City of Bozeman, Montana

City of Bozeman

Wastewater Collection Facilities Plan Update Final

June 2015

MORRISON MAIERLE, INC.
ENGINEERS • SURVEYORS • PLANNERS • SCIENTISTS

HDR

Bozeman Wastewater Collection Facilities Plan Update

Final Document

June 2015

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Prepared by:

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Bozeman Wastewater Collection Facilities Plan Update

Chapter 1

Executive Summary

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1.1 INTRODUCTION

Periodically, the City of Bozeman undertakes a comprehensive planning effort to update and evaluate its existing wastewater collection system, and to estimate and plan for future expansion based on current population and land use trends. This document represents the most recent effort, which was triggered by land development occurring on the west side of the community. This executive summary briefly describes the plan's contents, conclusions, and recommendations. The goals of this Wastewater Plan are:

- Define and evaluate the existing infrastructure in order to determine capacity and existing flows.
- Estimate location and nature of future population growth, associated increases in wastewater quantities, and their effect on existing infrastructure
- Develop a comprehensive plan to address deficiencies and meet present and future requirements, while continuing to plan for and accommodate the City's growth.

1.2 BASIS OF PLANNING

The basis of planning defines geographical planning limits, existing population and land uses, projected future population increases including density and distribution, and associated projections of wastewater flow increases throughout the wastewater collection system. The planning period is through the year 2034. The planning basis is used to quantify inputs to the sewer model that is used in subsequent chapters to evaluate the existing and future collection systems.

The study area covers approximately 42,400 acres. This area sets boundaries for future development over the planning period. For planning purposes, this study assumes the service area will encompass the entire study area.

Population projections are based on an average yearly population growth of 3.0 percent. For the purposes of this plan, the population/land use distribution is based on a saturation density or full build-out conditions. Projected wastewater flows are based on future land uses.

The current overall "per capita" wastewater flow rate of 128 gallons per capita per day was used as the basis for characterizing current and projected wastewater flows. This value is derived by dividing the total wastewater flow entering the wastewater plant by the population of the city. This value is within the range experienced by most municipalities with predominantly domestic wastes. It is conservatively assumed that water conservation efforts and continuing rehabilitation work on the collection system to reduce infiltration will not have a significant impact in lowering per capita flows over the planning period; these conservation efforts will be offset to some extent as the remaining infrastructure continues to age and potentially deteriorate. Using the City's data on potable water use during non-irrigation months as a good proportional indicator of wastewater generation, the overall "per capita" flow rate was allocated to different land uses in Table 1-1.

Table 1-1 Estimated Wastewater Generation –Average Day

User Category	Percentage of Wastewater Generation	Wastewater (gal/capita/day) [2]
Residential	50%	64.4
Commercial	26%	33.8
Top 8 Water Users [1]	8%	9.6
Montana State University	11%	14.6
Government	3%	4.0
Industry	1%	1.7
Total	100.0%	128

1. Eight highest water consumers; includes hotels and hospital.

2. Per capita value in this table is a direct per capita use per person for residential use and indirect per capita use for the other categories.

The 64.4 gallons per capita per day "residential user" category combined with a value of 2.17 persons per household based on the 2010 census data results in 140 gallons per day per dwelling unit, which was used in wastewater generation estimates for residential uses. Future commercial flows were allocated based on an estimated flow per acre. Using the above wastewater generation rates, population densities and other information, wastewater generation rates were calculated on a "per acre" basis for a defined set of land use categories. The "per acre" values were then applied to existing uses; future development areas with current land use designations or zoning; and future development of areas with currently undefined land use. Details are provided in Chapter 2.

The wastewater flow value of 64.4 gallons per capita per day developed with this plan is significantly lower than values used in previous planning efforts. The 2007 plan used a value of 89 gallons per day per capita. The difference is attributed to a combination of using more precise data, reduced levels of infiltration due to rehabilitation projects and an increase in percentage of homes that use lower flow fixtures than were used prior to the Energy Policy Act of 1992 which was the first substantial national legislation focused on lowering water use for fixtures.

1.3 EXISTING SYSTEM DATABASE

This study combines and updates inventories from the 1998 City of Bozeman Wastewater Facility Plan, 2007 update, and the current GIS inventory. Additionally, the database was updated to include data from the City's record drawings for recent projects. Limitations to the database include some missing manhole/pipe elevation and slope data, and data on a variety of elevation datums which cannot be readily correlated. To fill in this missing information for modeling purposes, estimated values were arrived at based on surrounding elevations. At the time of this study, the wastewater collection system is made up of approximately 210 miles of gravity sewer mains, 4,200 manholes, twelve lift stations, and associated force mains that route wastewater to the treatment plant.

1.3.1 Data Base Recommendations

As the wastewater collection system expands, it is important to routinely update the inventory data base. The data base provided with this plan does not include the most recently constructed portions of the collection system and it will need to be updated routinely as development continues to occur.

A process should be established to ensure that GPS data is collected and that the data from the record drawings is processed and placed in the data base on a routine basis. Standard procedures should be developed regarding data acquisition and reporting in order to maintain an accurate data base. Ideally, and particularly for interceptors, trunk sewers and connection points for collectors and laterals, "as-built" surveyed manhole rim elevations, and measured drops to pipe inverts would greatly aid in resolving inconsistent elevation datums found in the database, and therefore providing more accurate identification of deficiencies in the flow models.

The data base improvements recommended in Chapter 3 can be phased in over time as staff time or outside contracts allow. Care needs to be taken to maintain the model as the data is revised or expanded. A program should be developed so that both physical and condition information is collected whenever a pipe or manhole is inspected.

1.4 MODEL DEVELOPMENT AND CALIBRATION

Since the 2007 Facility Plan was completed, the collection system has increased by approximately 55 miles of gravity sewer mains (37 percent increase), 850 manholes (26 percent increase), and 6 additional lift stations and associated force mains (100 percent increase). The Davis-Fowler Interceptor was the most significant addition; it provides relief to the Baxter Interceptor and diverts flows into the 27th Ave/Cattail Creek Interceptor.

The basis for evaluations of the existing and future collection systems is a revised wastewater collection system model, which was established for this study using Innovyze's InfoSWMM version 12, service pack 1, update 3 sewer modeling software.

1.4.1 Model Development

As discussed in Chapter 3, City GIS information was compiled with as-recorded drawings and system databases for import into the collection system model. To remain consistent with the City's design standards, a Manning's roughness coefficient of 0.013 was assigned to all pipes regardless of material and age. All force mains were assigned a Hazen-Williams roughness coefficient (C value) of 120.

For modeling purposes, the gravity wastewater collection system has been subdivided into three categories: *collectors/laterals*, which primarily feed into *trunk sewers*, which then feed primarily into *interceptor sewers*. Although collectors and laterals comprise about 75 percent of the total pipe length of the system, their importance is limited to the point where they connect into trunk sewers. In these upstream portions of the system, high variability in the timing and related volume of the actual system flows correlates to low confidence in model flows and related system capacity. System evaluation is therefore focused on the trunk lines and interceptors. Trunk sewers in Bozeman's system generally range in size from 10 to 18 inches in diameter, and comprise about 15 percent of the total pipe length of the system; Interceptors, the largest pipes in the system, make up the remaining 10 percent of the system's gravity pipes. This study identifies eight interceptors.

