control requirements of this Chapter:

1. This ordinance is not applicable to construction activities conducted by a state agency for state owned projects, as defined under s. 227.01 (1), Wis. Stats., but also including the office of district attorney, which is subject to the state plan promulgated or a memorandum of understanding entered into under s. 281.33 (2), Wis. Stats.

(b) Notwithstanding the language of Chapter 30.04 (3)(a)1., the following activities are subject to the requirements of this ordinance:

1. Development of ponds or infiltration facilities, storm water systems or sewers, channelized water courses and commercial or institutional parks. However, the Public Works Department may certify that an Erosion and Sediment Control Plan developed by a state agency meets the requirements of this ordinance.

(c) The following activities are subject to the requirements of this ordinance:

1. Buildings and activities of municipalities
2. Buildings and activities of the School District
3. Local highway projects
4. Municipal streets
5. County roadways

30.05 PERMITTING OR CAUSING EROSION PROHIBITED
No person shall cause or permit erosion or the tracking or dropping of soil or sediment deposits on adjacent land, public streets or bodies of water from any land whether otherwise subject to this ordinance or not.

30.06 DEFINITIONS.

(1) “Administering authority” means a Village employee that is designated by the Village Board to administer this ordinance. The Public Works Department is designated this authority for plan review and issuing the permit. The Village Inspector is designated this authority for construction oversight. In the absence of the Village Inspector the Public Works Department is designated for construction oversight.

(2) “Agricultural facilities and practices ” has the meaning in s. 281.16(1), Wis. Stats.

(3) “Average annual rainfall” means a calendar year of precipitation, excluding snow, which is considered typical.

(4) ”Best management practice” or “BMP” means structural or non–structural measures, practices, techniques or devices employed to avoid or minimize soil, sediment or pollutants carried in runoff to waters of the state.

(5) “Business day” means a day the office of the [administering authority] is routinely and customarily open for business.

(6) "Cease and desist order" means a court–issued order to halt land disturbing construction activity that is being conducted without the required permit.
(7) "Construction site" means an area upon which one or more land disturbing construction activities occur, including areas that are part of a larger Common plan of development or sale where multiple separate and distinct land disturbing construction activities may be taking place at different times on different schedules but under one plan.
(8) "Division of land" means the creation from one parcel of two or more parcels or building sites where such creation occurs at one time or through the successive partition within a 5 year period.
(9) "Erosion" means the process by which the land's surface is worn away by the action of wind, water, ice or gravity.
(10) "Erosion and sediment control plan" means a comprehensive plan developed to address pollution caused by erosion and sedimentation of soil particles or rock fragments during construction.
(11) "Extraterritorial" means the unincorporated area within 3 miles of the Sister Bay corporate limits.
(12) "Final stabilization" means that all land disturbing construction activities at the construction site have been completed and that a uniform perennial vegetative cover has been established, with a density of at least 70 percent of the cover, for the unpaved areas and areas not covered by permanent structures, or that employ equivalent permanent stabilization measures.
(13) "Governing body" means the Village council.
(14) "Land disturbing construction activity" means any man–made alteration of the land surface resulting in a change in the topography or existing vegetative or non– vegetative soil cover, that may result in runoff and lead to an increase in soil erosion and movement of sediment into waters of the state. Land disturbing construction activity includes clearing and grubbing, demolition, excavating, pit trench dewatering, filling and grading activities.
(15) "MEP" or "maximum extent practicable" means a level of implementing best management practices in order to achieve a performance standard specified in this Chapter which takes into account the best available technology, cost effectiveness and other competing issues such as human safety and welfare, endangered and threatened resources, historic properties and geographic features. MEP allows flexibility in the way to meet the performance standards and may vary based on the performance standard and site conditions.
(16) "Performance standard" means a narrative or measurable number specifying the minimum acceptable outcome for a facility or practice.
(17) "Permit" means a written authorization made by the Public Works Department to the applicant to conduct land disturbing construction activity or to discharge post-construction runoff to waters of the state.
(18) "Pollutant" has the meaning given in s. 283.01 (13), Wis. Stats.
(19) "Pollution" has the meaning given in s. 281.01 (10), Wis. Stats.
(20) "Responsible party" means any entity holding fee title to the property or performing services to meet the performance standards of this ordinance through a contract or other agreement.
(21) "Runoff" means storm water or precipitation including rain, snow or ice melt or similar water that moves on the land surface via sheet or channelized flow.
(22) "Sediment" means settleable solid material that is transported by runoff, suspended within runoff or deposited by runoff away from its original location.
(23) "Separate storm sewer" means a conveyance or system of conveyances including roads with drainage systems, streets, catch basins, curbs, gutters, ditches, constructed channels or storm drains, which meets all of the following criteria:
   (a) Is designed or used for collecting water or conveying runoff.
   (b) Is not part of a combined sewer system.
   (c) Is not draining to a storm water treatment device or system.
   (d) Discharges directly or indirectly to waters of the state.

(24) "Site" means the entire area included in the legal description of the land on which the land disturbing construction activity is proposed in the permit application.
(25) "Stop work order" means an order issued by the Village Inspector which requires that all construction activity on the site be stopped.
(26) "Technical standard" means a document that specifies design, predicted performance and operation and maintenance specifications for a material, device or method.
(27) "Waters of the state" has the meaning given in s. 281.01 (18), Wis. Stats.

30.07 TECHNICAL STANDARDS.

(1) STANDARDS AND SPECIFICATIONS.
All BMPs required to comply with this ordinance shall meet the design criteria, standards and specifications based on any of the following:

(a) Applicable design criteria, standards and specifications identified in the Wisconsin Construction Site Best Management Practice Handbook, WDNR Pub. WR–222 November 1993 Revision, or its successors.
(b) Other design guidance and technical standards identified or developed by the Wisconsin Department of Natural Resources under subchapter V of chapter NR 151, Wis. Adm. Code.
(c) Conservation practice standards maintained by the Standards Oversight Council or cooperating agency.
(d) For this ordinance, average annual basis is calculated using the appropriate annual rainfall or runoff factor, also referred to as the R factor, or an equivalent design storm using a NRCS type II distribution, with consideration given to the period of disturbance.
(e) U.S. Natural Resources Conservation Service's "Field Office Technical Guide;" or its successor.
(f) U.S. Natural Resources Conservation Service's Engineering Field Manual for Conservation Practices.
(g) U.S. Natural Resources Conservation Service's Engineering Handbook.

(2) DESIGN CRITERIA. The applicant for a permit may employ structural or nonstructural measures necessary to achieve all applicable standards set out in this ordinance. However, these measures will be evaluated to determine that they follow currently accepted design criteria and engineering standards. The following general principles shall be used when evaluating control plans and granting permits under this ordinance:
(a) The smallest area of land shall be exposed for the shortest period at any given time during development.
(b) The rough grading of the lot shall include backfilling the basement. All excess earth shall be hauled off the lot before the end of construction.
(c) Control of the increased runoff and pollutants caused by changed soil and surface conditions during and after development.
(d) The plan of development shall relate to the topography and soils of the site so that the lowest potential for erosion is created.
(e) Clearing activities shall consider protection of natural resources such as trees, wetlands and water bodies.
(f) Grading shall retain natural drainage patterns.
(g) Phasing shall be required on all disturbed sites greater than thirty acres, with the size of each phase to be established during plan review.
(h) Soil must be stabilized within five days of clearing or inactivity in construction.
(i) Soil stockpiles must be stabilized or covered at the end of each work day.
(j) At the close of the construction season, the entire site must be stabilized using a method that does not require germination to control erosion.
(k) Techniques shall be employed to prevent the blowing of dust or sediment from the site.

(l) Techniques that divert runoff around disturbed areas shall be employed.

(3) OTHER TECHNICAL STANDARDS. Other technical standards not identified or developed in Chapter 30.07 (1), may be used provided that the methods have been approved by the Public Works Department.

30.08 PERFORMANCE STANDARDS.

(1) RESPONSIBLE PARTY. The responsible party shall implement an erosion and sediment control plan, developed in accordance with Chapter 30.11, that incorporates the requirements of this section.

(2) PLAN. A written plan shall be developed in accordance with Chapter 30.11 and implemented for each construction site.

(3) EROSION AND OTHER POLLUTANT CONTROL REQUIREMENTS. The plan required under Chapter 30.08 (2) shall include the following:

(a) BMPs that, by design, achieve to the maximum extent practicable, a reduction of 80% of the sediment load carried in runoff, on an average annual basis, as compared with no sediment or erosion controls until the construction site has undergone final stabilization. No person shall be required to exceed an 80% sediment reduction to meet the requirements of this paragraph. Erosion and sediment control BMPs may be used alone or in combination to meet the requirements of this paragraph. Credit toward meeting the sediment reduction shall be given for limiting the duration or area, or both, of land disturbing construction activity, or other appropriate mechanism.

1. Soil loss prediction tools that estimate the sediment load leaving the construction site under varying land and management conditions, or methodology identified in subch. V. of ch. NR 151, Wis. Adm. Code, may be used to calculate sediment reduction.

(b) Notwithstanding Chapter 30.08 (3) (a), if BMPs cannot be designed and implemented to reduce the sediment load by 80%, on an average annual basis, the plan shall include a written and site–specific explanation as to why the 80% reduction goal is not attainable and the sediment load shall be reduced to the maximum extent practicable.

(c) Where appropriate, the plan shall include sediment controls to do all of the following to the maximum extent practicable:

1. Prevent tracking of sediment from the construction site onto roads and other paved surfaces.
2. Prevent the discharge of sediment as part of site de–watering.
3. Protect the separate storm drain inlet structure from receiving sediment.

(d) The use, storage and disposal of chemicals, cement and other compounds and materials used on the construction site shall be managed during the construction period, to prevent their entrance into waters of the state. However, projects that require the placement of these materials in waters of the state, such as constructing bridge footings or BMP installations, are not prohibited by this paragraph.

(4) LOCATION. The BMPs used to comply with this section shall be located prior to runoff entering waters of the state.

(5) ALTERNATE REQUIREMENTS. The Public Works Department may establish storm water management requirements more stringent than those set forth in this section if the Public Works Department determines that an added level of protection is needed for sensitive resources.

30.09 POST CONSTRUCTION STORM WATER MANAGEMENT

(1) POST CONSTRUCTION STORM WATER MANAGEMENT. Construction sites required to meet the requirements of this ordinance shall also meet the requirements of Chapter 31, Storm Water Management, of the Village Code of Ordinances.

30.10 PERMITTING REQUIREMENTS, PROCEDURES AND FEES.

(1) PERMIT REQUIRED. No responsible party may Commence a land disturbing construction activity subject to this ordinance without receiving prior approval of an erosion and sediment control plan for the site and a permit from the Public Works Department.

(2) PERMIT APPLICATION AND FEES. At least one responsible party desiring to undertake a land disturbing construction activity subject to this ordinance shall submit an application for a permit and an erosion and sediment control plan that meets the requirements of Chapter 30.11 and shall pay an application fee that is determined by the Board of Public Works to the Public Works Department. By submitting an application, the applicant is authorizing the Public Works Department to enter the site to obtain information required for the review of the erosion and sediment control plan.

(3) REVIEW AND APPROVAL OF PERMIT APPLICATION. The Public Works Department shall review any permit application that is submitted with an erosion and sediment control plan, and the required fee. The following approval procedure shall be used:

(a) Within 30 business days of the receipt of a complete permit application, as required by Chapter 30.10 (2), the Public Works Department shall inform the applicant whether the application and plan are approved or disapproved based on the requirements of this ordinance.

(b) If the permit application and plan are approved, the Public Works Department shall issue the permit.

(c) If the permit application or plan is disapproved, the Public Works Department shall state in writing the reasons for disapproval.

(d) The Public Works Department may request additional information from the applicant. If additional information is submitted, the Public Works Department shall have 10 business days from the date the additional information is received to inform the applicant that the plan is either approved or disapproved.

(e) Failure by the Public Works Department to inform the permit applicant of a decision within 30 business days of a required submittal shall be deemed to mean approval of the submittal and the applicant may proceed as if a permit had been issued.

(4) SURETY BOND. As a condition of approval and issuance of the permit, the Public Works Department shall require the applicant to deposit a surety bond or irrevocable letter of credit to guarantee a good faith execution of the approved erosion control plan and any permit conditions.

(5) PERMIT REQUIREMENTS. All permits shall require the responsible party to:

(a) Notify the Village Inspector a minimum of 2 business days prior to Commencing any land disturbing construction activity.

(b) Notify the Village Inspector of completion of any BMPs within 7 days after their installation.

(c) Obtain permission in writing from the Public Works Department prior to any modification pursuant to Chapter 30.08(3) of the Erosion and Sediment Control Plan.

(d) Install all BMPs as identified in the approved erosion and sediment control plan.

(e) Maintain all road drainage systems, stormwater drainage systems, BMPs and other facilities identified in the erosion and sediment control plan.

(f) Repair any siltation or erosion damage to adjoining surfaces and drainage ways resulting from land disturbing construction activities and document repairs in a site erosion control log.

(g) Inspect the BMPs within 24 hours after each rain of 0.5 inches or more which results in runoff during active construction periods, and at least once each week make needed repairs and document the findings of the inspections in a site erosion control log with the date of inspection, the name of the person conducting the inspection, and a description of the present phase of the construction at the site.

(h) Allow the Public Works Department and Village Inspector to enter the site for the purpose of inspecting compliance with the erosion and sediment control plan or for performing any work necessary to bring the site into compliance with the control plan. Keep a copy of the erosion and sediment control plan at the construction site.

(6) PERMIT CONDITIONS. Permits issued under this section may include conditions established by Public Works Department in addition to the requirements set forth in Chapter 30.10 (5), where needed to assure compliance with the performance standards in Chapter 30.08.

(7) PERMIT DURATION. Permits issued under this section shall be valid for a period of 180 days, or the length of the building permit or other construction authorizations, whichever is longer, from the date of issuance. The Public Works Department may extend the period one or more times for up to an additional 180 days. The Public Works Department may require additional BMPs as a condition of the extension if they are necessary to meet the requirements of this ordinance.

(8) MAINTENANCE. The responsible party throughout the duration of the construction activities shall maintain all BMPs necessary to meet the requirements of this ordinance until the site has undergone final stabilization.

(9) PERMIT TRANSFERS. A permit may be transferred to a new Responsible Party according to the following conditions:

(a) When a permittee and landowner act to transfer an interest in property subject to an approved plan prior to completion of the proposed steps to attain soil stabilization, the permittee must secure approval from the Administrative Authority.

(b) When a permittee and landowner transfers ownership, possession or control of real estate subject to either or both an uncompleted erosion control plan or stormwater management plan, the successor in interest to any portion of the real estate shall be responsible to control soil erosion and runoff and shall comply with the minimum standards provided in this ordinance.

(c) When ownership, possession or control of property subject to an uncompleted erosion control or stormwater management plan, or both, is transferred, the former owner (seller) shall notify the new owner (buyer) as to the current status of compliance with notice to the Administrative Authority, and provide a copy of the erosion control plan.

(d) Transfers of interest in real estate subject to an approved, uncompleted plan may be conducted consistent with this ordinance under any of the following arrangements.

1. The transferee shall file a new, approved erosion control plan with the Administrative Authority;
2. The transferee shall obtain an approved assignment from the Administrative Authority as sub-permittee to complete that portion of the approved plan regulating soil erosion and runoff on the transferee's property;
3. The permittee shall provide the Administrative Authority with a duly completed and executed continuing surety bond or certified check in an amount sufficient to complete the work proposed in the approved plan; at the time of transfer the permittee may seek to reduce the surety bond or certified check to the appropriate amount to complete remaining work. If the transferor enters into escrow agreements with transferees to complete an approved plan, these funds shall be available to the Administrative Authority to attain plan compliance. When an approved erosion control plan and, if required, a stormwater management plan is or are not completed as proposed, the Administrative Authority may use the surety bond to complete remaining work to achieve plan compliance.

30.11 EROSION AND SEDIMENT CONTROL PLAN, STATEMENT, AND AMENDMENTS.

(1) EROSION AND SEDIMENT CONTROL PLAN.

(a) An erosion and sediment control plan shall be prepared and submitted to the Public Works Department.

(b) The erosion and sediment control plan shall be designed to meet the performance standards in Chapter 30.08 and other requirements of this ordinance.