The model includes eleven existing lift stations within the City's wastewater system; seven of these are owned and operated by the City. Three of the City-operated lift stations have been installed since the 2007 Facility Plan, and all of these are located on the west side of the City.

1.4.2 Model Calibration

In the spring of 2014, flow was monitored over a period of 1½ to 2 months near the discharge points of four of the interceptors: 27th Avenue/Cattail Creek; Baxter; 19th Avenue/11th Avenue; and North Frontage Road. Flow was also monitored at the influent of the wastewater treatment plant during the first six months of 2014. During the period of flow monitoring, rainfall data was monitored at three different locations within the City; this provided some spatial variability to the rainfall data and allowed for different rainfall profiles to be applied to different parts of the system. The rainfall data provided the basis for wet weather calibration of the model.

The model was calibrated using a rainfall event on May 18, 2014 and calibrating against the associated flow monitoring data. The system manholes were split into seven different drainage areas, each with its own hydrograph, based on which flow monitor they drain to and which rain gauge is closest. The wet weather scenarios were calibrated by modifying the unit hydrographs until a reasonable match was seen between the modeled flow and the monitored flow. The dry weather and wet weather calibration results were generally within 10 percent of the monitor results and for the purpose of this study, the model is a good indicator for the system performance. For the purposes of evaluating the capacity of the existing and future system a 25-year design storm was utilized.

1.5 EXISTING SYSTEM EVALUATION

The collection system was evaluated to identify deficiencies, defined by the City's design criteria as a depth of flow to pipe diameter (d/D) ratio exceeding 0.75. These deficiencies indicate potential capacity issues in the collection system. Based on this criterion, the existing system did not exhibit capacity deficiencies at average dry-weather flows or at peak wet-weather flows. Of the eight interceptors, the model shows the North Frontage Road Interceptor to be nearest to capacity, with a capacity of 7.0 MGD and a peak wet weather design flow of 4.6 MGD in the section of parallel 20-inch diameter pipes.

Under existing conditions, all of the lift stations have enough capacity to pump the peak wet weather design flow, and no deficiencies were identified. In conjunction with planning for future expansion and build-out, the existing lift stations will be evaluated further to compare the costs of constructing larger regional lift stations, as proposed in the 2007 Facility Plan, to using smaller, localized lift stations similar to the lift stations that have been built since the 2007 Facility Plan.

1.5.1 Recommendations

The highest confidence in the model results was in the interceptors in the vicinity of monitoring locations; confidence is reduced upstream in the trunk and collector sewers. The capacity of the interceptors and the capacity modeled both depend on the slope of the interceptor. There is high uncertainty in the elevations used for inverts in the model due to various vertical datum values used over time in new development areas within the City. As more accurate pipe invert elevations are collected, the model should be re-evaluated for capacity.

The model assumes the diameter of each pipe is accurate based on GIS data provided by the City. The internal diameter may be significantly smaller than the reported diameter and this may cause capacity constraints. This is especially true in large diameter PVC and HDPE pipes as well as pipes that have been lined. If there are pipes with actual internal diameters not in agreement with the current GIS data, the GIS data should be updated in the model to re-evaluate pipe capacities.

Although not used in the evaluation of the existing system, the pipe flow velocities calculated in the modeling effort can be used in the City's maintenance program; pipes with velocities less than 2 fps should be regularly cleaned due to potential for settling/line blockage that could cause reduced capacity, and pipes with velocities greater than 5 fps should be regularly checked for structural problems such as excessive scouring.

The City should continue with the recommendations from the previous master plan to reduce infiltration and inflow into the system. Specifically, the City should take the following steps:

- Enforce existing ordinances to prevent crawl space sump pumps from discharging to the sewer system.
- Rehab or replace old clay pipe in the older parts of the City.
- Continue the program to CCTV and record approximately 20 percent of the existing collection system each year to identify specific problem areas associated with aging pipe.

The City should implement a routine flow monitoring program to track wastewater flows in the collection system. It is recommended that, at a minimum, the four flow monitoring sites utilized in this study be monitored on a biannual basis. As the city expands, a more robust flow monitoring would be appropriate. Project specific flow monitoring should be considered for larger scale development projects to confirm model data, particularly in areas located far from the study's flow monitoring sites.

1.6 FUTURE SYSTEM EVALUATION

Expansion and improvements to the existing wastewater infrastructure will be needed to serve population growth within the service area. Most of the undeveloped and vacant land within the existing city limits is in the west, north and northeast portions of the City. As these areas develop, they will add flows to the interceptors on the east and west side of the system and new collectors will need to be constructed to connect the new development to the existing interceptors.

Future interceptors, trunk lines and lift stations were added to the existing system model to evaluate future system needs. The gravity sewer main extensions were planned to follow existing roadways where roads existed and followed drainage basins where roads did not exist, and to follow existing topography to maximize gravity flow. Lift stations and force mains were planned where gravity flow would not be feasible. The model was used to preliminarily size proposed extensions, including lift stations and force mains. In particular, the areas of expansion to the north and west are at a lower elevation than the existing infrastructure and will require lift stations.

The future system was evaluated for two planning periods. The near-term planning period, called "Existing and Obligated," assumes the entire area within the current city limits is completely developed. The "Existing and Obligated" model will show if the existing infrastructure has capacity to meet current obligations for service, and what improvements would be required. The long-term planning period, called "Study Area Build-Out," assumes the entire study area is completely developed based on land uses in the Community Plan. The "Study Area Build-Out" model shows what improvements and expansions to the wastewater system will be required to serve the entire future service area at full build-out. The "wet weather" models determine system capacity; gravity main capacity deficiencies are noted based on the City's criteria that peak-hourly wet-weather flow depth should not exceed 75 percent of the pipe diameter.

1.6.1 System Deficiencies

The Existing and Obligated planning model identified several areas that, before full build-out of this scenario, will become deficient and will require sewer main extensions, replacement, new parallel main, or pumping. The major items are:

- The existing trunk sewer from the intersection of I-90 and Main Street to the Rouse Interceptor (contingent on growth on the east side of the City).
- The North Frontage Road Interceptor, at two separate sections of parallel 20-inch pipes.
- The South University area.

The Study Area Build-Out planning model, which would increase the service area from 11,700 to 42,500 acres, identified several areas that will become deficient before full build-out of this scenario and will require sewer main extensions, replacement, new parallel main, or pumping. The major items, additional to the above-described "Existing and Obligated" items, are:

- Replace the older section of the Davis-Fowler Interceptor with a 24-inch diameter pipe.
- Increase capacity of the entire length of the North Frontage Road Interceptor.
- Replace the 30-inch WWTP interceptor, which receives flow from the entire City, with a 48-inch pipe.
- The 15-inch section of the Norton East Ranch Outfall Sewer (at Baxter Lane and Flanders Mill Road) will reach capacity as the Aajker Creek drainage basin develops. Improvements could involve either a partial or full diversion of these flows, with pumping implications described in Chapter 5 - Future Collection System Evaluation.
- Much of the Aajker Creek basin drains to areas at lower elevation than any existing sewer infrastructure. It is possible for the southern portion of the Aajker Creek basin to drain by gravity to the Norton East Ranch Outfall Sewer Interceptor. However, in order to reduce the ultimate flow to the existing infrastructure a new diversion can be built to divert some of the flow north to the new lift station required to service the rest of the basin (the proposed future Davis Lane Lift Station). The diversion will not be needed until approximately 40 to 60 percent of the basin to the south of this new diversion is developed.
- Expand capacity of the Norton Ranch Lift Station before the Norton Ranch Subdivision is fully developed.
- Numerous new extensions are required to provide full service to the planning area as identified in Chapter 5.

- In order to serve areas north of the existing system, four lift stations are proposed to pump flows to existing infrastructure: Gooch Hill, Hidden Valley, Davis Lane and Spring Hill.

The need and timing of these improvements will largely be driven by development, either to extend service to an area or to provide adequate capacity in the existing system. The city should look for opportunities to install the infrastructure in an economically efficient fashion where possible. One example of this is to install the Davis-Fowler Interceptor at the time the underlying property annexes while it is still undeveloped instead of waiting for the timing to be driven by capacity needs.

1.6.2 Recommended Improvements

As identified in Chapter 5 and summarized in Chapter 6, a series of projects are recommended. Due to uncertainty in development timing and locations within the service area, it is difficult to determine when future improvements will be needed. System flows depend on what development has occurred within the contributing area, and improvements should be timed based on development timing.

The projects summarized in Table 1-2 are recommended to serve future growth within the existing city limits. This includes providing new sewer to areas that currently aren't developed and only includes pipes sized 12-inches and greater. The analysis performed to identify the projects and more details on the projects is provided in Chapter 5 of this report.

Table 1-2 – Category 1 Improvements

Project Name	Improvements Description	Probable Cost
Front Street Interceptor	Replace or parallel 8,500 feet of sewer along Front St and Haggerty Ln from E Tamarack St to Ellis St	$ 2,180,000
North Frontage Road Interceptor	Replace or parallel 11,500 feet of the North Frontage Rd Interceptor between Springhill Rd and Bridger Dr	$ 5,290,000
South University District	New 5,000 feet of sewer to divert South University District development flows to the Davis-Fowler Interceptor	$ 1,120,000
Norton Ranch Lift Station	Increase capacity at the Norton Ranch Lift Station to support further development of the Norton Ranch Subdivision	$ 500,000
Davis Lane Lift Station	Construct small initial Davis Lane Lift Station to serve area north of the Cattail Lake Lift Station	$ 500,000
Bridger Drive Extension	Install a new 2.5 mgd capacity sewer along Bridger Drive from Birdie Drive to Story Mill Rd	$ 300,000

Additionally, 29,800 feet of sewer extensions were identified to serve currently undeveloped areas within the city limits that were less than 12-inches in diameter. The total probable cost of these extensions was estimated at $6,680,000.

The projects summarized in Table 1-2 are recommended to serve future growth outside the existing city limits. The analysis performed to identify the projects and more details on the projects is provided in Chapter 5 of this report. Many of these projects will be required outside of the City's normal 20 year planning period and should be re-evaluated as the land use and growth patterns are updated. It is important to note that if development is concentrated in a basin contributing to the project, the project may be required sooner to serve new development.

Table 1-3 – Category 2 Improvements

Project Name	Improvements Description	Probable Cost
Davis-Fowler Interceptor	Replace or parallel 2700 feet of the Davis-Fowler Interceptor between Durston Rd and W Oak St	$ 760,000
WWTP Interceptor	Replace or parallel 1200 feet of sewer from I-90 to the WRF	$ 420,000
27th Avenue/Cattail Creek	Replace or parallel 3300 feet of the WWTP Interceptor Sewer	$ 950,000
Bridger Creek Golf Course to North Frontage Rd Sewer	Replace or parallel 7700 feet of sewer around the Bridger Creek Golf Course and from the Course to North Frontage Rd	$ 1,590,000
Bridger Dr to North Rouse Ave	Replace or parallel 1300 ft of sewer along Bridger Drive between Birdie Drive and North Rouse Avenue	$ 320,000
Norton East Ranch Diversion	Divert flow from the Norton East Ranch Sewer to the Davis Lane Lift Station	$ 3,320,000
Davis Lane Lift Station Expansion	Expand the Davis Lane Lift Station for the Norton East Ranch Diversion flows	$ 5,300,000
Gooch Hill Lift Station and Forcemain	Construct a 6.2 mgd lift station and forcemain	$ 7,820,000
Hidden Valley Lift Station and Forcemain	Construct a 1.5 mgd lift station and forcemain	$ 5,190,000
Spring Hill Lift Station and Forcemain	Construct a 4.2 mgd lift station and forcemain	$ 4,650,000

In addition to the improvements described above, an additional 380,000 feet of trunk lines at an estimated cost of $78,000,000 will be required to serve areas outside of the current city limits and within the Community Plan Boundary. All of the expansions to the City is heavily dependent on where development occurs and service is required.

1.7 POLICY AND PROGRAM RECOMMENDATIONS

A number of policy and program related recommendations were identified as part of the planning process. These include both new policies and important existing policies. The following is a list of those recommendations:

1. Drainage Basin Integrity
   It is important that the drainage basins be maintained as outlined in the report to avoid impacting the available capacity assigned to each drainage basin. The proposed improvements are sized based on the contributing area defined within each drainage basin and any significant changes in the basin boundaries may result in a lack of capacity in the planned extensions. It is recommended that the city implement a policy to prohibit transferring flow from one basin to another.
2. Collect Accurate Elevation Data – Existing System
   For the existing system, particularly for trunk and interceptor sewers, obtaining accurate manhole rim and invert elevation would improve the accuracy of future wastewater modelling efforts.
3. Standardize Elevation Datum – Future System
   The standard process for establishing project specific datums has historically relied on the city's fire hydrant benchmark program. This method served the city well for many years; however, as the city has expanded the benchmark system is prone to compounding errors. With current survey technology it is recommended that a new standard system be developed.

4. Sump Pumps and Roof Drains
   Continuing to enforce current policies to prevent sump pump and roof drain connections to the sewer system is critical to maintaining capacity for wastewater flows.
5. Collection System Rehabilitation
   Continuing an annual rehabilitation program at current or higher levels is important in order to continue to reduce infiltration and maintain system integrity as the overall system ages.
6. Flow Monitoring
   A routine program of flow monitoring in the collection system is recommended.

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Bozeman Wastewater Collection Facilities Plan Update

Chapter 2

Basis of Planning

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2.1 INTRODUCTION

The Basis of Planning chapter sets the stage for the detailed technical planning needed for wastewater collection facilities planning. The planning period is defined as 20 years with the end of the planning period being the year 2034. The area to be included in the plan is defined in this chapter. A literature review of existing population trends is presented along with an analysis of future populations. Land use patterns are examined and future land uses are reviewed as they impact wastewater planning. Existing wastewater flows at the treatment plant are presented along with projections of flows through the planning period. Recommendations are provided for distribution of wastewater flows within the study area.

2.2 STUDY AREA

2.2.1 Introduction

In order to begin an analysis of the wastewater needs of a community, a study area needs to be defined. A study area provides a geographic boundary which defines a reasonable area for future development over the planning period while focusing the planning efforts to a defined boundary.