(c) The erosion and sediment control plan shall address pollution caused by soil erosion and sedimentation during construction and up to final stabilization of the site. The erosion and sediment control plan shall include, at a minimum, the following items:

1. The name(s) and address(es) of the owner or developer of the site, and of any consulting firm retained by the applicant, together with the name of the applicant's principal contact at such firm. The application shall also include start and end dates for construction.
2. Description of the site and the nature of the construction activity, including representation of the limits of land disturbance on a Village street map, or, for extraterritorial plat reviews, a United States Geological Service 7.5 minute series topographic map.
3. A sequence of construction of the development site, including stripping and clearing; rough grading; construction of utilities, infrastructure, and buildings; and final grading and landscaping. Sequencing shall identify the expected date on which clearing will begin, the estimated duration of exposure of cleared areas, areas of clearing, installation of temporary erosion and sediment control measures, and establishment of permanent vegetation.
4. Estimates of the total area of the site and the total area of the site that is expected to be disturbed by construction activities.
5. Estimates, including calculations, if any, of the runoff coefficient of the site before and after construction activities are completed.
6. Calculations to show the expected percent reduction in the average annual sediment load carried in runoff as compared to no sediment or erosion controls.
7. Existing data describing the surface soil as well as subsoils.
8. Depth to groundwater, as indicated by existing information.
9. Name of the immediate named receiving water from the United States Geological Service 7.5 minute series topographic maps.

(d) The erosion and sediment control plan shall include a site map. The site map shall include the following items and shall be at a scale not greater than 50 feet per inch and at a contour interval not to exceed two feet.

1. Existing topography, vegetative cover, natural and engineered drainage systems, roads and surface waters. Lakes, streams, wetlands, channels, ditches and other watercourses on and immediately adjacent to the site shall be shown. Any identified 100–year flood plains, flood fringes and floodways shall also be shown.
2. Boundaries of the construction site.
3. Limits of disturbed areas.
4. Limits of impervious surfaces.
5. Drainage patterns and approximate slopes anticipated after major grading activities.
6. Areas of soil disturbance.
7. Location of structural and non–structural controls identified in the plan including dewatering measures.
8. Location of areas where stabilization practices will be employed.
9. Cross sections and profiles within road ditches.

10. Kinds of utilities and areas of installation, including special erosion control practices for utility installation.
11. Culvert sizes.
12. Plans and cross sections for storm water facilities.
13. Watershed size for each drainage area.
14. Design discharges for ditches and structural measures.
15. Runoff velocities.
16. Areas which will be vegetated following construction.
17. Fertilizer and seed types, rates of applications and recommendations.
18. Time schedule for stabilization of ditches and slopes.
19. Provision for sequential steps mitigating erosive effect of land disturbing activities to be followed in appropriate order and in a manner consistent with accepted erosion control methodology suitable to proposed sites and amenable to prompt revegetation.
20. Provisions such as stone access drives to prevent mud-tracking off-site onto public thoroughfares during the construction period and all incidental mud tracking shall be cleaned up and removed by the end of each working day using proper cleaning and disposal methods.
21. Provision to provide protection of existing infiltration facilities from sedimentation, whether publicly or privately owned.
22. Areal extent of wetland acreage on the site and locations where storm water is discharged to a surface water or wetland.
23. Locations of all surface waters and wetlands within one mile of the construction site as depicted on existing maps.
24. An alphanumeric or equivalent grid overlying the entire construction site map.
25. Any other information necessary to reasonably determine the location, nature and condition of any physical or environmental features of the site.

(e) Each erosion and sediment control plan shall include a description of appropriate controls and measures that will be performed at the site to prevent pollutants from reaching waters of the state. The plan shall clearly describe the appropriate control measures for each major activity and the timing during the construction process that the measures will be implemented. The description of erosion controls shall include, when appropriate, the following minimum requirements:

1. Description of interim and permanent stabilization practices, including a practice implementation schedule. Site plans shall ensure that existing vegetation is preserved where attainable and that disturbed portions of the site are stabilized.
2. Description of structural practices to divert flow away from exposed soils, store flows or otherwise limit runoff and the discharge of pollutants from the site. Unless otherwise specifically approved in writing by the Public Works Department, structural measures shall be installed on upland soils.
3. Management of overland flow at all sites, unless otherwise controlled by outfall controls.
4. Trapping of sediment in channelized flow.
5. Staging construction to limit bare areas subject to erosion.
6. Protection of downslope drainage inlets where they occur.
7. Minimization of tracking at all sites.
8. Clean up of off–site sediment deposits.
9. Proper disposal of building and waste materials at all sites.
10. Stabilization of drainage ways.
11. Control of soil erosion from dirt stockpiles.

12. Installation of permanent stabilization practices as soon as possible after final grading.
13. Minimization of dust to the maximum extent practicable.

(f) The erosion and sediment control plan shall require that velocity dissipation devices be placed at discharge locations and along the length of any outfall channel, as necessary, to provide a non–erosive flow from the structure to a water course so that the natural physical and biological characteristics and functions are maintained and protected.

(2) EROSION AND SEDIMENT CONTROL PLAN STATEMENT. For each construction site identified under Chapter 30.04 (1), an erosion and sediment control plan statement shall be prepared. This statement shall be submitted to the Public Works Department. The control plan statement shall briefly describe the site, including a site map. Further, it shall also include the best management practices that will be used to meet the requirements of the ordinance, including the site development schedule.

(3) AMENDMENTS. The applicant shall amend the plan if any of the following occur:

(a) There is a change in design, construction, operation or maintenance at the site which has the reasonable potential for the discharge of pollutants to waters of the state and which has not otherwise been addressed in the plan.

(b) The actions required by the plan fail to reduce the impacts of pollutants carried by construction site runoff.

(c) The Public Works Department notifies the applicant of changes needed in the plan.

30.12 FEE SCHEDULE.

The fees referred to in other sections of this ordinance shall be established by the Board of Public Works and may from time to time be modified by resolution. A schedule of the fees established by the Board of Public Works shall be available for review in the Village Engineers office.

30.13 INSPECTION.

If land disturbing construction activities are being carried out without a permit required by this ordinance, the Public Works Department or Village Inspector may enter the land pursuant to the provisions of ss. 66.0119(1), (2), and (3), Wis. Stats.

30.14 ENFORCEMENT.

(1) The Village Inspector may post a stop–work order if any of the following occurs:

(a) Any land disturbing construction activity regulated under this ordinance is being undertaken without a permit.

(b) The erosion and sediment control plan is not being implemented in a good faith manner.

(c) The conditions of the permit are not being met.

(2) If the responsible party does not cease activity as required in a stop–work order posted under this section or fails to comply with the erosion and sediment control plan or permit conditions, the Village Inspector or the Public Works Department may revoke the permit.

(3) If the responsible party, where no permit has been issued, does not cease the activity after being notified by the Village Inspector, or if a responsible party violates a stop–work order posted under sub. (1), the Village Inspector may request the Village attorney to obtain a cease and desist order in any court with jurisdiction.

(4) The Village Inspector may retract the stop–work order issued under Chapter 30.14 (1) or the permit revocation under Chapter 30.14 (2). Completion of or changes to the Erosion and Sediment Control required for permit compliance must be approved by the Public Works Department prior to retraction of the stop-work order.

(5) After posting a stop–work order under Chapter 30.14 (1), the Village Inspector may issue a notice of intent to the responsible party of its intent to perform work necessary to comply with

this ordinance. The Public Works Department or Village Inspector may go on the land and Commence the work after issuing the notice of intent. The costs of the work performed under this subsection by the Sister Bay, plus interest at the rate authorized by the Sister Bay shall be billed to the responsible party. In the event a responsible party fails to pay the amount due, the clerk shall enter the amount due on the tax rolls and collect as a special assessment against the property pursuant to subch. VII of ch. 66, Wis. Stats.

(6) Any person violating any of the provisions of this ordinance shall be subject to a forfeiture of not less than $250.00 nor more than $500.00 and the costs of prosecution for each violation. Each day a violation exists shall constitute a separate offense.

(7) Compliance with the provisions of this ordinance may also be enforced by injunction in any court with jurisdiction. It shall not be necessary to prosecute for forfeiture or a cease and desist order before resorting to injunctional proceedings.

30.15 APPEALS.

(1) BOARD OF PUBLIC WORKS. The board of public works created pursuant to Chapter 1 of the Village's ordinance pursuant to s. 62.23(7)(e), Wis. Stats.:

(a) Shall hear and decide appeals where it is alleged that there is error in any order, decision or determination made by the administering authority in administering this ordinance except for cease and desist orders obtained under Chapter 30.14 (3).

(b) Upon appeal, may authorize variances from the provisions of this ordinance which are not contrary to the public interest and where owing to special conditions a literal enforcement of the provisions of the ordinance will result in unnecessary hardship; and

(c) Shall use the rules, procedures, duties and powers authorized by statute in hearing and deciding appeals and authorizing variances.

(2) WHO MAY APPEAL. Appeals to the board of public works may be taken by any aggrieved person or by any office, department, board, or bureau of the Sister Bay affected by any decision of the administering authority.

30.16 SEVERABILITY.

If a court of competent jurisdiction judges any section, clause, provision or portion of this ordinance unconstitutional or invalid, the remainder of the ordinance shall remain in force and not be affected by such judgment.

30.17 EFFECTIVE DATE.

This ordinance shall be in force and effect from and after its adoption and publication. The above and foregoing ordinance was duly adopted by the Village Board of the Sister Bay on the __ day of _____________ 200_.

Dated this ___ day of ___________ 200_.

First Reading ____ ______________________________

Trustee

Second Reading _____________________________________

Adopted____

Approved ______________________________________
President

Attest ____
Village Clerk

Published ______________________________________

ILLICIT DISCHARGE PROGRAM PROPOSAL

I. PURPOSE

- Screening of storm water and system outfalls to identify possible illicit connections to the municipal storm water conveyance system.

II. PROGRAM GUIDELINES

- All inspections of the municipal storm water conveyance system for illicit discharges will be done during a low-flow period. (No rain events for the previous 72 hours).
- All known outfalls will be inspected on a periodic basis by Village Staff under the low flow conditions to determine if any unknown cross connections are present.
- The inspection reports will be logged and analyzed for changes that are occurring within the system.

III. DETECTION PROCEDURES

- During the annual inspections, if any unusual flow or possible contaminated discharge is observed, the following steps will be taken:
    - Identify the outfall, and possible connections to conveyance system.
    - If needed, collect a grab sample of the discharge and make observations that are pertinent for determination of the discharge.
    - If feasible, identify the cross connection point, or business and notify the appropriate person of the cross connection.
    - Determine a remediation process or method, and at this point the Wisconsin Department of Natural Resources will be advised of the illicit connection.
    - If someone can be held responsible for the illicit connections, they will have to take the appropriate actions or steps to correct the problem, if no person or group can be identified, the Village of Sister Bay or appropriate agency will begin the remediation process.

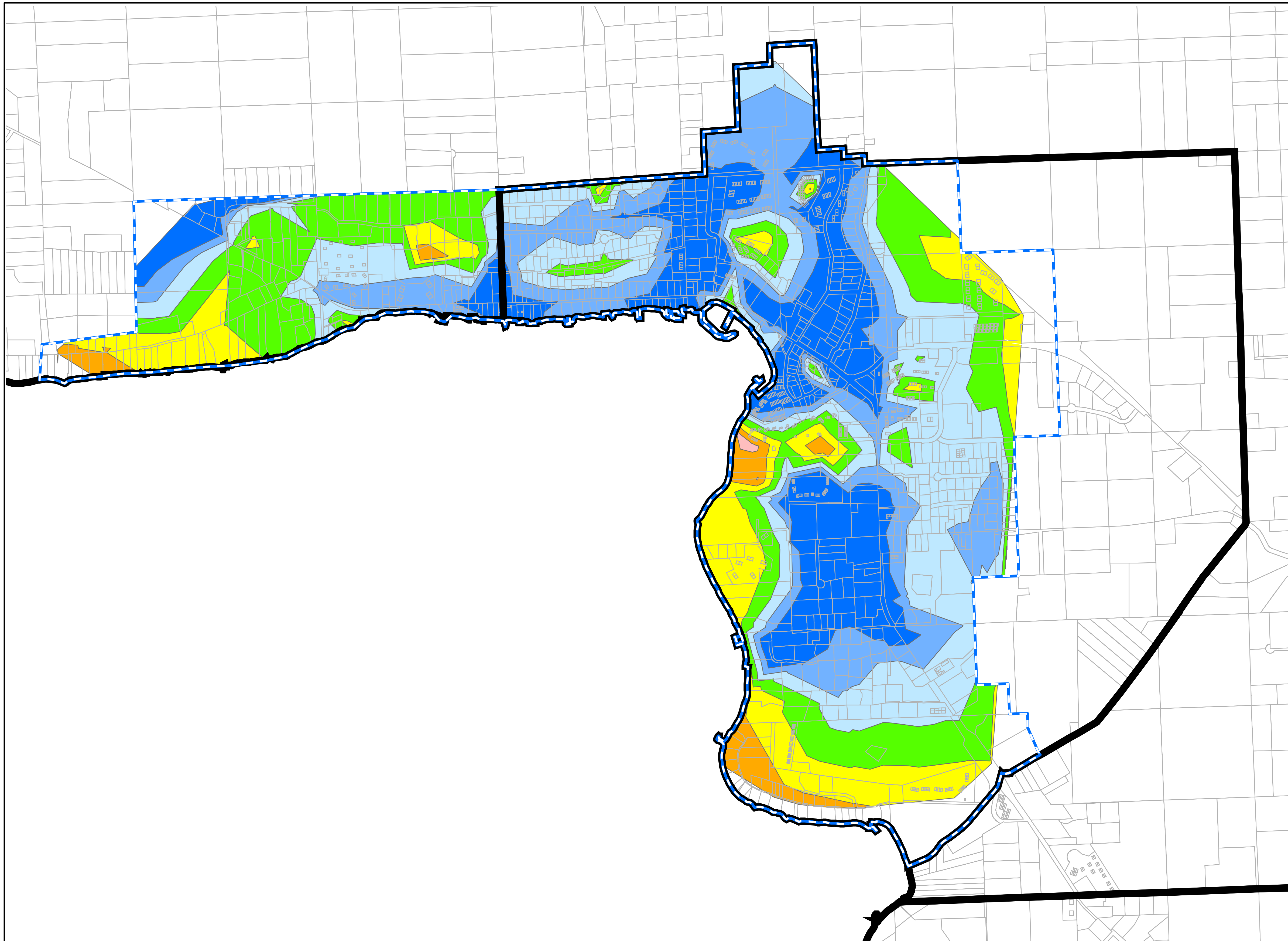
IV. PUBLIC AWARENESS

- Pamphlets will be made available on what to do when a member of the public finds an illicit discharge.
- Building inspections and general inspections will be done in areas where possible illicit connections could be present.
- During the building permit process, builders and contractors will be advised to contact the municipality if any illicit connections are found during the course of the projects.