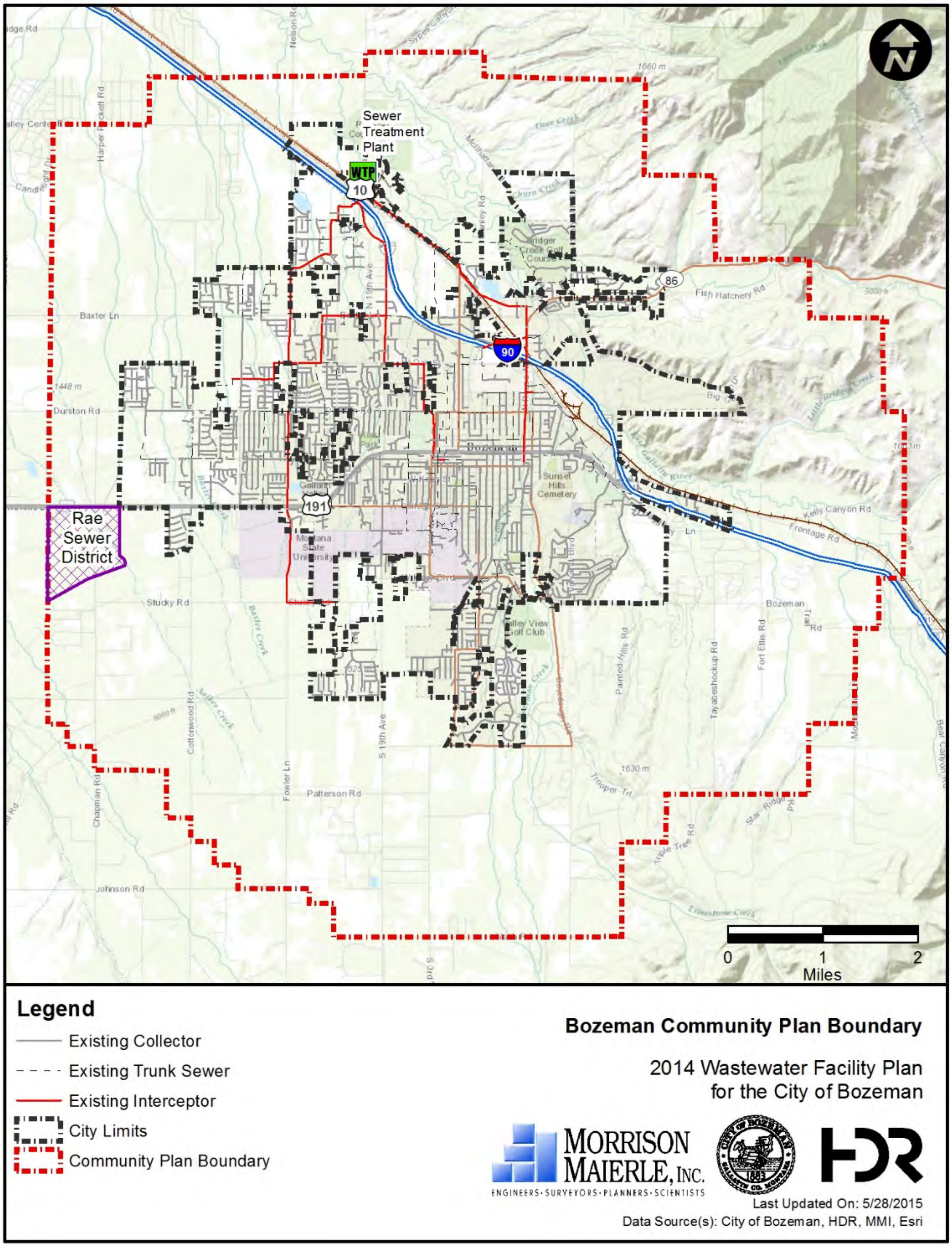
2.2.2 Study Area Boundary

Based on discussions with City Staff as part of the project scoping, the study area utilized for this study is the same that was developed for the 2007 Bozeman Wastewater Facility Plan. The study area was developed at that time through an analysis of geographical boundaries, a review of available planning documents, a cursory analysis of development trends and discussions with City staff. The final study area boundary was presented to the City Commission on March 7, 2005 and adopted as part of the overall plan.

2.2.3 Study Area Description

The study area includes approximately 42,400 acres and is shown in Figure 2-1. The study area generally consists of land that it is moderately sloped and generally slopes from south to north. The northeast corner of the study area consists of land with steeper slopes and varied terrain. The area is bisected by a variety of ditches, creeks, and rivers. The notable water bodies include the East Gallatin River, Bozeman Creek, Rocky Creek, and Bridger Creek. A network of small creeks and irrigation ditches crosses the westerly side of the study area. Groundwater is near the ground surface in many areas within the study area which provides for a significant amount of wetlands. In general, the study area is conducive of development which can make use of central wastewater facilities. A more comprehensive review of the history, physiography, and description of the study area can be found in the Bozeman Community Plan. Study area characteristics which impact the planning of wastewater facilities is located in Section 2.7.2.

FIGURE 2-1


Sewer Treatment Plant
WTP
10
86
90
191
Rae Sewer District
Montana State University
Bozeman
Baxter Ln
Durston Rd
Stucky Rd
Patterson Rd
Johnson Rd
Fish Hatchery Rd
Kelly Canyon Rd
Frontage Rd
Valley View Golf Club
Sunset Hills Cemetery
Bridger Creek Golf Course
0
1
2
Miles
Legend
Existing Collector
Existing Trunk Sewer
Existing Interceptor
City Limits
Community Plan Boundary
Bozeman Community Plan Boundary
2014 Wastewater Facility Plan for the City of Bozeman
MORRISON MAIERLE, INC.
ENGINEERS · SURVEYORS · PLANNERS · SCIENTISTS
HDR
Last Updated On: 5/28/2015
Data Source(s): City of Bozeman, HDR, MMI, Esri

2.3 POPULATION AND LAND USE

2.3.1 Introduction

Current and projected population data is required in order to evaluate the existing wastewater facilities and to plan for the future. A review of land use patterns is also an important part of analyzing existing capacities and future needs. An analysis of population trends along with a review of published population projections has been completed.

2.3.2 Existing Population Conditions

The population of the City of Bozeman has been steadily growing over the past twenty five years. A review of historic population figures for Bozeman is discussed and a more detailed review of population trends since the year 2000 is provided below.

The United States Census Bureau conducts a detailed census of the United States every 10 years and makes the data from each census available to the public. The following table provides the City's historic ten-year population trends based on United States Census Bureau data from the year 1900 to the year 2010:

Table 2-1 – City of Bozeman Historic Population Trends by Decade

Year	Population	Percent Increase per Decade
1900	3,419	
1910	5,107	49.4%
1920	6,183	21.1%
1930	8,855	43.2%
1940	8,665	-2.1%
1950	11,325	30.7%
1960	13,361	18.0%
1970	18,670	39.7%
1980	21,645	15.9%
1990	22,660	4.7%
2000	27,509	21.4%
2010	37,280	35.5%

In addition to the detailed census completed each decade, the United States Census Bureau provides annual population estimates each year. These estimates are based on the population in the month of July compared to the decennial census which is based on the population in the month of April. The following table lists the Census Bureau's yearly estimates from the year 2011 to 2013.

Table 2-2 – City of Bozeman Recent Population Trends by Year

Year	Population	Percent Increase per Year
2011	38,116	2.2%
2012	38,753	1.7%
2013	39,860	2.9%

Based on the 2011- 2013 trend estimates, the population continues to grow at a rate similar to the last 20 year average. The average yearly percent of increase over the period is approximately 2.3 percent. Current development activity indicates that the growth rate is increasing.

2.3.3 Existing Land Use Conditions

Every community is unique in terms of its wastewater flow and loads. Characteristics that provide for variation in flows and loading are related to the type of user. For example, some communities have large industrial users which generate either very high wastewater flows or very high wastewater loadings. In other communities, the commercial and industrial flows and loadings may be very small, such as in a "bedroom" community. Some of the unique characteristics of Bozeman that relate to wastewater include the following: 1) It is the home of Montana State University. 2) There is a limited amount of industrial activity. 3) It serves as a regional commercial center.

Montana State University is the largest employer in Bozeman and contributes greatly to the population base. The student population of MSU is approximately 15,000 with approximately 75% of the students living within the City limits of Bozeman. The student population has recently increased significantly from the 12,000 student level that persisted from approximately 1996 to 2008. Based on the City of Bozeman Community Plan, MSU directly accounts for approximately 11% (2,400) of the total employees working within the City limits. The university conducts a large amount of research in a wide array of fields. MSU also attracts a large number of people to the community for athletic events, conferences, and other activities.

Bozeman has limited industrial activity. The industrial business in Bozeman generates a minor portion of both the wastewater flow and loads to the wastewater treatment plant. In many communities, industrial users, such as, dairy processors, breweries and other large water users, can greatly impact wastewater treatment plants. The City should monitor industrial activity, and if a large user locates in Bozeman, the impact on the City's wastewater facilities should be evaluated.

Bozeman serves as an established regional commercial center drawing employees, shoppers, health patients, and others from Gallatin County and the surrounding areas. This activity increases wastewater loads and flows beyond what is considered normal for a typical city the size of Bozeman. The temporary visitors to the community contribute wastewater to the system to a varying degree. For example, a person staying at a motel may generate a similar amount of wastewater as a resident of the City, while a consumer or an employee that spends a day in the city will generate much less wastewater than a typical fulltime resident.

Land use patterns have been provided by the City of Bozeman GIS Department based on land use categories listed in the Bozeman Community Plan. Figure 2-1 shows the location of the various types of land use with the city limits of Bozeman. Table 2-3 lists the land use categories and associated acreage based on the 2013 Land Use Inventory Report for all property within the City limits.

Table 2-3 – City of Bozeman Land Use 2013

Land Use Category	Acreage
Commercial/Auto	119
Commercial/Retail	449
Hotel/Motel	99
Light Manufacturing	368
Mixed Use	269
Restaurant/Bar	41
Public Facility/Park	1,768
Administrative/Professional	215
Church	90
Duplex/Triplex Residential	268
Mobile Home/Mobile Park	87
Multi-Family Residential	567
Single-Family Residential	1,694
School/Educational/Facility	865
Golf Course	177
Right of Way	2,347
Vacant	3,377
Total	**12,797**

2.3.4 Population Projections

Projections of future populations are important for infrastructure and financial planning. Existing information available from previous City of Bozeman studies provide the base line for population projections for this study. The most current work has been completed as part of the Integrated Water Resource Plan. As baseline information the data in the 2007 Bozeman Area Transportation Plan and the Bozeman Community Plan have been reviewed.

The Integrated Water Resource Plan identified a moderate and a high growth population projection. The moderate growth rate was estimated at 2% over the first 20 years of the projections and the high growth rate was estimated at 3% over the first 20 years of the projections. For conservative planning purposes, the high growth rate is recommended for this plan.

The Montana Department of Commerce data suggests an annual population increase of 1.8% in the early years of the planning period tapering to approximately 0.5% at the end of the planning period for Gallatin County as a whole.

Based on the 3% growth rate projection, Table 2-4 shows the projected population through the planning period.

Table 2-4 – City of Bozeman Population Projections – 2014 to 2034

	2013	2014	2024	2034
Projected Population	39,860	41,056	55,176	63,964

2.3.5 Land Use Projections

Projecting future land uses in a detailed manner is not the primary focus of an infrastructure planning study. However, it is useful to examine trends and available planning documents that impact overall land use patterns.

The best available land use planning document for the City of Bozeman is the Bozeman Community Plan. The Bozeman Community Plan provides land use designations for all land within the plan boundary which provides a useful basis for wastewater system planning.

Land use trends that impact wastewater collection system planning include overall population/land use densities and significant changes in percentages of various land use categories. City of Bozeman census data from the last two censuses indicate a downward trend in the overall average number of people occupying a dwelling unit. In 1990, the average number of people per dwelling unit was 2.5. In 2000, the average number declined to 2.3 people per dwelling unit. The average number further declined in 2010 to 2.17 people per dwelling unit. This decrease in occupancy rates results in an increased number of total dwelling units for a given population.

Another noted trend is a decrease in lot size, resulting in increased housing density that has occurred over the past fifteen years. The City of Bozeman's Unified Development Code encourages higher density development than previous City planning regulations. The current economic climate in terms of lot supply/demand and cost encourages developers to market smaller lots and increase project densities.

The general mix of land use categories is not expected to dramatically change over the duration of the planning period. However, increasing lot density is an important factor in the planning of wastewater collection facilities.

2.4 SERVICE AREA

2.4.1 Introduction

A service area is a defined boundary wherein a community makes a commitment to provide wastewater service as facilities become available. For the purposes of facility planning, a service area can be a subset of the study area. In many communities, service area boundaries adjoin the boundaries of other wastewater districts or other community wastewater system service areas. The reasons for having a distinct service area can be based on a number of geographical, topographical and political reasons.

2.4.2 Service Area Delineation

The recommended service area for this plan is the entire defined study area. This is based on the general trend of the City to accept annexations to expand the size of the community and to promote development that utilizes central wastewater services. At the present time, it does not appear that any other wastewater district or community, other than the City of Bozeman, is reasonably positioned to provide wastewater service to significant amounts of land within the study area.

2.5 POPULATION AND LAND USE DISTRIBUTION

2.5.1 Introduction

The existing and future distribution of population through the study area is an important consideration in wastewater collection system planning. The land use patterns and the distribution of population establish the basis for wastewater collection system needs. These patterns can also be utilized in the evaluation of potential regional wastewater treatment or collection system improvements. It is important to note that this document is not intended to be a land use planning document. Thus, the population and land use projections are made on a generalized basis in order to accommodate future growth. For the purposes of this plan, the population/land use distribution is based on a saturation density or full build-out conditions. The specific location of future growth and land are predicted in a general fashion. In order to adequately size the collection system for conservative long term growth scenarios, a saturation model is appropriate.