APPENDIX E

WATER SYSTEM PRESSURE AND FIRE FLOW MAPS

ALTERNATE NEW TOWER LOCATIONS



Legend

Maximum Day Fire Flow

- 0-500 gpm
- 500-1000 gpm
- 1000-1500 gpm
- 1500-2000 gpm
- 2000-2500 gpm
- 2500-3000 gpm
- 3000-3500 gpm
- 3500+ gpm
- Municipal Boundaries
- Parcels
- Existing Water Service Area

Conditions:
-Proposed Water Tower at Northern Site
-Combined Pressure Zone

N

0 750 1,500
Feet

Source:
Door County and Bay Lake RPC.
Projection:
Wisconsin State Plane
Map by:
SEH

SEH
425 WEST WATER STREET
SUITE 300
APPLETON, WI 54911-6058
PHONE: (920) 380-2800
FAX: (920) 380-2801
www.sehinc.com

PROJECT:
ASISTB0502.00

DATE:
10/18/2006

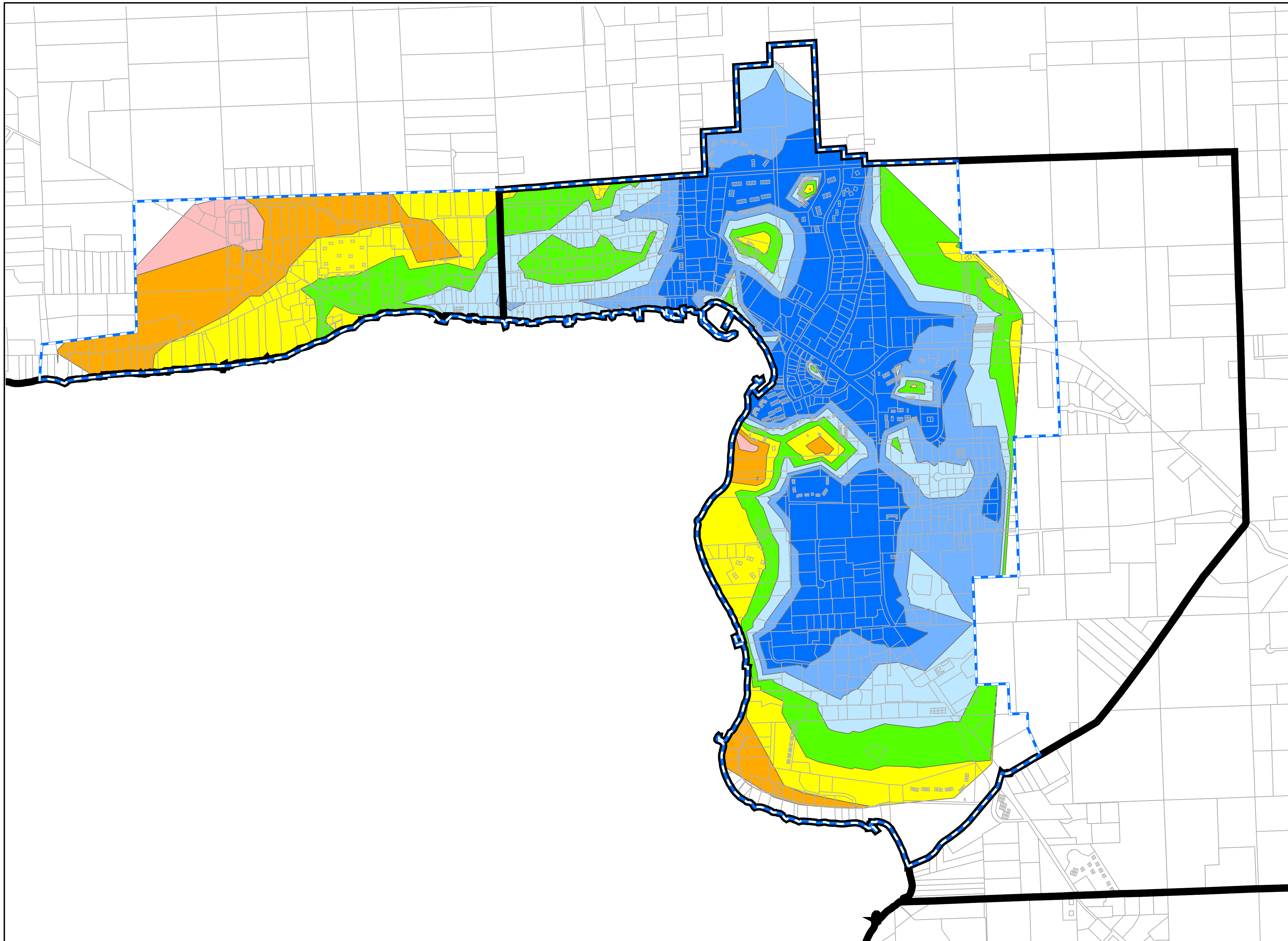
COMPREHENSIVE UTILITIES PLAN

Village of Sister Bay

Maximum Day Available Fire Flow

Figure 1

Map Document: (S:\PT\S\Sistb\050200\GIS\WaterModel\Additional_Scenarios_MXDs\Fig01.mxd)
10/17/2006 -- 4:58:35 PM



Legend

Maximum Day Fire Flow

- 0-500 gpm
- 500-1000 gpm
- 1000-1500 gpm
- 1500-2000 gpm
- 2000-2500 gpm
- 2500-3000 gpm
- 3000-3500 gpm
- 3500+ gpm
- Municipal Boundaries
- Parcels
- Existing Water Service Area

Conditions:
-Proposed Water Tower at Standpipe Site
-Combined Pressure Zone

N

0 750 1,500
Feet

Source:
Door County and Bay Lake RPC.
Projection:
Wisconsin State Plane
Map by:
SEH

SEH
425 WEST WATER STREET
SUITE 300
APPLETON, WI 54911-6058
PHONE: (920) 380-2800
FAX: (920) 380-2801
www.sehinc.com

PROJECT:
ASISTB0502.00

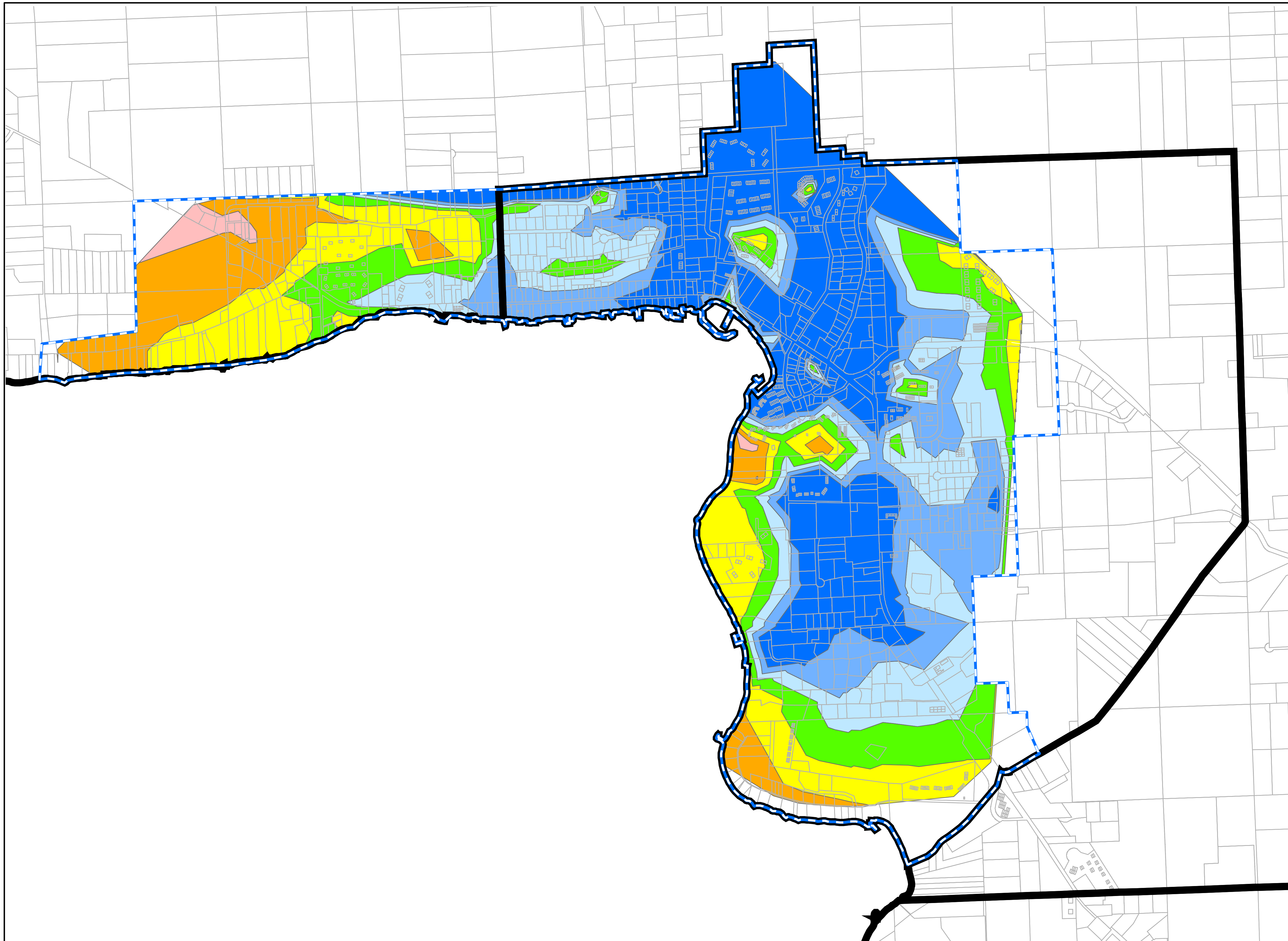
DATE:
10/18/2006

COMPREHENSIVE UTILITIES PLAN
Village of Sister Bay

Maximum Day Available Fire Flow

Figure 2

Map Document: (S:\PT\S\Sistb\050200\GIS\WaterModel\Additional_Scenarios_MXDs\Fig02.mxd)
10/17/2006 -- 5:03:09 PM



Legend

Maximum Day Fire Flow

- 0-500 gpm
- 500-1000 gpm
- 1000-1500 gpm
- 1500-2000 gpm
- 2000-2500 gpm
- 2500-3000 gpm
- 3000-3500 gpm
- 3500+ gpm
- Municipal Boundaries
- Parcels
- Existing Water Service Area

Conditions:
-Proposed Water Tower at WWTP Site
-Combined Pressure Zone

N

0 750 1,500
Feet

Source:
Door County and Bay Lake RPC.
Projection:
Wisconsin State Plane
Map by:
SEH

SEH
425 WEST WATER STREET
SUITE 300
APPLETON, WI 54911-6058
PHONE: (920) 380-2800
FAX: (920) 380-2801
www.sehinc.com

PROJECT:
ASISTB0502.00

DATE:
10/18/2006

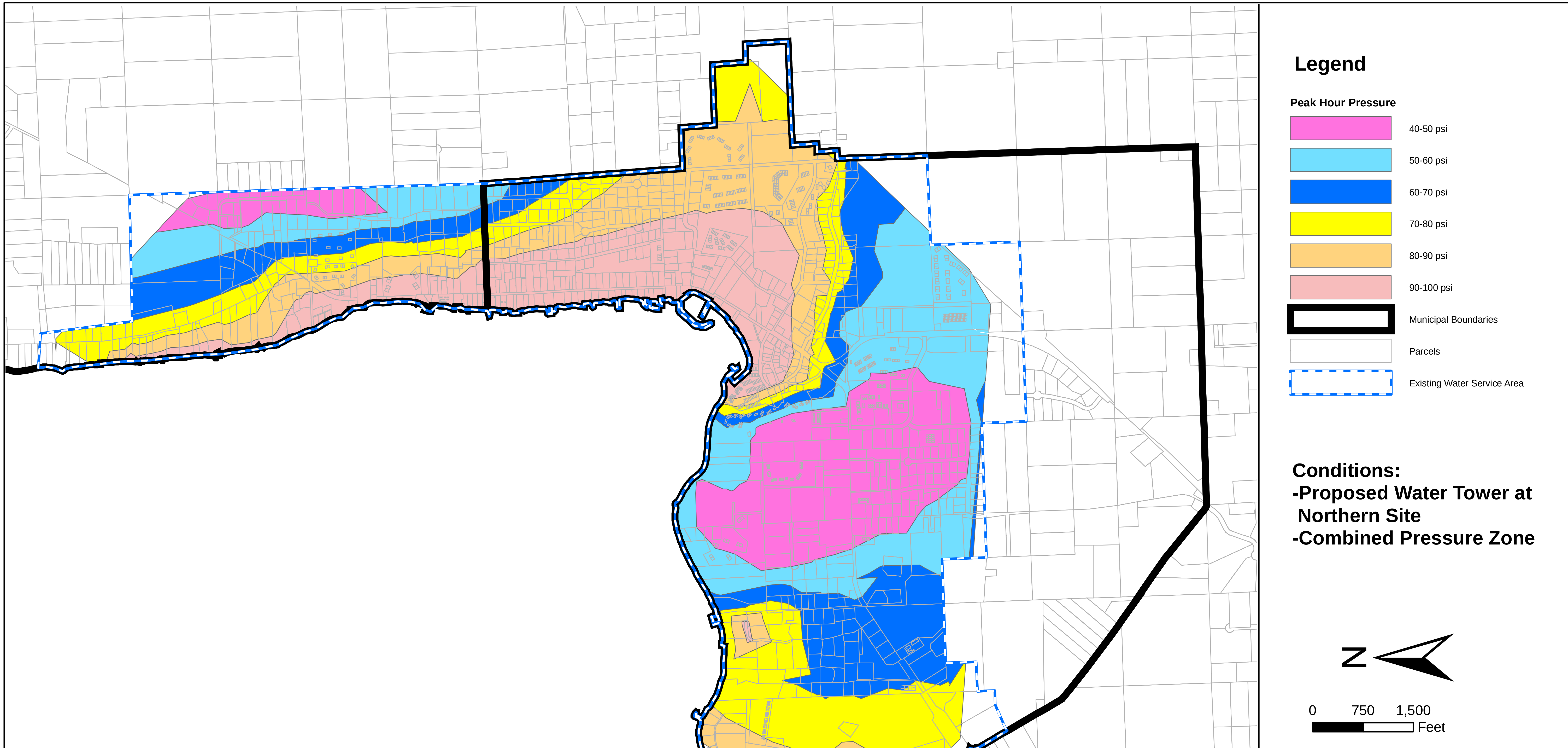
COMPREHENSIVE UTILITIES PLAN

Village of Sister Bay

Maximum Day Available Fire Flow

Figure 3

Map Document: (S:\PT\S\Sistb\050200\GIS\WaterModel\Additional_Scenarios_MXDs\Fig03.mxd)
10/17/2006 -- 5:05:13 PM



Source:
Door County and Bay Lake RPC.
Projection:
Wisconsin State Plane
Map by:
SEH

SEH
425 WEST WATER STREET
SUITE 300
APPLETON, WI 54911-6058
PHONE: (920) 380-2800
FAX: (920) 380-2801
www.sehinc.com

PROJECT:
ASISTB0502.00

DATE:
10/18/2006

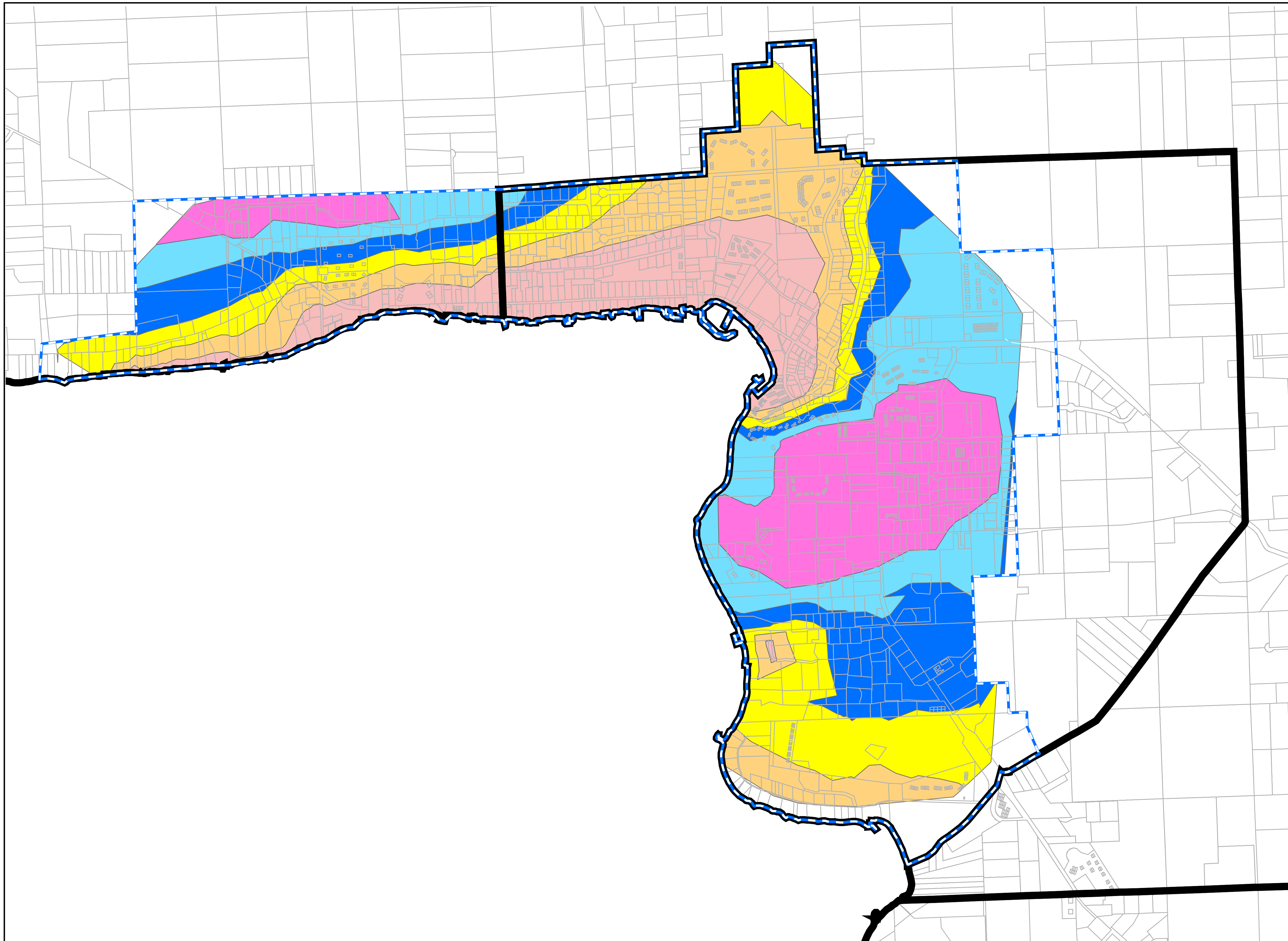
COMPREHENSIVE UTILITIES PLAN

Village of Sister Bay

Peak Hour Pressure

Figure 4

Map Document: (S:\PT\S\Sistb\050200\GIS\WaterModel\Additional_Scenarios_MXDs\Fig04.mxd)
10/17/2006 -- 5:23:43 PM