2.5.2 Future Study Area Characteristics

The study area includes a mix of developed and undeveloped lands and a variety of land uses. The study area contains numerous micro scale features that impact development potential. These include streams, wetlands, areas of steep slopes, utility and transportation corridors, and other similar features. While these features impact development potential at a micro scale within the study area, they do not significantly impact the projection of wastewater flows on a basin-wide basis. The two significant land features that will likely limit development in specific areas include designated flood plains and areas of excessively steep slopes. Planning for these areas is discussed in Section 2.7.3. There are many areas within the study area, which are developed at rural densities and would therefore place a low demand on wastewater collection facilities. However, it is probable that many of the areas will re-develop at higher densities within the 20 year planning period or within the 50 to 100 year useful life of the collection system infrastructure. With these items in mind, the future characteristics of the study area are assumed to mirror the characteristics of those within the existing City limits of Bozeman for the purpose of wastewater collection system planning.

2.5.3 Recommended Distribution Methodology

The recommended procedure for distributing population and associated wastewater flows throughout the study area is based on land use classifications. The study area has been divided into four general categories for this purpose. These categories are as follows:

1. Developed areas within the City limits.

   For developed areas of the City, the land use is defined based on the net area of the land and is defined by a variety of land use categories.

2. Vacant areas within the City limits.

   This land has a defined general land use designation and assigned zoning.

3. Land outside of the existing City limits but within the Bozeman Community Plan boundary that has defined land uses.

   This land is generally undeveloped and unzoned, however, the general land use is defined.

4. Land within the Bozeman Community Plan boundary that is designated present rural.

   This category of land does not have a defined future land use. The Bozeman Community Plan designates it as land that will be incorporated into the City; but, the use of the land is undefined.

The methodology noted above is utilized to assign existing wastewater flows for areas connected to the wastewater system and predict future wastewater flow distribution within the study area for the purposes of wastewater collection system planning. A detailed description of flow distribution is provided in Section 2.7.

2.6 WASTEWATER FLOWS AND LOADS

2.6.1 Introduction

This section is intended to provide a brief summary of historic wastewater flows and loads and establish a basis for projecting future flows. A more detailed review of both wastewater flows and loads is provided in Chapter 5.

2.6.2 Historic Wastewater Flows

The City of Bozeman maintains wastewater flow records based on a flow recorder located at the head works of the treatment plant. The following table shows total average annual average wastewater flows from 1988 to 2014.

Table 2-5 – Bozeman WWTP Service Area Wastewater Flows

Year	Annual Average Influent Flow (mgd)
1988	4.58
1989	4.46
1990	4.33
1991	4.27
1992	4.21
1993	5.03
1994	4.47
1995	5.36
1996	4.79
1997	5.77
1998	5.07
1999	4.77
2000	4.53
2001	4.96
2002	4.97
2003	4.84
2004	4.91
2005	4.77
2006	5.01
2007	5.46
2008	5.58
2009	5.58
2010	5.37
2011	5.64
2012	4.60
2013	4.71
2014	5.17

Table 2-6 reports additional detail for wastewater flows for the years 2011 through 2014.

Table 2-6 – Current Bozeman WWTP Service Area Wastewater Flows

	Influent Flow Rate (mgd)			
Year	**Annual Average**	**Maximum Monthly Average**	**Peak Day**	**Peak Hour**
2011	5.64	7.91	(1)	(1)
2012	4.60	5.11	6.74	8.16
2013	4.71	5.63	5.73	9.69
2014	5.17	6.29	6.79 [2]	8.81 [2]
Average	5.03	6.24	6.42	8.93

1. Data not available
2. Peak data through May 21, 2014. Data not used in average.

2.6.3 Wastewater Flow Per Capita Characteristics

One way of characterizing wastewater flows for a collection system is to summarize the flows on a per capita basis which is beneficial to understand the overall wastewater flows for a community. The per capita value for this evaluation is based on the flow to the plant divided by the population of the city. It does not represent the wastewater directly generated from a household. Table 2-7 summarizes the per capita flow data for the years 2011 through 2013.

Table 2-7 – Current Bozeman Service Area Per Capita Wastewater Flows

Flow Parameters Average Annual Conditions	**2011 Historical Data**		**2012 Historical Data**		**2013 Historical Data**		**2014 Historical Data**		**Average Historical Data**	
	Parameter Value	**Per Capita Value**	**Parameter Value**	**Per Capita Value**	**Parameter Value**	**Per Capita Value**	**Parameter Value**	**Per Capita Value**	**Parameter Value**	**Per Capita Value**
Flow (mgd, gal/capita/day)	5.64	148	4.62	119	4.71	118	5.17	126	4.98	128

The City of Bozeman completed work on rehabilitation of the collection system in 2011 including repairing break-in sewer taps in the Thompson Addition which reduced infiltration in the system. It is likely that this work in addition to other rehabilitation activities has helped reduce flows to the Water Reclamation Facility.

The current per capita flows and loads have significantly decreased through time and are now closer to being within the ranges experienced by most municipalities with predominantly domestic wastes. The data indicates a downward trending flow on a per capita basis through the course of the last three studies. The differences are demonstrated below in Table 2-8:

Table 2-8 – Per Capita Wastewater Flow and Load Data

Parameter	**Normal Literature Values**	**1998 Facility Plan Values**	**2007 Facility Plan Values**	**Current Values**
Average Flow (gal/capita/day)	100 – 120	173	157	128

The recent flow data has been compared to water use data compiled in the City of Bozeman Integrated Water Resource Plan. The data provided in that plan shows a similar trend in a reduction in water use over the time period. The peak demand year for water use was 211 gallons per capita per day (gpcpd) in 1993 and has reduced to 134 gpcpd in 2010. Indoor water use was estimated to be 114 gallons per capita per day in 2010. The indoor water use value includes 23 gpcpd of accounted water. The unaccounted water is likely a combination of system losses and metering accuracy problems. Given the indoor water use that reaches the wastewater collection system is less than the 114 gpcpd when reduced by a portion of the unaccounted water and that there is some infiltration and inflow into the wastewater collection system, the recent per capita flow data from the wastewater plant appears reasonable.

Water conservation efforts to reduce indoor water use and continued rehabilitation work on the collection system to reduce infiltration will likely have some impact in lowering per capita flows over the planning period. For planning purposes, it is conservatively assumed that these efforts will provide some reduction in flows but overall will not have a significant impact. For example, as the rehabilitation effort makes progress on the oldest infrastructure, the remaining infrastructure continues to age and potentially deteriorate.

2.7 RECOMMENDED WASTEWATER FLOW PROJECTIONS AND DISTRIBUTIONS

2.7.1 Introduction

The projection of future wastewater flows is an important component of planning for future wastewater collection and treatment facility needs. The predicted distribution of the wastewater flow within the study area is used for collection system planning. The recommended flow projections and distributions along with the chosen methodology are discussed below.

2.7.2 Wastewater Flow Projections

There are a number of methods of estimating and projecting flows for wastewater facilities. Each method has its pros and cons which are discussed in the following sections.

The most straight forward approach to quantify and project flows is to utilize the existing per capita flow rates and projected flow rates based on future population estimates. The advantage of this approach is that population data and historical flows and loading data is readily available for past and current conditions. Additionally, projections of future population conditions are available on a routine basis. This allows for future users of the Wastewater Facility Plan to easily compare the projections completed at the time of the study to existing conditions in the future. The disadvantage of this method is that the ratio of commercial and industrial flow and loads compared to domestic flows and loads is assumed to remain constant for the projections.