Legend

Peak Hour Pressure

- 40-50 psi
- 50-60 psi
- 60-70 psi
- 70-80 psi
- 80-90 psi
- 90-100 psi
- Municipal Boundaries
- Parcels
- Existing Water Service Area

Conditions:
-Proposed Water Tower at Standpipe Site
-Combined Pressure Zone

N

0 750 1,500
Feet

Source:
Door County and Bay Lake RPC.
Projection:
Wisconsin State Plane
Map by:
SEH

SEH
425 WEST WATER STREET
SUITE 300
APPLETON, WI 54911-6058
PHONE: (920) 380-2800
FAX: (920) 380-2801
www.sehinc.com

PROJECT: ASISTB0502.00

DATE: 10/18/2006

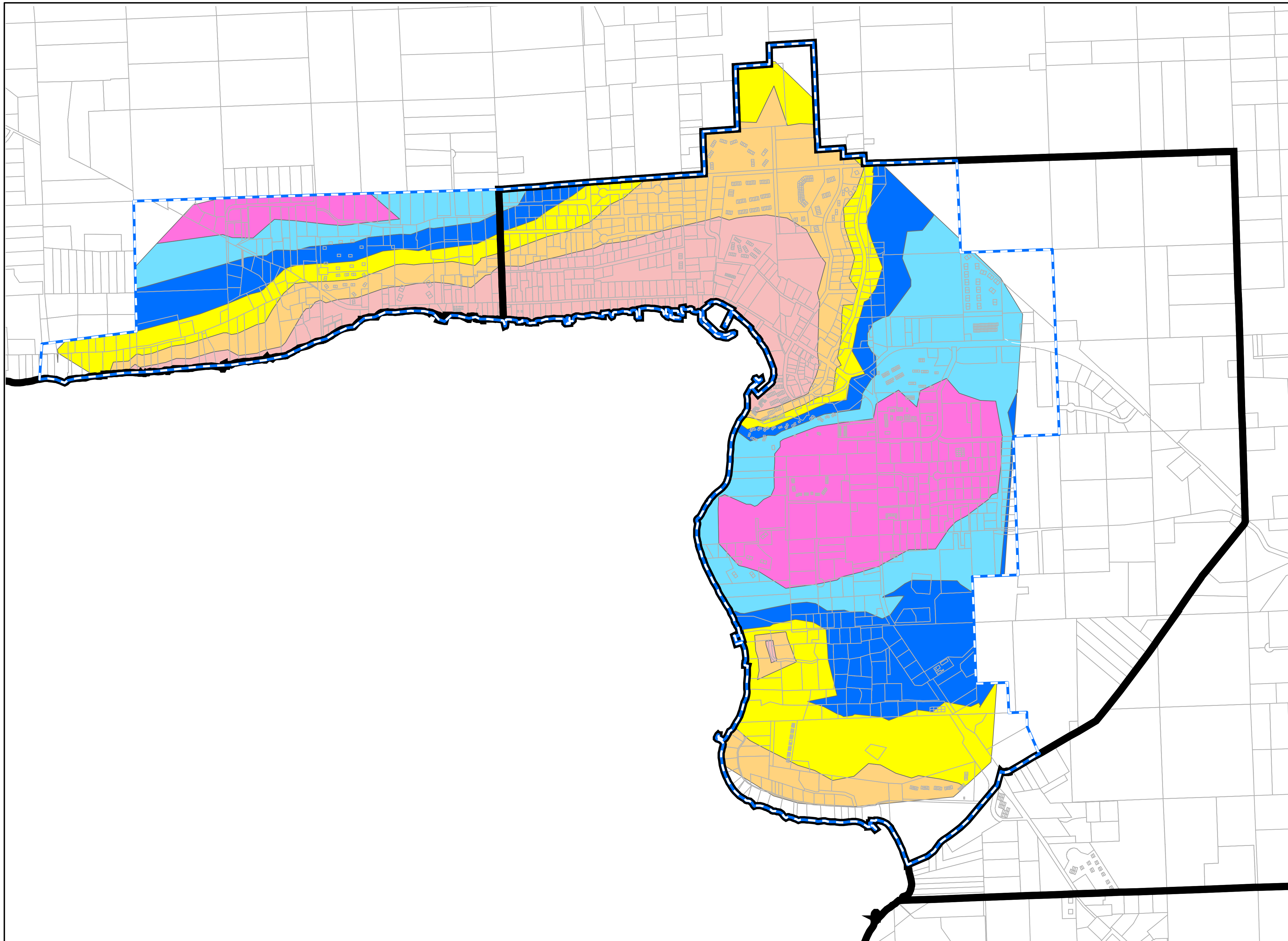
COMPREHENSIVE UTILITIES PLAN

Village of Sister Bay

Peak Hour Pressure

Figure 5

Map Document: (S:\PT\S\Sistb\050200\GIS\WaterModel\Additional_Scenarios_MXDs\Fig05.mxd)
10/17/2006 -- 5:26:27 PM



Legend

Peak Hour Pressure

- 40-50 psi
- 50-60 psi
- 60-70 psi
- 70-80 psi
- 80-90 psi
- 90-100 psi
- Municipal Boundaries
- Parcels
- Existing Water Service Area

Conditions:
-Proposed Water Tower at WWTP Site
-Combined Pressure Zone

N

0 750 1,500 Feet

Source:
Door County and Bay Lake RPC.
Projection:
Wisconsin State Plane
Map by:
SEH

SEH
425 WEST WATER STREET
SUITE 300
APPLETON, WI 54911-6058
PHONE: (920) 380-2800
FAX: (920) 380-2801
www.sehinc.com

PROJECT: ASISTB0502.00

DATE: 10/18/2006

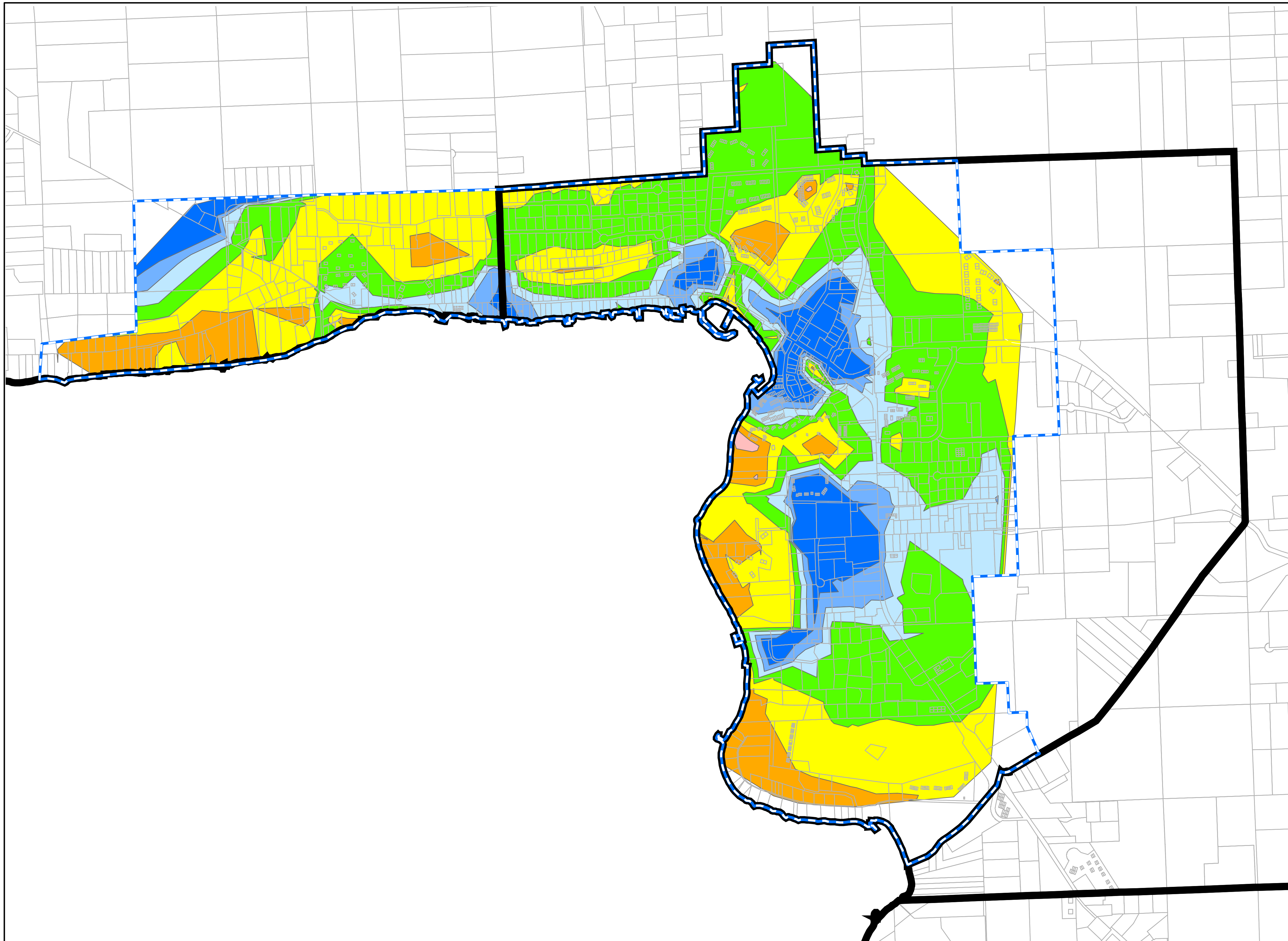
COMPREHENSIVE UTILITIES PLAN

Village of Sister Bay

Peak Hour Pressure

Figure 6

Map Document: (S:\PT\S\Sistb\050200\GIS\WaterModel\Additional_Scenarios_MXDs\Fig06.mxd)
10/17/2006 -- 5:27:15 PM



Legend

Maximum Day Fire Flow

- 0-500 gpm
- 500-1000 gpm
- 1000-1500 gpm
- 1500-2000 gpm
- 2000-2500 gpm
- 2500-3000 gpm
- 3000-3500 gpm
- 3500+ gpm
- Municipal Boundaries
- Parcels
- Existing Water Service Area

Conditions:
-Proposed Water Tower at Northern Site
-Separated Pressure Zones

N

0 750 1,500 Feet

Source:
Door County and Bay Lake RPC.
Projection:
Wisconsin State Plane
Map by:
SEH

SEH
425 WEST WATER STREET
SUITE 300
APPLETON, WI 54911-6058
PHONE: (920) 380-2800
FAX: (920) 380-2801
www.sehinc.com

PROJECT:
ASISTB0502.00

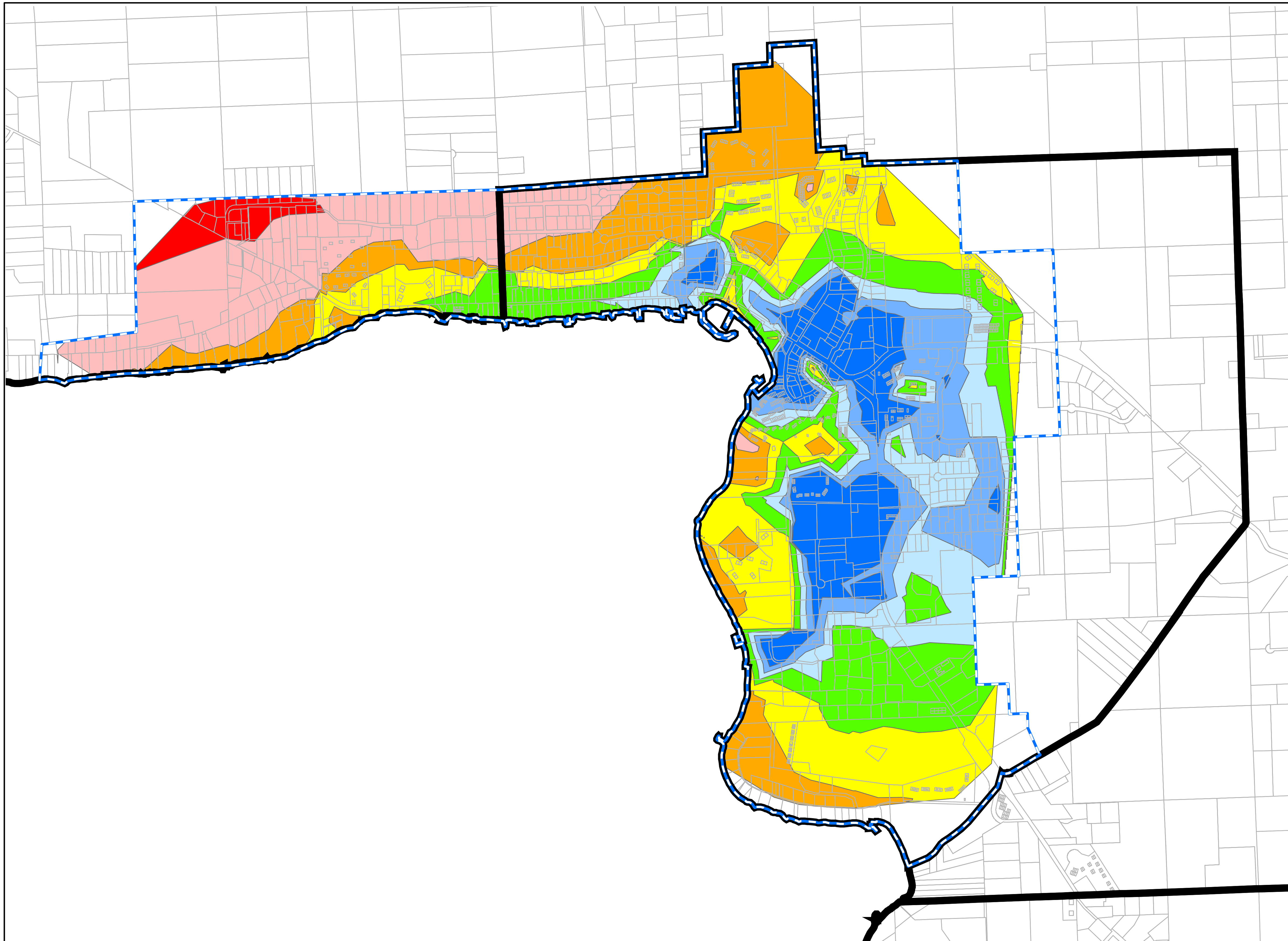
DATE:
10/18/2006

COMPREHENSIVE UTILITIES PLAN
Village of Sister Bay

Maximum Day Available Fire Flow

Figure 7

Map Document: (S:\PT\S\Sistb\050200\GIS\WaterModel\Additional_Scenarios_MXDs\Fig07.mxd)
10/18/2006 -- 12:02:44 PM



Legend

Maximum Day Fire Flow

- 0-500 gpm
- 500-1000 gpm
- 1000-1500 gpm
- 1500-2000 gpm
- 2000-2500 gpm
- 2500-3000 gpm
- 3000-3500 gpm
- 3500+ gpm
- Municipal Boundaries
- Parcels
- Existing Water Service Area

Conditions:
-Proposed Water Tower at Standpipe Site
-Separated Pressure Zones

N

0 750 1,500 Feet

Source:
Door County and Bay Lake RPC.
Projection:
Wisconsin State Plane
Map by:
SEH

SEH
425 WEST WATER STREET
SUITE 300
APPLETON, WI 54911-6058
PHONE: (920) 380-2800
FAX: (920) 380-2801
www.sehinc.com