Another approach to projecting wastewater flows involves using a combination of employment and population data. Based on literature values and a detailed review of commercial and industrial flows, flow can be estimated based on a per capita and a per employee basis. The advantage of adding the employment numbers into the equation is that it provides a measure of economic and commercial activity to the projections which may better reflect changes in the commercial and industrial wastewater contributors. Population projections and employment data are routinely updated for planning purposes.

Depending on the type of employment, the amount of wastewater attributed to each employment sector can vary substantially. For example, a restaurant with 10 employees will generate much more wastewater on a per capita basis than an office with 10 employees. However, the number of employees in a community is an indicator of the economic activity, which directly correlates to transient populations such as consumers, employees, conference attendees, and motel guests, which contribute to wastewater flows and loads.

A third approach is to conduct a detailed flow assessment for current classes of connections to the City's wastewater system. This can be completed by a detailed review and analysis of all commercial wastewater accounts and a review of other information such as the number of motel rooms in the community. This methodology allows for flows to be assigned to particular classes of activities such as single family houses, apartments, offices, restaurants, motels, etc. The advantage of this method is that it provides for a direct accounting and distribution of flows generated from the system. The disadvantage of utilizing this methodology on a system wide basis is that it is very difficult to accurately project specific uses into the planning period. One other factor that makes the projections less useful than other methodologies is that the data is difficult to track and update. There is also no simple method to substantiate whether the projections are accurate through time.

Having considered the existing and future anticipated community characteristics, the available data sources and anticipated future trends, it was determined that the most appropriate methodology for this plan is to base flow projections on existing per capita flow and loading data and future population projections. The community character, in terms of the percentage of various facets of the community, is anticipated to remain similar through the planning period thus the population approach is applicable.

Based on existing per capita flow values, a flow value of 128 gallons per day per capita is the recommended value to use for planning purposes. The per capita flow value should be routinely evaluated over time. If it increases or decreases substantially, the impact on the plan projections should be evaluated. Table 2-9 summarizes the design population and resulting projected wastewater flow and loads through the planning period.

Table 2-9 – Projected Influent Wastewater Flows

FLOW PARAMETERS	2013	2014	2024	2034
Design Population	39,860	41,056	55,176	63,964
Annual Average Flow, mgd	4.7	5.2	7.1	8.2

2.7.3 Wastewater Flow Distribution

The distribution of wastewater flow throughout the study area can be based on a variety of methodologies depending on how the underlying land use is defined. The following categories of land use definitions have been defined as discussed in Section 2.4.3.

1. Developed areas within the City limits.
2. Vacant zoned areas within the City limits.
3. Land within the Bozeman Community Plan boundary that has defined land uses.

4. Land within the Bozeman Community Plan boundary that is designated present rural.

The flow distribution for these areas is based on existing conditions, water use, and total wastewater flow values. Potable water use and wastewater generation are related. Water use data is available so the water use data is a good tool to use in determining wastewater flows. On a system wide basis, the City's water use during non-irrigation months is a good proportional indicator of wastewater generation. Table 2-10 shows the estimated percentage of water use for six categories identified in the Integrated Water Resource Plan.

Table 2-10 Water Use – 2010

User Category	Percentage of Water Use	Water Use (gal/capita/day) [3]
Residential	40%	45.6
Commercial	21%	23.9
Top 8 Water Users [1]	6%	6.8
Montana State University	9%	10.3
Government	2%	2.8
Industry	1%	1.2
Total	79.0% [2]	90.6

1. Eight highest water consumers; includes hotels and hospital.
2. Remaining water use is reported as unaccounted.
3. Per capita value in this table is a direct per capita use per person for residential use and indirect per capita use for the other categories.

The data in Table 2-10 is extrapolated in Table 2-11 to account for the 128 gpcpd of average wastewater flow.

Table 2-11 Estimated Wastewater Generation –Average Day

User Category	Percentage of Wastewater Generation	Wastewater (gal/capita/day) [2]
Residential	50%	64.4
Commercial	26%	33.8
Top 8 Water Users [1]	8%	9.6
Montana State University	11%	14.6
Government	3%	4.0
Industry	1%	1.7
Total	100.0%	128

1. Eight highest water consumers; includes hotels and hospital.

2. Per capita value in this table is a direct per capita use per person for residential use and indirect per capita use for the other categories.

The computed per capita flow for the residential user category is 64.4 gallons per day per capita as shown in Table 2-10. It is important to note that this is significantly different than the 128 gallon per day per capita that is recommended for use in projecting flows for the wastewater treatment plant. The 63.6 gallon per day per capita difference is due to flow attributed to the commercial, Montana State University, government and industry categories. The 64.4 gallon per day per capita figure is to be used only in connection with the collection system in residential use areas.

Existing wastewater flows were allocated to land use categories based on the estimated 2010 flow allocation shown in Table 2-10. Residential wastewater flow was distributed based on a per capita flow basis of 64.4 gallons per day per capita derived from the data in Table 2-10 and estimated dwelling unit densities for the four classifications of residential land uses. A value of 2.17 persons per household is used based on the 2010 census data. This results in 140 gallons per day per dwelling unit. Commercial flows were allocated utilizing data from the 2007 City of Bozeman Wastewater Facility Plan. Additional information on wastewater flow allocation for the existing system is included in Appendix 2A.

Table 2-12 shows the wastewater flow allocated over the various land use categories based on the methodology described above.

Table 2-12 Wastewater Flow Rate by Land Use Category for Existing Uses [1]

Category	Dwelling Units per Acre	GAL./ACRE/DAY
Commercial/Auto	-	50
Commercial/Retail	-	1,000
Hotel/Motel	-	6,000
Light Manufacturing	-	800
Mixed Use	-	1,000
Restaurant/Bar	-	3,500
Public Facility/Park	-	25
Administrative/Professional	-	1,000
Church	-	360
Duplex/Triplex Residential	8.8	1,232
Mobile Home/Mobile Park	8.8	1,232
Multi-Family Residential	8.8	1,232
Single-Family Residential	6.1	854
School/Educational/Facility	-	400
Golf Course	-	30
Right of Way	-	0
Vacant	-	0
MSU [2]	-	2,220

1. The flow allocation in this table is based on net area.
2. Flow rate is based on MSU property east of South 19th Avenue.

To evaluate future collection system needs, wastewater loading needs to be assigned to areas based on anticipated future land use characteristics. In areas within the City limits, the assigned zoning provides the best tool to approximate future wastewater loadings. Table 2-13 shows the recommended wastewater flow rates based on a per acre basis for zoned areas. Detailed information on how the rates were developed is located in Appendix 2A.

Table 2-13 Wastewater Flow Rate for Zoned Undeveloped Areas [1]

Designation	Dwelling Units per Acre	GAL./ACRE/DAY
R-S	6.5	910
R-1	3.9	546
R-2	5.2	728
R-3	6.5	910
R-4	10.4	1,456
R-O	5.2	728
RMH	5.2	728
B-1		1,000
B-2		2,000
B-3		3,000
M-1		960
M-2		960
B-P		960
NEHMU	6.5	910
UMU	10.4	1,456
REMU	10.4	1,456
PLI		1,030

1. The flow allocation in this table is based on gross area.

In areas within the Bozeman Community Plan Boundary that have a defined land use, wastewater flows can be allocated on the land use designation. Table 2-14 provides the recommended wastewater flow rate by land use designation.