PROJECT:
ASISTB0502.00

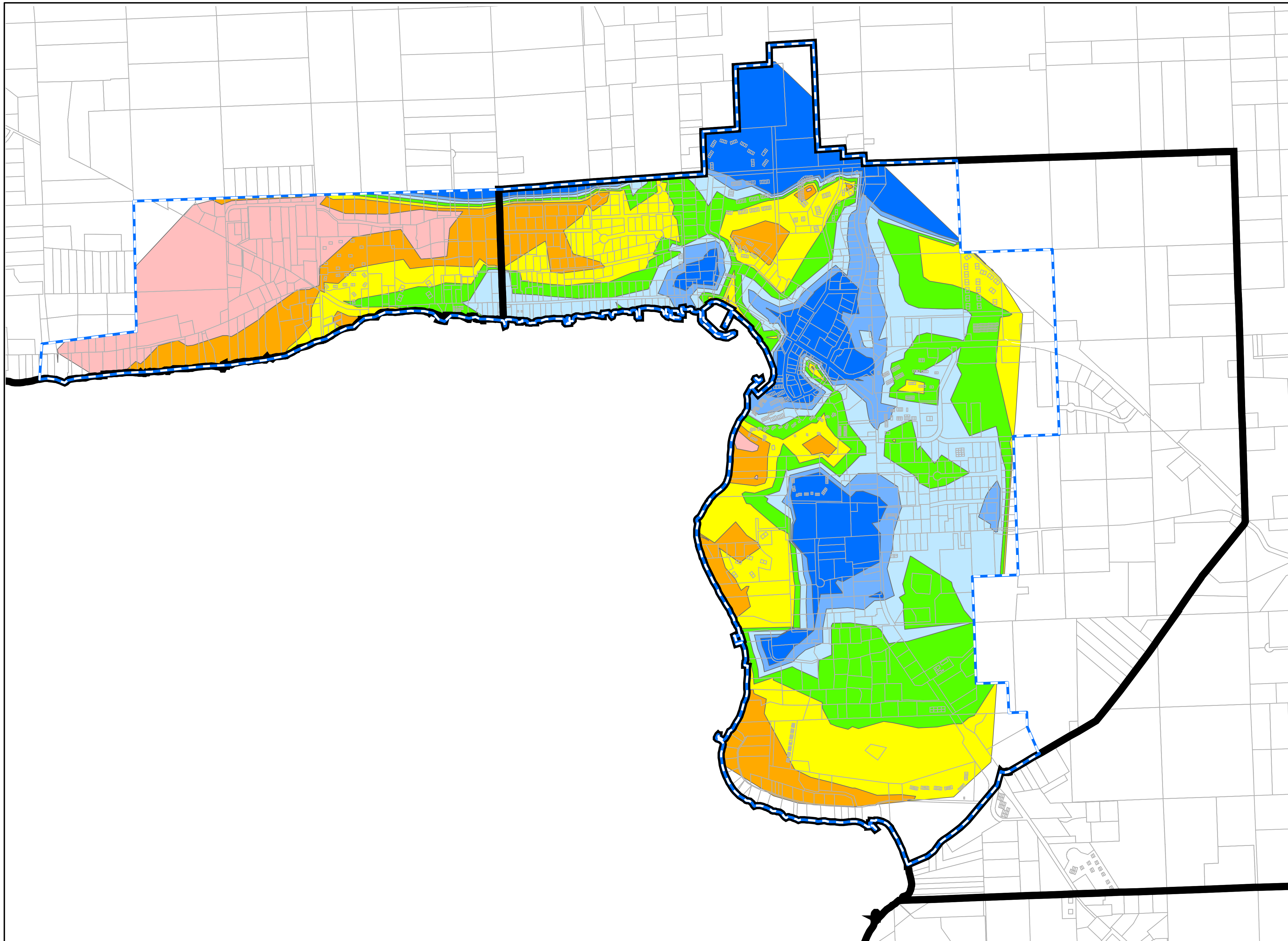
DATE:
10/18/2006

COMPREHENSIVE UTILITIES PLAN
Village of Sister Bay

Maximum Day Available Fire Flow

Figure 8

Map Document: (S:\PT\S\Sistb\050200\GIS\WaterModel\Additional_Scenarios_MXDs\Fig08.mxd)
10/18/2006 -- 12:06:22 PM



Legend

Maximum Day Fire Flow

- 0-500 gpm
- 500-1000 gpm
- 1000-1500 gpm
- 1500-2000 gpm
- 2000-2500 gpm
- 2500-3000 gpm
- 3000-3500 gpm
- 3500+ gpm
- Municipal Boundaries
- Parcels
- Existing Water Service Area

Conditions:
-Proposed Water Tower at WWTP Site
-Separated Pressure Zones

N

0 750 1,500
Feet

Source:
Door County and Bay Lake RPC.
Projection:
Wisconsin State Plane
Map by:
SEH

Map Document: (S:\PT\S\Sistb\050200\GIS\WaterModel\Additional_Scenarios_MXDs\Fig09.mxd)
10/18/2006 -- 12:07:21 PM

SEH
425 WEST WATER STREET
SUITE 300
APPLETON, WI 54911-6058
PHONE: (920) 380-2800
FAX: (920) 380-2801
www.sehinc.com

PROJECT:
ASISTB0502.00

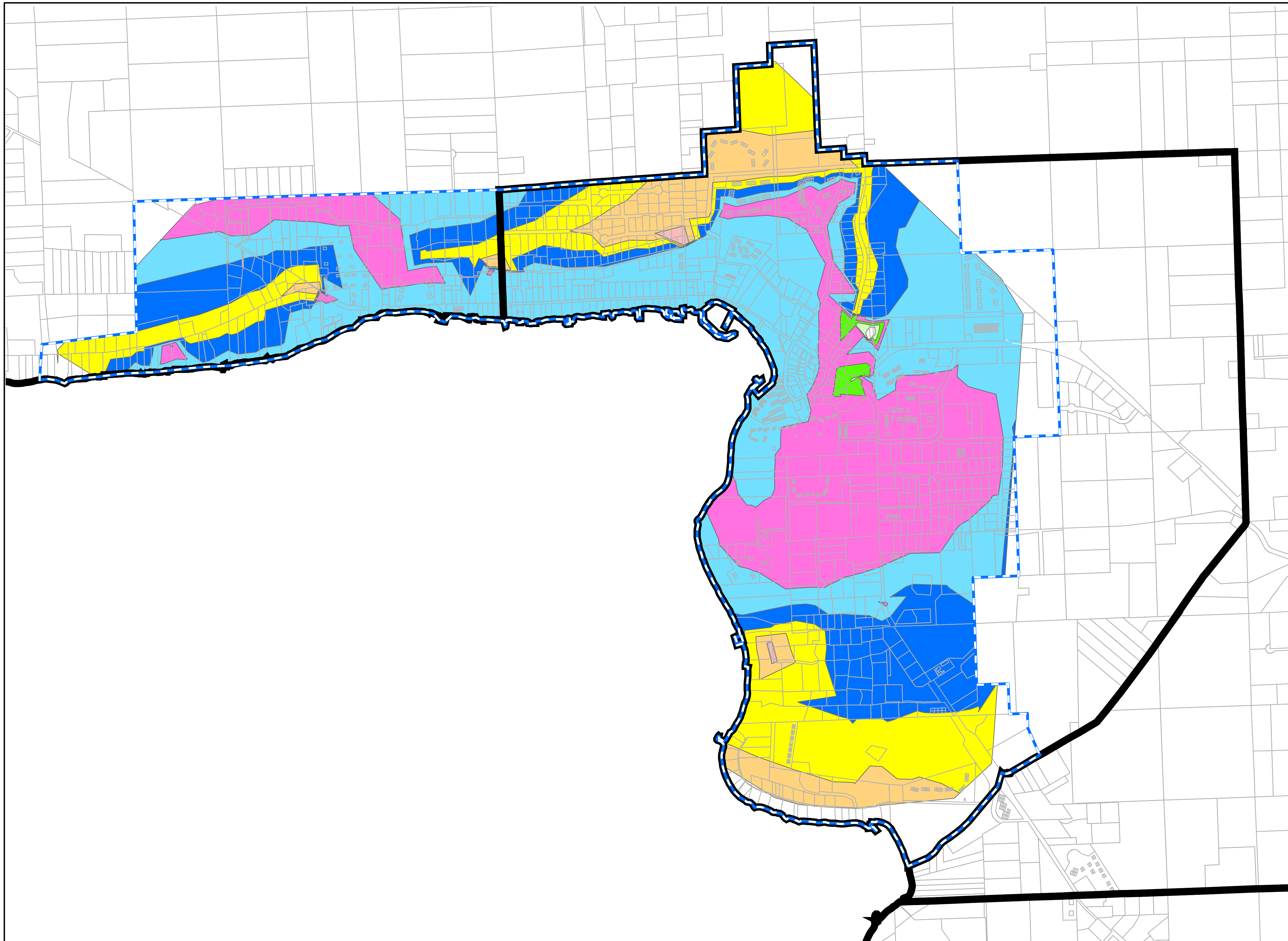
DATE:
10/18/2006

COMPREHENSIVE UTILITIES PLAN

Village of Sister Bay

Maximum Day Available Fire Flow

Figure 9



Legend

Peak Hour Pressure

- 20-30 psi
- 30-40 psi
- 40-50 psi
- 50-60 psi
- 60-70 psi
- 70-80 psi
- 80-90 psi
- 90-100 psi
- Municipal Boundaries
- Parcels
- Existing Water Service Area

Conditions:
-Proposed Water Tower at Northern Site
-Separated Pressure Zones

N

0 750 1,500
Feet

Source:
Door County and Bay Lake RPC.
Projection:
Wisconsin State Plane
Map by:
SEH

SEH
425 WEST WATER STREET
SUITE 300
APPLETON, WI 54911-6058
PHONE: (920) 380-2800
FAX: (920) 380-2801
www.sehinc.com

PROJECT:
ASISTB0502.00

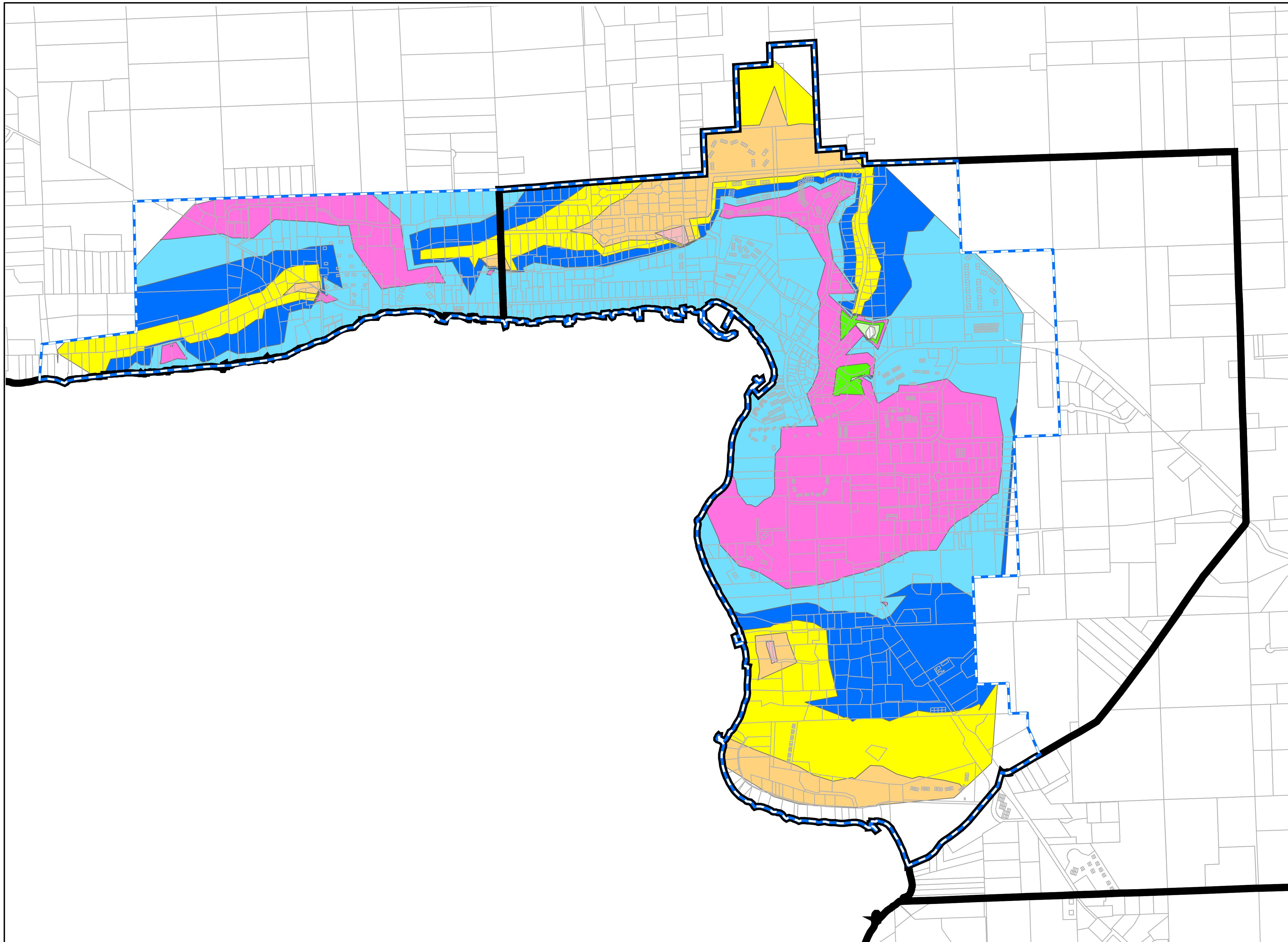
DATE:
10/18/2006

COMPREHENSIVE UTILITIES PLAN
Village of Sister Bay

Peak Hour Pressure

Figure 10

Map Document: (S:\PT\S\Sistb\050200\GIS\WaterModel\Additional_Scenarios_MXDs\Fig10.mxd)
10/17/2006 -- 5:33:36 PM



Legend

Peak Hour Pressure

- 20-30 psi
- 30-40 psi
- 40-50 psi
- 50-60 psi
- 60-70 psi
- 70-80 psi
- 80-90 psi
- 90-100 psi
- Municipal Boundaries
- Parcels
- Existing Water Service Area

Conditions:
-Proposed Water Tower at Standpipe Site
-Separated Pressure Zones

N

0 750 1,500
Feet

Source:
Door County and Bay Lake RPC.
Projection:
Wisconsin State Plane
Map by:
SEH

SEH
425 WEST WATER STREET
SUITE 300
APPLETON, WI 54911-6058
PHONE: (920) 380-2800
FAX: (920) 380-2801
www.sehinc.com

PROJECT:
ASISTB0502.00

DATE:
10/18/2006

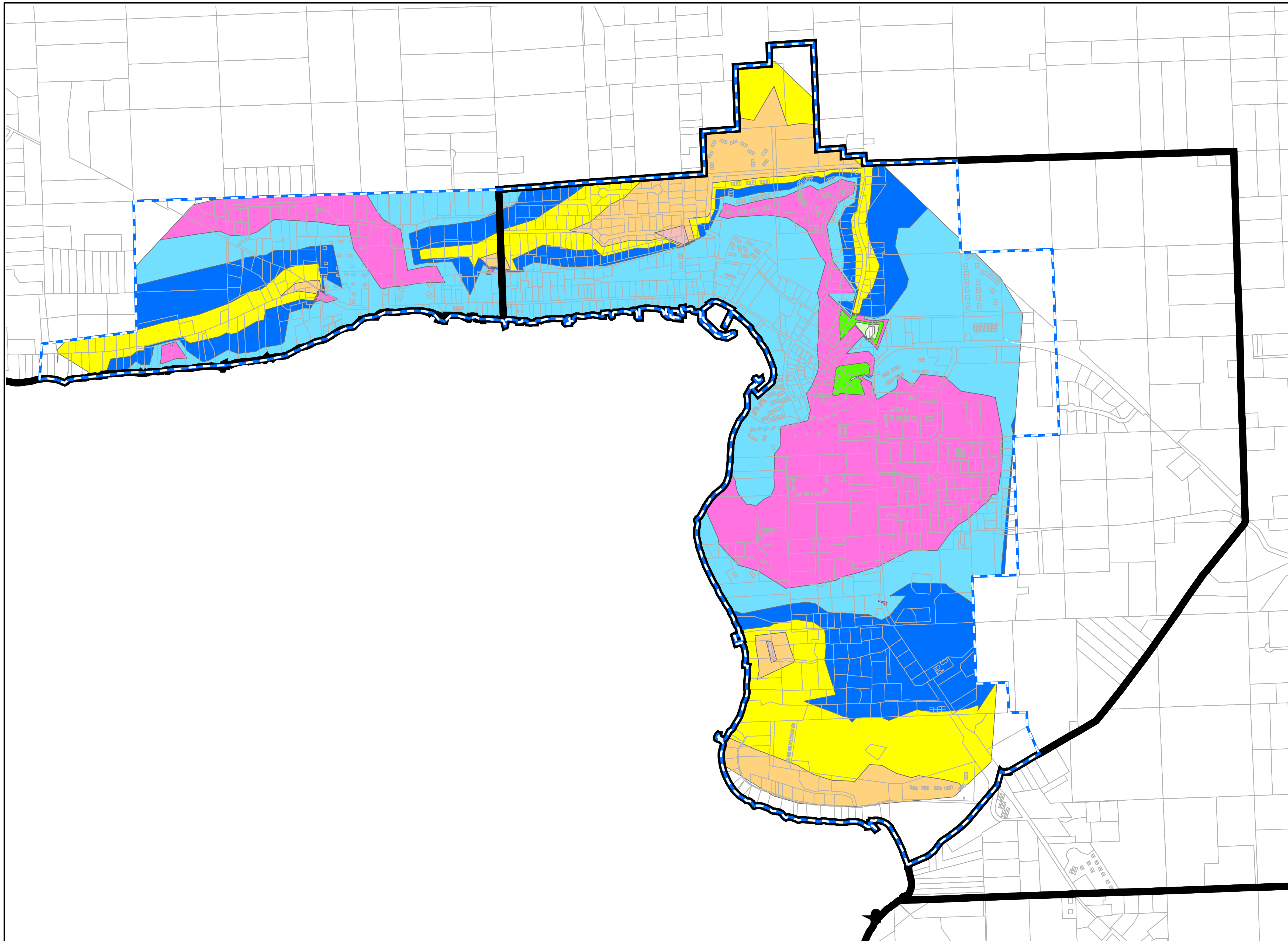
COMPREHENSIVE UTILITIES PLAN

Village of Sister Bay

Peak Hour Pressure

Figure 11

Map Document: (S:\PT\S\Sistb\050200\GIS\WaterModel\Additional_Scenarios_MXDs\Fig11.mxd)
10/17/2006 -- 5:36:40 PM



Legend

Peak Hour Pressure

- 20-30 psi
- 30-40 psi
- 40-50 psi
- 50-60 psi
- 60-70 psi
- 70-80 psi
- 80-90 psi
- 90-100 psi
- Municipal Boundaries
- Parcels
- Existing Water Service Area

Conditions:
-Proposed Water Tower at WWTP Site
-Separated Pressure Zones

N

0 750 1,500 Feet

Source:
Door County and Bay Lake RPC.
Projection:
Wisconsin State Plane
Map by:
SEH

SEH
425 WEST WATER STREET
SUITE 300
APPLETON, WI 54911-6058
PHONE: (920) 380-2800
FAX: (920) 380-2801
www.sehinc.com

PROJECT: ASISTB0502.00

DATE: 10/18/2006

COMPREHENSIVE UTILITIES PLAN
Village of Sister Bay

Peak Hour Pressure

Figure 12

Map Document: (S:\PT\S\Sistb\050200\GIS\WaterModel\Additional_Scenarios_MXDs\Fig12.mxd)
10/17/2006 -- 5:39:50 PM

APPENDIX F

MEMORANDUMS ON ALTERNATIVES FOR SERVING REGION H WITH SANITARY SEWER

MEMORANDUM

TO: Bob Kufrin, Village Administrator

FROM: David F. Simons, P.E., SEH

DATE: November 30, 2006

RE: Supplemental Analysis requested at Nov. 6, 2006 CUP Workshop on Sanitary Sewer
SEH No. ASISTB0502

This memorandum documents the results of the supplemental analysis which was requested at the November 6, 2006 CUP Workshop on Sanitary Sewer. Supplemental analysis was requested on 2 items:

1. What is the Maximum Size Temporary Lift Station Which Could be Installed in the NE part of Region H and Routed Through the Existing System Without Surcharging the Existing Piping?

Based on the model assumptions, it is projected that the existing system could accommodate a small temporary lift station located in the NE part of Region H as an interim way to provide service in this area. This would only be a temporary solution until such time as Lift Station H and the associated downstream improvements were constructed.

If the diversion were constructed from MH 39 to MH 193, it is estimated that a temporary lift station with duel 100 gpm pumps could be constructed somewhere in the NE part of Region H. The maximum pumping rate of the station should not exceed 150 gpm with both pumps operating simultaneously. The flow could be temporarily pumped to MH 317, which is located on the south end of Smith Drive. For comparison, this is the approximate size of the Fieldcrest Lift Station.

If the diversion were not constructed, there would not be capacity in the Mill Road sewer to accommodate this additional flow, so the diversion must be constructed first.

In addition, as part of the design on this temporary lift station, a detailed feasibility study would be required to confirm that the downstream system is adequate to accept the flows resulting from the specific pumps and pumping conditions being proposed for that lift station.

Finally, if the station were to be constructed, it is recommended that flow monitors be periodically installed in the downstream system to monitor the flow levels after the station comes on line. This would allow the Village to observe the flow levels and take appropriate action if the levels become too high. The monitors should be installed on the critical segments of both sides of the diversion (i.e., the Maple/Mill Road side and the Claflin/South Spring side). Lift Station 1 should also be monitored for any capacity issues.

2. What is the Feasibility and Approximate Cost for the Following Alternative Alignment: Route Future Flows From Region H Along Bay Shore Drive to Lift Station 1 Instead of Piping them Directly to the Treatment Plant Via a Future Trunk Sewer Constructed Around the East Side of the Village?

This alternative would involve running the future force main from Lift Station H north along Hwy 57 to the crest of the hill north of Country Walk Drive. At the crest of the hill, the force main could discharge into a new gravity pipe that could run down Hwy 57 and Bay Shore Drive to Lift Station 1. Flows at Maple Drive and Mill Road could be picked up by this new pipe. In addition, the existing sewer pipe located through the back yards east of Bay Shore Drive could be abandoned (but would not necessarily need to be). Lift Station 1 would need to be upgraded, but the existing force mains pumping to the WWTP appear to have sufficient capacity to accommodate the additional flow.

Under this alternative, the force main from Lift Station H to the crest of the hill would be an 8 inch diameter pipe. The downstream trunk sewer to Lift Station 1 would steadily increase in size as it proceeds downstream, and would range from approximately a 10" pipe to an 18" pipe, depending on the location along the route and the number of connections to the existing system which are made.

Lift Station 1 would need to be upgraded by enlarging the pumps. More study would be needed to determine the exact improvements needed at the station, but at this time it is estimated that the total pumping capacity at the station would need to increase by approximately 50%. As the service area fills in and the existing system expands in other areas, Lift Station 1 may need to be expanded even further in the future.

The net cost difference between this alternative and routing pipes around the east side of the Village is listed in the table below. Although this alternative is more expensive than routing pipes around the east side of the Village (due to the additional length of pipe), there could be other advantages to the Village of considering this alternative:

- This alternative may allow more development to occur faster in the southern part of the service area, because it would not be dependent on development in Regions I and J.
- It would provide additional capacity in the downtown area.
- It would provide a level of redundancy that currently does not exist in the downtown area. This would allow some of the existing mains to be taken out of service for maintenance if needed, while still providing service to customers.
- If the project is done concurrently with the State's resurfacing project, there may be cost sharing opportunities with the State on the surface restoration.

Detailed feasibility studies should be conducted to confirm the preliminary findings before any improvements are designed or constructed.

Preliminary Opinion of Cost
Comparison Between Options

Improvement	Route Around East Side Village	Route Down Bay Shore Drive
FM. H	$1,368,900	$1,095,200
P147	$860,100	$793,000
LS. J	$301,100	$249,600
FM. J	$418,000	$400,200
New Trunk to LS 1	N/A	$775,200
Upgrade of LS 1	N/A	$218,400
Totals	$2,948,100	$3,531,600

SEH MEMORANDUM

TO: Bob Kufrin, Village Administrator

FROM: David F. Simons, P.E., SEH

DATE: December 27, 2006

RE: Supplemental Sanitary Sewer Analysis requested at December 4, 2006 CUPAC Meeting
SEH No. ASISTB0502

This memorandum documents the results of the supplemental sanitary sewer analysis which was requested at the December 4, 2006 CUPAC Meeting. Supplemental analysis was requested on the following items:

1. What would be the approximate service boundary of a temporary sanitary sewer lift station which could be located in the northeast part of Region H and routed through the existing sewer system without surcharging the existing piping?

As discussed in our November 30, 2006 memorandum, it is projected that the existing system could accommodate a small temporary lift station located in the northeast part of Region H as an interim way to provide sanitary sewer service in this area. This would only be a temporary solution until such time as the recommended Lift Station H and the associated downstream improvements were constructed.

If the recommended sewer diversion is constructed from MH 39 to MH 193, it is estimated that a temporary lift station with dual 100 gallon per minute (gpm) pumps could be constructed to serve the northeast part of Region H. If the diversion is not constructed, there would not be capacity in the Mill Road sanitary sewer to accommodate this additional flow, so the diversion must be constructed first. The maximum pumping rate of the temporary station should not exceed 150 gpm, with both pumps operating simultaneously. The flow could be temporarily pumped to MH 317, which is located on the south end of Smith Drive. For comparison, this is the approximate size of the Fieldcrest Lift Station.

Using the flow rates assumed in the CUP draft report, the maximum pumping rate of 150 gpm translates into approximately 150 single family homes. Although the minimum lot size for R-1 is 20,000 square feet (approx. 0.5 acre), the amount of land typically attributed to each new lot is significantly more if roadways, ponding areas, and open spaces are also included. For example, depending on the development layout, it would not be uncommon for each lot to consume 1 acre or more of land. Under this scenario, 150 residential units might require 150 acres of land.

Exhibits A and B show two different options for serving the northeast part of Region H. The approximate sewer service boundary for each option is shown based on the existing topography. Areas beyond the limits shown would be difficult to serve without additional lift stations.

Option A shown in Exhibit A illustrates providing service to only the undeveloped land north of Hwy 57. The service area represents approximately 45 acres. The temporary lift station is located northeast of Northwoods Drive in the lowest part of the service area to save costs on sewer pipe depth. The force main would pump northeast along Hwy 57 to MH 317.

Short Elliott Hendrickson Inc., 156 High Street, Suite 300, New Richmond, WI 54017-1128
SEH is an equal opportunity employer | **www.sehinc.com** | 715.246.9906 | 888.881.4281 | 715.246.9413 fax

Option B shown in Exhibit B illustrates serving areas both north and south of Hwy 57. The Option B service area represents about 165 acres. The temporary lift station is located along Orchard Drive in one of the lowest parts of the service area to reduce pipe depth and the associated cost. The force main would pump north along Orchard Drive to MH 317.

2. What is the estimated cost for a temporary lift station?

For the purposes of this cost estimate, it has been assumed that the size of the lift station will be the same for both Option A and Option B. A cheaper grinder pump system was considered for Option A due to the smaller design flows, but it was determined that the flows could exceed the recommended threshold for a grinder pump. In addition, using a 100 gpm duplex station for each option would provide the Village with the most flexibility for providing sanitary sewer service to this developing area.

Planning level costs are summarized below for each option. The force main cost is based on an assumed 6 inch diameter force main, and contains a significant allowance for rock excavation, restoration and contingencies. Detailed feasibility studies should be conducted to confirm the preliminary findings before any improvements are designed or constructed.

Planning Level Cost Estimate for Temporary Lift Station

Improvement	Option A	Option B
Lift Station	$135,000	$135,000
Force Main	$370,500	$530,400
Totals	**$506,000**	**$666,000**

dfs

EXHIBIT A

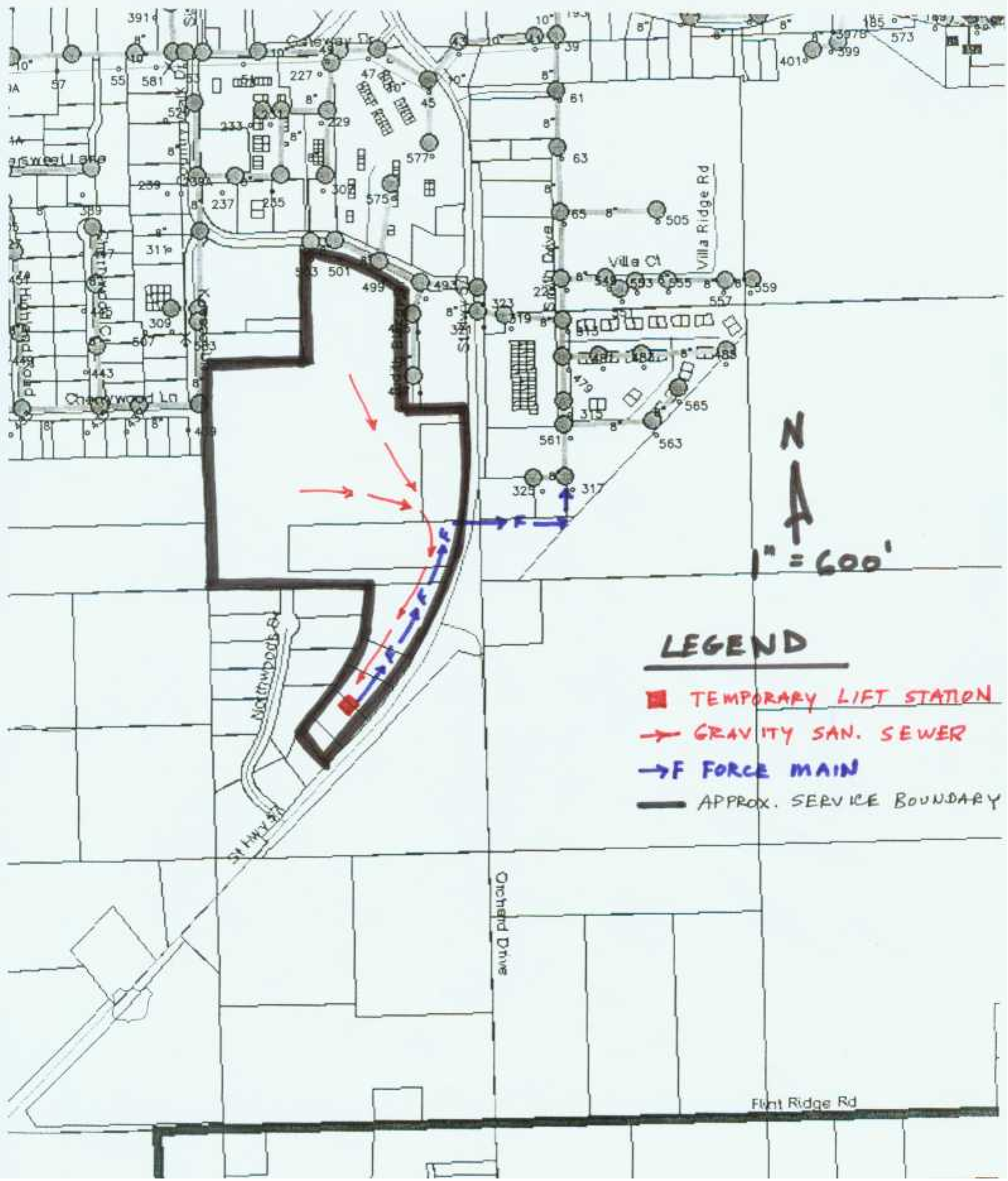
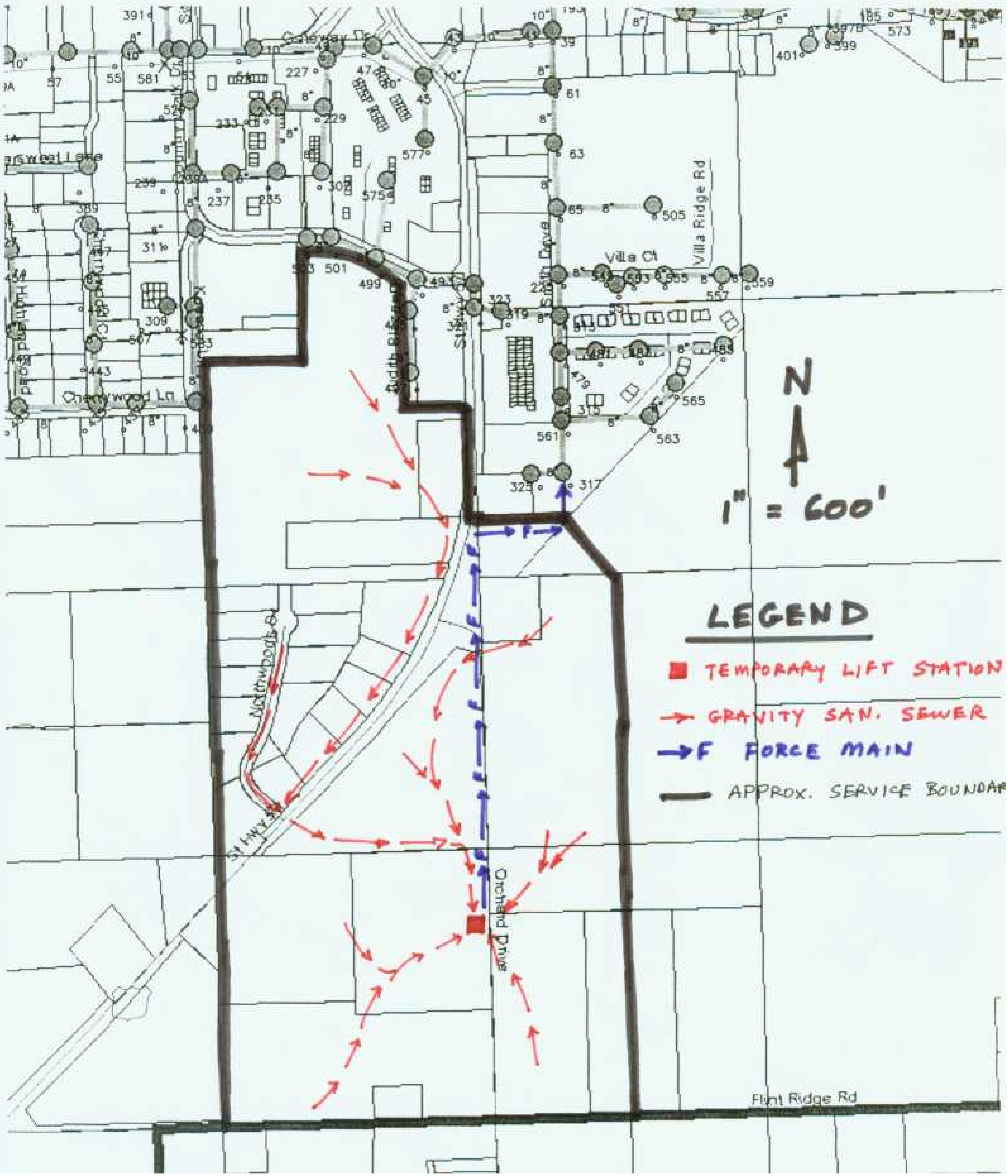

N
1" = 600'
LEGEND
TEMPORARY LIFT STATION
GRAVITY SAN. SEWER
F FORCE MAIN
APPROX. SERVICE BOUNDARY
Villa Ridge Rd
Villa Ct
Orchard Drive
Flint Ridge Rd
St Hwy
Cherrywood Ln

EXHIBIT B



N
1" = 600'
LEGEND
TEMPORARY LIFT STATION
GRAVITY SAN. SEWER
F FORCE MAIN
APPROX. SERVICE BOUNDAR
Villa Ridge Rd
Villa Ct
Orchard Drive
Flint Ridge Rd
St Hwy 30
Northwoods Ct
Cherrywood Ln

SEH MEMORANDUM

TO: Bob Kufrin, Village Administrator

FROM: David F. Simons, P.E., SEH

DATE: December 5, 2007

RE: December 10, 2007 CUPAC meeting
SEH No. ASISTB0502

In preparation for the December 10th CUPAC meeting, I have provided a few thoughts related to the following questions that you asked:

"New survey information has shown that there would be no additional developable land freed up east of Bay Shore Drive and north of Mill Road as a result of the relocation of the sewer and water from behind the businesses to Bay Shore Drive. If there is no economic value to relocating these utilities, what are the Village's options?"

Gaining additional developable land behind these businesses was not the original reason for considering the relocations. The primary reason for considering the sewer relocation was to provide additional capacity for development growth to the south, and to take advantage of the State resurfacing project by getting the sewer installed while there is an opportunity. Since the existing sewer pipes are too small to handle new development to the south, the other option is to build a new trunk main from the future southern expansion area to the northeast, all the way around the existing system and directly to the WWTP. This option would involve waiting to install sewer until downstream areas become developed, or building downstream utilities prematurely across undeveloped land, neither of which are ideal situations.

The alternative option (referred to below as the Alternative Concept) would involve running the future force main from Lift Station H north along Hwy 57 to the crest of the hill north of Country Walk Drive. At the crest of the hill, the force main could discharge into a new gravity pipe that could run down Hwy 57 and Bay Shore Drive to Lift Station 1. Flows at Maple Drive and Mill Road could be picked up by this new pipe. In addition, the existing sewer pipe located through the back yards east of Bay Shore Drive could be abandoned (but would not necessarily need to be). Lift Station 1 would need to be upgraded, but the existing force mains pumping to the WWTP appear to have sufficient capacity to accommodate the additional flow.

The net cost difference between the options is described in our November 30, 2006 memorandum to the Village. Although the Alternative Concept is slightly more expensive than the option shown in the original report (due to the additional length of pipe), there are other advantages to the Village of considering this alternative:

- This alternative may allow more development to occur faster in the southern part of the service area, because it would not be dependent on development in Regions I and J.
- It would provide additional capacity in the downtown area.
- It would provide a level of redundancy that currently does not exist in the downtown area. This would allow some of the existing mains to be taken out of service for maintenance if needed, while still providing service to customers.

- If the project is done concurrently with the State's resurfacing project, there may be cost sharing opportunities with the State on the surface restoration.

In response to the question listed above, we believe that the relocation of the sewer line to Bay Shore Drive is still an attractive option for the Village, regardless of whether the relocation frees up any developable land behind the businesses.

"What should the Village consider as next steps with regard to the sanitary sewer relocation issue?"

We suggest that the Village consider the following steps:

1. Meet with the State to discuss the possible inclusion of the utility relocation work in the State resurfacing project. Discuss the timing, cost sharing responsibilities, and other parameters.

2. Televise the existing sanitary sewer main to determine its current condition and remaining life expectancy. If the pipe has less than 20 years of remaining life expectancy, the relocation as part of the resurfacing project should be strongly considered.

3. Determine the expected timing and extent of future development within Region H (to the best of the Village's ability). Timing and extent of development in this area will drive the decision making process.

4. If items 1 – 3 above appear to point toward relocation being the best option, then the Village should authorize that a preliminary plan and cost estimate be prepared for the relocation work. This will be needed in order to continue discussions with the State and to allow the Village to properly budget for the improvements.

5. If additional development is expected to occur south and west of the relocation area prior to the state resurfacing project, then the Village should consider constructing the diversion between MH 39 and MH 193 as soon as possible (on Maple Dr. at the cemetery). This would provide interim sewer capacity for up to an additional 150 homes prior to and during the state resurfacing project.

6. If the Village decides to relocate the sewer, but if no additional development is expected to occur south and west of the relocation area prior to the state resurfacing project, then the diversion between MH 39 and MH 193 is not necessary because the relocated sewer will provide additional capacity.

7. If development pressure occurs in the northeast part of Region H (near Northwoods Drive), the Village should consider an interim lift station and force main as described in the CUP.

dfs

MEMORANDUM

TO: Bob Kufrin, Village Administrator

FROM: David F. Simons, P.E., SEH

DATE: March 6, 2008

RE: March 17, 2008 CUPAC meeting
SEH No. ASISTB0502

In preparation for the March 17th CUPAC meeting, we are submitting cost estimates and sketches for the 3 sanitary sewer routing options that were discussed at the December 10, 2007 CUPAC meeting. A summary of the initial costs for each option is shown below.

Option	Initial Cost
Option 1 – STH 57 and North Shore Drive	$585,000
Option 2 – Maple to Mill Between Lots Then Down Spring	$759,000
Option 3 – Maple to Mill Between Lots Then Behind Businesses	$699,000

Option 1 has the lowest initial cost because the pavement removal and replacement cost on Bay Shore Drive would be paid for by the State as part of their resurfacing project. In addition to evaluating the initial cost for each option, we also evaluated the future cost of each option over the next 20 years. This was done so that the 3 options could be compared on an equal basis, by comparing the total cost to the Village over time.

Option 1 and Option 2 have a high future maintenance cost because they do not replace the old 12” concrete line behind the businesses. Therefore, additional costs will be expended in the future under Options 1 and 2 in order to maintain and eventually replace the 12” line.

The future cost of the old 12” concrete pipe was assumed to include regular cleaning, regular minor repairs (which will get more frequent and costly as the pipe ages), and eventual replacement of the pipe in 20 years. The future maintenance costs on the new pipes were assumed to include only periodic cleaning and occasional minor repairs. The old pipe on Spring Road is made of PVC, and has a longer life expectancy than the 12” concrete pipe.

When the future costs are added to the initial costs for each option, the resulting total present worth costs over a 20 year period change significantly.

Option	Total Present Worth Cost
Option 1 – STH 57 and North Shore Drive	$716,000
Option 2 – Maple to Mill Between Lots Then Down Spring	$891,000
Option 3 – Maple to Mill Between Lots Then Behind Businesses	$714,000

Another factor to consider is the inconvenience of the various routes. Option 1 would likely result in the highest level of inconvenience to the community, followed by Option 2 and then by Option 3.

I will be available at the March 17th CUPAC meeting to review these options with the Commission.

p:\pt\s\sistb\050200\final report\appendix f\appendix f item 5.doc

OPINION OF COSTS - OPTION 1

STH 57 and North Bay Shore Drive to Main Lift Station

Item No.	Item	Unit	Est. Quantity	Unit Price	Total Price
SCHEDULE 1 - SANITARY SEWER					
1	12-INCH SANITARY SEWER	LF	900	$28	$25,200
2	15-INCH SANITARY SEWER	LF	900	$32	$28,800
3	18- INCH SANITARY SEWER	LF	1800	$36	$64,800
4	4- INCH SAN SERVICE	LF	700	$4	$2,450
5	48-INCH MANHOLE W/ CASTING	EA	12	$2,400	$28,800
6	CONNECT TO EXISTING MANHOLE	EA	2	$1,000	$2,000
7	CONNECT TO EXISTING PIPE	EA	8	$500	$4,000
8	CRUSHED ROCK PIPE FOUNDATION	TON	200	$30	$6,000
9	EXCESS MANHOLE DEPTH	LF	30	$100	$3,000
10	PAVEMENT RESTORATION	SY	2700	$26	$70,200
11	RECONNECT SERVICES	EA	14	$125	$1,750
12	REMOVE ASPHALTIC SURFACE	SF	25000	$0	$7,500
13	REMOVE EXISTING STRUCTURES	EA	9	$300	$2,700
14	REMOVE SAN SEWER	LF	1600	$4	$6,400
15	ROCK EXCAVATION	CY	300	$50	$15,000
16	TELEVISED SEWER INSPECTION	LF	3500	$1	$3,500
17	TEMP SEWER SERVICE	LS	1	$7,500	$7,500
18	WYES	EA	14	$125	$1,750
		SCHEDULE 1 - SANITARY SEWER TOTAL			**$281,350**
SCHEDULE 2 - WATER MAIN					
1	1-INCH COPPER SERVICE	LF	800	$14	$11,200
2	1-INCH CORPORATION	EA	14	$110	$1,540
3	1-INCH CURB STOP & BOX	EA	14	$110	$1,540
4	6-INCH DIP WATER MAIN	LF	300	$32	$9,600
5	6-INCH GATE VALVE & BOX	EA	9	$800	$7,200
6	8-INCH DIP WATER MAIN	LF	3600	$22	$79,200
7	8-INCH GATE VALVE & BOX	EA	6	$1,100	$6,600
8	CONNECT TO EXISTING WATERMAIN	EA	12	$250	$3,000
9	DIP FITTINGS	LB	1300	$4	$4,550
10	HYDRANT	EA	9	$2,200	$19,800
11	RECONNECT SERVICES	EA	14	$125	$1,750
12	REMOVE VALVES	EA	10	$200	$2,000
13	REMOVE WATERMAIN	LF	2000	$3	$6,000
14	TEMP WATER SERVICE	LS	1	$5,000	$5,000
15	TRAFFIC CONTROL	LS	1	$2,500	$2,500
		SCHEDULE 2 - WATER TOTAL			**$161,480**
		SUBTOTAL OPTION 1			**$442,800**
		10% CONTINGENCY			**$44,300**
		20% ENGINEERING			**$97,400**
		OPTION 1 TOTAL OPINION OF COST			**$584,500**

P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay study\March-April 2008 Report Revisions\Appendix\F\[Appe

OPINION OF COSTS - OPTION 2

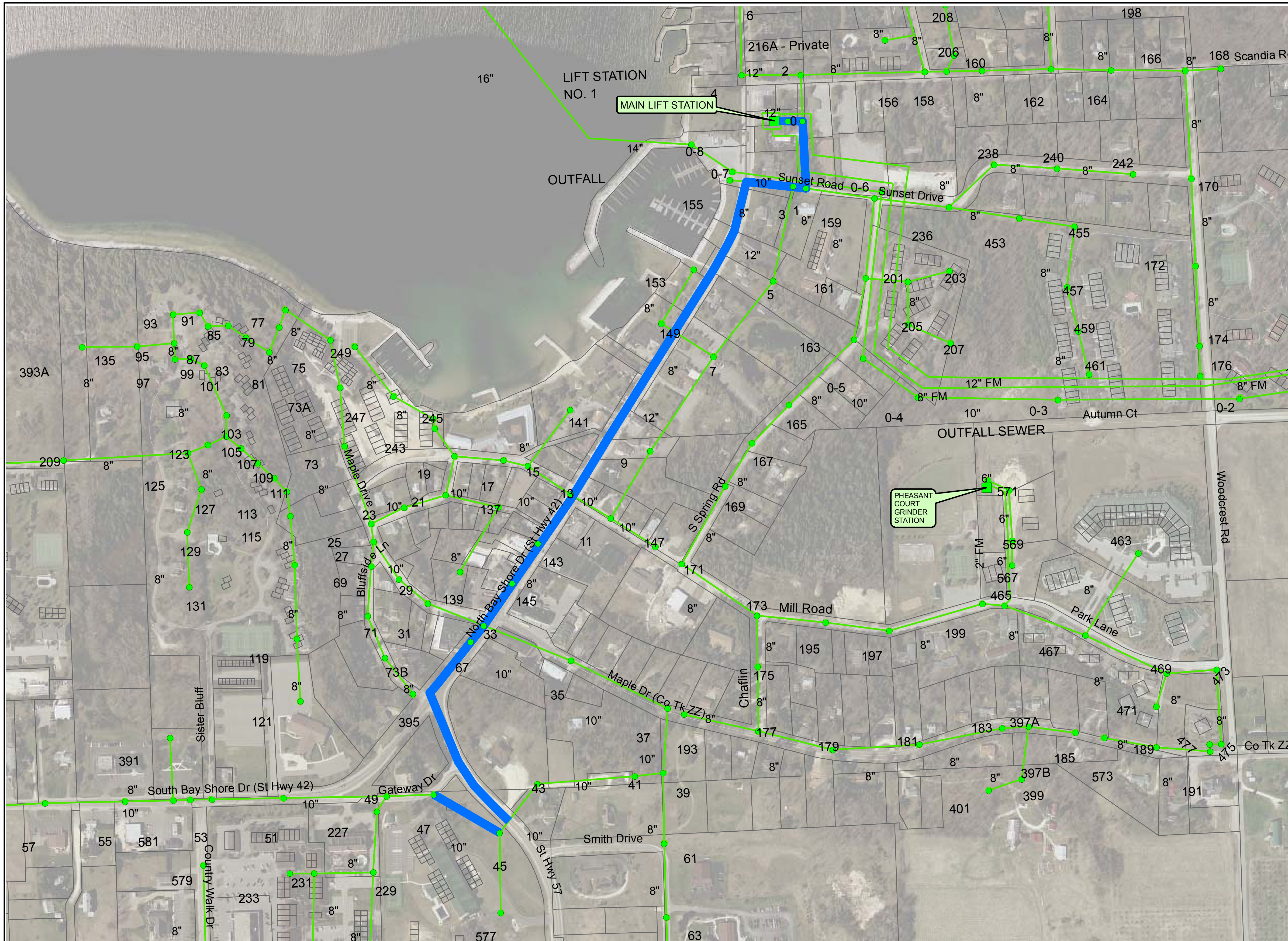
Maple to Mill Between Lots Then Down Spring to Main LS

Item No.	Item	Unit	Est. Quantity	Unit Price	Total Price
SCHEDULE 1 - SANITARY SEWER					
1	12-INCH SANITARY SEWER	LF	800	$28	$22,400
2	15-INCH SANITARY SEWER	LF	800	$32	$25,600
3	18-INCH SANITARY SEWER	LF	2300	$36	$82,800
4	48-INCH MANHOLE W/ CASTING	EA	12	$2,400	$28,800
5	4-INCH SERVICE	LF	350	$15	$5,250
6	CONNECT TO EXISTING MANHOLE	EA	2	$1,000	$2,000
7	CONNECT TO EXISTING SANITARY PIPE	EA	7	$500	$3,500
8	CRUSHED ROCK PIPE FOUNDATION	TON	200	$30	$6,000
9	EXCESS MANHOLE DEPTH	LF	25	$100	$2,500
10	PAVEMENT RESTORATION	SY	4000	$26	$104,000
11	RECONNECT SERVICES	EA	12	$125	$1,500
12	REMOVE ASPHALTIC PAVEMENT	SF	52000	$0	$15,600
13	REMOVE EXISTING SAN SEWER	LF	3000	$4	$12,000
14	REMOVE EXISTING STRUCTURES	EA	16	$300	$4,800
15	ROCK EXCAVATION	CY	300	$50	$15,000
16	TELEVISED SEWER INSPECTION	LF	3900	$1	$3,900
17	TEMP SEWER SERVICE	LS	1	$15,000	$15,000
18	TURF ESTABLISHMENT	SY	8000	$1	$11,200
19	WYES	EA	12	$125	$1,500
				SCHEDULE 1 - SANITARY SEWER TOTAL	**$363,350**
SCHEDULE 2 - WATER MAIN					
1	1-INCH COPPER SERVICE	LF	800	$14.00	$11,200.00
2	1-INCH CORPORATION	EA	12	$110	$1,320
3	1-INCH CURB STOP AND BOX	EA	12	$110	$1,320
4	6-INCH DIP WATER MAIN	LF	300	$32	$9,600
5	6-INCH GATE VALVE AND BOX	EA	9	$800	$7,200
6	8-INCH DIP WATERMAIN	LF	3700	$32	$118,400
7	8-INCH GATE VALVE AND BOX	EA	6	$1,100	$6,600
8	CONNECT TO EXISTING WATERMAIN	EA	9	$250	$2,250
9	DIP FITTINGS	LB	1500	$4	$5,250
10	HYDRANT	EA	9	$2,200	$19,800
11	REMOVE GATE VALVE & BOX	EA	8	$75	$600
12	REMOVE WATER MAIN	LF	3000	$3	$9,000
13	RECONNECT SERVICES	EA	12	$125	$1,500
14	TEMP WATER SERVICE	LS	1	$10,000	$10,000
15	TRAFFIC CONTROL	LS	1	$8,000	$8,000
				SCHEDULE 2 - WATER TOTAL	**$212,040**
				SUBTOTAL OPTION 2	**$575,400**
				10% CONTINGENCY	**$57,500**
				20% ENGINEERING	**$126,600**
				OPTION 2 TOTAL OPINION OF COST	**$759,500**

OPINION OF COSTS - OPTION 3

Maple to Mill Between Lots Then Down Behind the Businesses to Main LS

Item No.	Item	Unit	Est. Quantity	Unit Price	Total Price
SCHEDULE 1 - SANITARY SEWER					
1	12-INCH SANITARY SEWER	LF	1000	$28	$28,000
2	15-INCH SANITARY SEWER	LF	1000	$32	$32,000
3	18-INCH SANITARY SEWER	LF	1700	$36	$61,200
4	48-INCH MANHOLE W/ CASTING	EA	14	$2,400	$33,600
5	4-INCH PVC SERVICE PIPE	LF	600	$15	$9,000
6	CONNECT TO EXISTING MANHOLE	EA	2	$1,000	$2,000
7	CONNECT TO EXISTING SANITARY PIPE	EA	7	$500	$3,500
8	CRUSHED ROCK PIPE FOUNDATION	TON	200	$30	$6,000
9	EXCESS MANHOLE DEPTH	LF	25	$100	$2,500
10	PAVEMENT RESTORATION	SY	2700	$26	$70,200
11	RECONNECT SERVICES	EA	12	$125	$1,500
12	REMOVE ASPHALTIC SURFACE	SF	30000	$0	$9,000
13	REMOVE SANITARY SEWER PIPE	LF	3000	$4	$12,000
14	REMOVE STRUCTURES	EA	15	$250	$3,750
15	ROCK EXCAVATION	CY	250	$50	$12,500
16	TELEVISED SEWER INSPECTION	LF	3700	$1	$3,700
17	TEMP SEWER SERVICE	LS	1	$15,000	$15,000
18	TURF ESTABLISHMENT	SY	10000	$1	$14,000
19	WYES	EA	12	$125	$1,500
		SCHEDULE 1 - SANITARY SEWER TOTAL			**$320,950**
SCHEDULE 2 - WATER MAIN					
1	1-INCH COPPER SERVICE	LF	800	$14	$11,200
2	1-INCH CORPORATION	EA	12	$110	$1,320
3	1-INCH CURB STOP AND BOX	EA	12	$110	$1,320
4	6-INCH DIP WATER MAIN	LF	300	$32	$9,600
5	6-INCH GATE VALVE AND BOX	EA	9	$800	$7,200
6	8-INCH DIP WATERMAIN	LF	3500	$32	$112,000
7	8-INCH VALVE AND BOX	EA	12	$1,100	$13,200
8	CONNECT TO EXISTING WATERMAIN	EA	12	$250	$3,000
9	DIP FITTINGS	LB	1300	$4	$4,550
10	HYDRANT	EA	9	$2,200	$19,800
11	RECONNECT SERVICES	EA	12	$125	$1,500
12	REMOVE VALVES	EA	15	$200	$3,000
13	REMOVING WATERMAIN	LF	3000	$3	$9,000
14	TEMP WATER SERVICE	LS	1	$10,000	$10,000
15	TRAFFIC CONTROL	LS	1	$2,500	$2,500
		SCHEDULE 2 - WATER TOTAL			**$209,190**
		SUBTOTAL OPTION 3			**$530,100**
		10% CONTINGENCY			**$53,000**
		20% ENGINEERING			**$116,600**
	P:\PT\S\SISTB\050200_UTILITIES\Project\Sister Bay study\March-April 2008 Report Revisions\Appendix\F\[Appe	**OPTION 3 TOTAL OPINION OF COST**			**$699,700**



Legend

- Proposed Sewer System Improvements - Option 1
- Existing Sewer System
- Existing Lift Stations
- Existing Manhole

N

0 400 800 Feet

Source:
Door County and Bay Lake RPC.
Projection:
Wisconsin State Plane
Map by:
SEH

SEH
425 WEST WATER STREET
SUITE 300
APPLETON, WI 54911-6058
PHONE: (920) 380-2800
FAX: (920) 380-2801
www.sehinc.com

PROJECT:
ASISTB0502.00

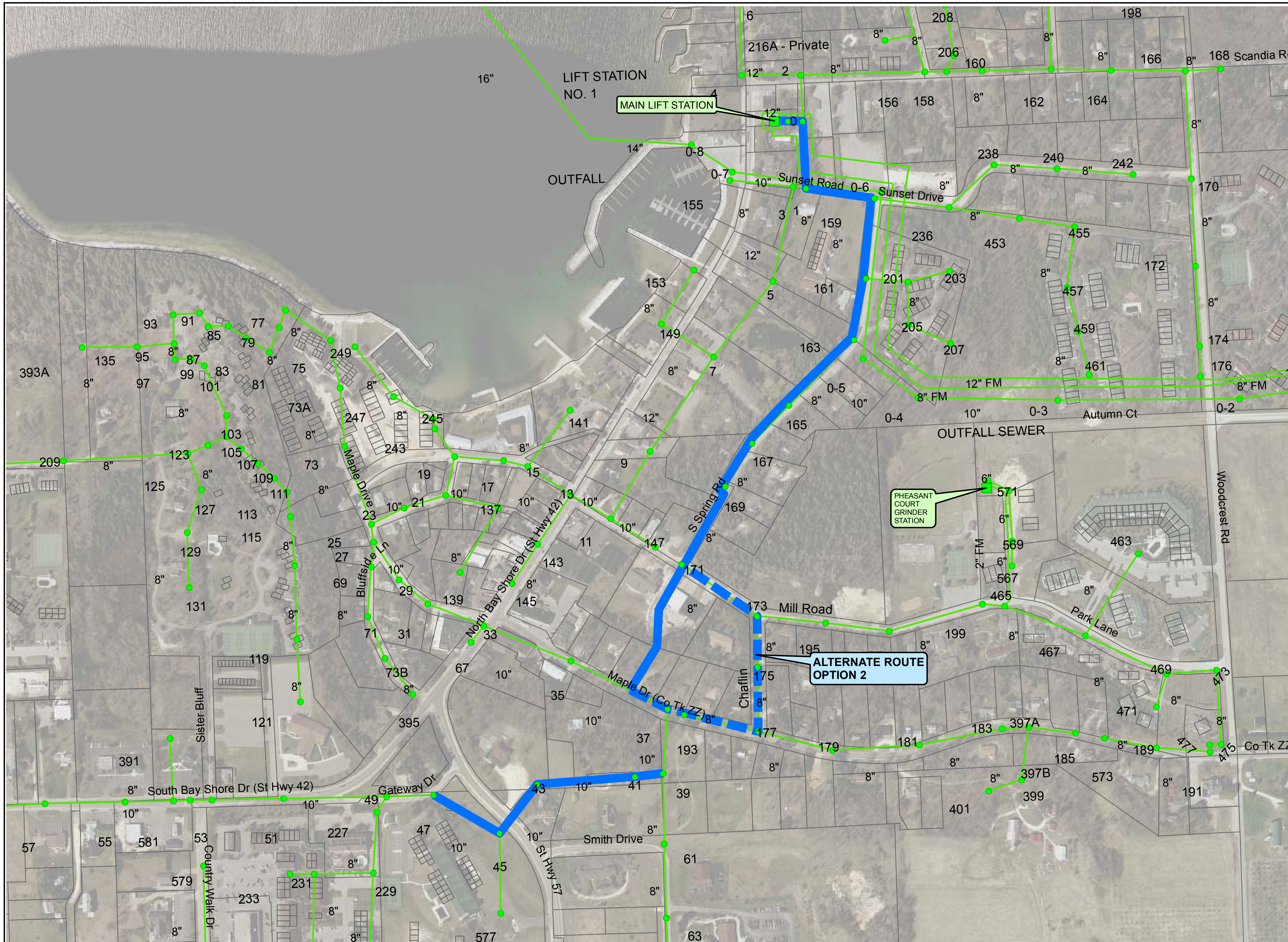
DATE:
02/20/2008

COMPREHENSIVE UTILITIES PLAN

Village of Sister Bay

Sanitary Sewer Routing Option 1

Figure
F-1



Legend

- Proposed Sewer System Improvements - Option 2
- Proposed Sewer System Improvements - Alternate Option 2
- Existing Sewer System
- Existing Lift Stations
- Existing Manhole

N

0 400 800 Feet

Source:
Door County and Bay Lake RPC.
Projection:
Wisconsin State Plane
Map by:
SEH

Map Document: (S:\PT\S\Sistb\050200\GIS\SanitaryModel\Fig_F-2_SanitaryRouting_Option2.mxd)
4/9/2008 -- 2:49:35 PM

SEH
425 WEST WATER STREET
SUITE 300
APPLETON, WI 54911-6058
PHONE: (920) 380-2800
FAX: (920) 380-2801
www.sehinc.com

PROJECT:
ASISTB0502.00

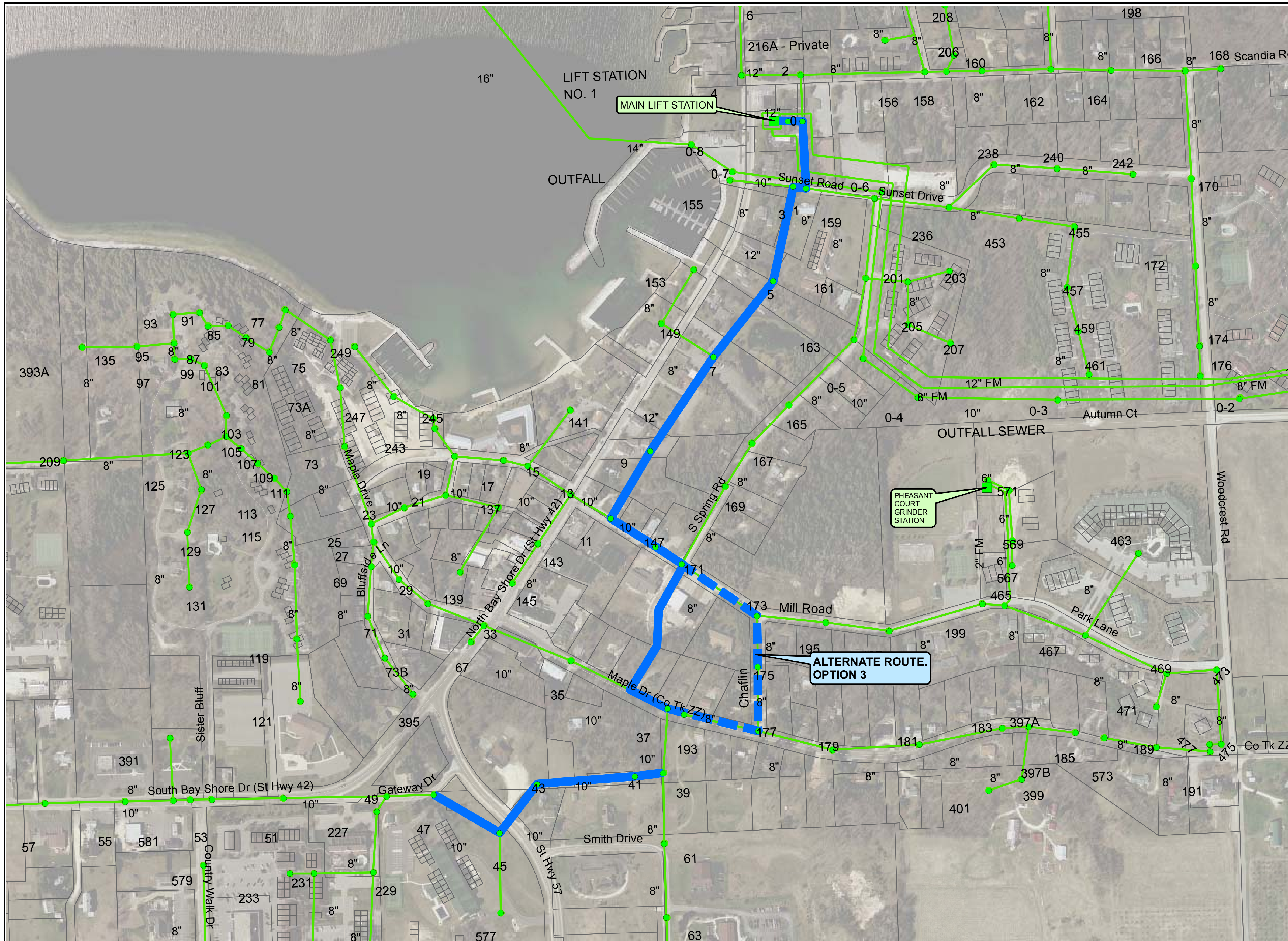
DATE:
02/20/2008

COMPREHENSIVE UTILITIES PLAN

Village of Sister Bay

Sanitary Sewer Routing Option 2

Figure F-2



Legend

- Proposed Sewer System Improvements - Option 3
- Proposed Sewer System Improvements - Alternate Option 3
- Existing Sewer System
- Existing Lift Stations
- Existing Manhole

N

0 400 800 Feet

Source:
Door County and Bay Lake RPC.
Projection:
Wisconsin State Plane
Map by:
SEH

SEH
425 WEST WATER STREET
SUITE 300
APPLETON, WI 54911-6058
PHONE: (920) 380-2800
FAX: (920) 380-2801
www.sehinc.com

PROJECT: ASISTB0502.00

DATE: 02/20/2008

COMPREHENSIVE UTILITIES PLAN

Village of Sister Bay

Sanitary Sewer Routing Option 3

Figure F-3

Map Document: (S:\PT\S\Sistb\050200\GIS\SanitaryModel\Fig_F-3_SanitaryRouting_Option3.mxd)
4/9/2008 -- 2:51:12 PM