Table 2-14 Wastewater Flow Rate by Land Use Designation [1]

Designation	Dwelling Units per Acre	GAL./ACRE/DAY
Industrial	-	960
Neighborhood Commercial	-	1,200
Community Commercial	-	2,400
Regional Commercial	-	1,600
Business Park	-	960
Public Institutions	-	1,030
Residential	5.5	770
Suburban Residential	1.3	182
Park& Open Space	-	25
Other Public Lands	-	1,030
Golf Course	-	30
MSU	-	2,780
MSU West		1,030

1. The flow allocation in this table is based on gross area as land area

Table 2-15 identifies the recommended flow allocation for areas that are defined as present rural urban by the Bozeman Community Plan. These areas are the least defined in terms of land use. As such, the projected flows are based on equivalent residential dwelling unit densities. Additional detail is provided in Appendix 2A.

Table 2-15 Wastewater Flow Rate for Undefined Land Use Designations [1]

Designation	Dwelling Units per Acre	GAL./ACRE/DAY
Present Rural	5.5	770

1. The flow allocation in this table is based on gross area

The wastewater flow rates presented above are used to develop input values for computer modeling of the existing and future collection system. It is anticipated that some adjustments to the input values may be necessary to calibrate the flow model over time. For instance, flow rate measurement points in the collection system may indicate that adjustment of the flow rate from a specific land use class is needed.

2.8 REFERENCES

City of Bozeman, Bozeman Community Plan
City of Bozeman (2012), Integrated Water Resource Plan – AE2S
City of Bozeman (2014), Land Use Inventory Report
City of Bozeman, Wastewater Flow Data
US Census Bureau, (2012), 2010 Census Data
US Census Bureau, (2014) Population Estimates

Bozeman Wastewater Collection Facilities Plan Update

Chapter 3

Existing Collection System Database

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3.1 INTRODUCTION

The City of Bozeman is served by a wastewater collection system made up of approximately 210 miles of gravity sewer mains, approximately 4,200 manholes and twelve lift stations and associated force mains that route wastewater to the treatment plant. This chapter is intended to provide an overview of the development of the collection system data base and recommendations for maintenance and future improvements to the data base.

3.2 EXISTING DATA

3.2.1 Introduction

The collection system inventory is based on numerous sources of existing data. This section reviews previously completed inventories and reviews other data currently available to compile an inventory appropriate for this planning effort.

3.2.2 Previous Collection System Inventories

There are three primary collection system inventories that have been completed in the recent past:

- An inventory developed in a sanitary sewer model format completed as part of the 1998 City of Bozeman Wastewater Facility Plan;
- The current GIS inventory, prepared and maintained by City staff;
- A 2007 update of the 1998 City of Bozeman Wastewater Facility Plan.

The 1998 inventory was completed concurrently with sewer collection system modeling using a software package called SANSYS. This program utilized pipe size and slope information to assign upstream and downstream manholes and invert elevations to each pipe, to numerically define the collection system. The inventory was specific to the software package. The pipe and connectivity data was based on a combination of previous SANSYS model data and additional data from more recent record drawings. The data base did not include coordinate information. The pipe network drawings developed with this model were schematic in nature, based on the data available and not tied to the software model data.

The current GIS inventory was developed by the City's GIS Department and has been kept current to the extent feasible; it includes the vast majority of pipes and manholes and lift stations installed up to the present time. The GIS data base inventory includes the following collection system attributes:

- Pipe location
- Pipe size
- Pipe material type
- Pipe installation year (portion of system)
- Manhole identifier

- Manhole location coordinates based on GPS data (excluding elevations)

This GIS data base is a good tool for day-to-day operation and maintenance of the collection system and for general use by City staff such as the Planning and Engineering Departments. The 2007 study improved the GIS inventory so it could be utilized for collection system flow modeling. This study used the expanded, current GIS inventory.

After first modifying and expanding the 1998 inventory to include structures shown on recently completed record drawings, the 2007 Wastewater Facilities Plan integrated the available data into a database for use in SewerCAD, the City's selected software at that time.

3.2.3 Available Data

In addition to the inventory information noted in Section 3.2.2, the following collection system information was available from City records:

- Record drawings and construction drawings for sewer projects that became available after the most recent data utilized in the 2007 Wastewater Facilities Plan.
- Quarter-section maps.
- City "Sewer Boards" for sewer lines installed before record drawings became a standard.

Limitations on the available data were reviewed and the following conclusions were drawn:

- Elevation data was not available for all manholes and pipes.
- Where elevation data was available, it was based on a variety of datums which cannot be readily correlated.
- Pipe slope data was not available for all pipes.

3.3 DATA BASE DEVELOPMENT

3.3.1 Introduction

The improved data base needed to meet a number of criteria in order to be useful for the wastewater collection system analysis and planning effort. These included a need to be compatible with the City's GIS system and also with the City's selected upgrade for sewer collection modeling software, InfoSWMM. In addition, the development of the data base had to be completed within a reasonable budget. The following sections explain how the data base was developed and summarize the completed data base.

3.3.2 Methodology

The previous collection system inventories and currently available data including the existing GIS data base and recent record drawings provide the starting point for creating a revised inventory and data base that can be used to update the existing collection system flow modeling. GIS technology was determined to be the most appropriate means of revising the inventory and is relatively easy to update. The pipe and manhole data were developed using the City's existing GIS files; data gaps were filled by referencing record drawings compiled by the City since the 2007 Wastewater Facilities Plan.

In this update to the 2007 plan, the City's GIS files were converted to an Excel spreadsheet for data entry; this data was then converted back to GIS shape files for use in the InfoSWMM wastewater model. As part of the process of transferring pipe and manhole information from record drawings to the spreadsheet data base, numerous discrepancies and data omissions were identified. These were resolved to the extent feasible prior to modeling; the remaining discrepancies and omissions were identified and resolved as needed during the modeling.

The following is a list of the data categories included in the inventory, and an explanation of how the data was obtained or created.

- Pipes, Slope Data

  In the previous (2007) study and originally coming from the 1998 SANSYS model, slope data was provided by the City Engineering Office. In a few cases, slope data was unavailable. In such cases, a hydraulically conservative slope of 50% of the standard minimum slope was assigned and noted in the data base. Slope data for the more recently constructed pipes was added from the record drawings reviewed in the 2007 plan.

  Manhole inverts, rather than pipe slopes, were input from the record drawings reviewed for this study. Pipe slopes are computed in the InfoSWMM model based on invert elevations at the manhole; therefore, slopes were not directly entered from the record drawings. Manhole invert data, combined with manhole locations and pipe layout, resulted in the pipe slopes used in the model. To greatly increase efficiency of initial data input, the "invert out" was the only invert provided for modeling, and all pipe inverts at each manhole were assumed to match the manhole's "invert out." The elevation drop in a typical manhole is 0.1 to 0.2 feet which is not hydraulically significant.

- Pipes, Other Data

  - Diameter - Pipe diameter was taken from the GIS data base and the record drawings. Where obvious discrepancies or omissions existed, the quarter section maps and record drawings were utilized to correct the data.
  - Length - Pipe length is computed based on GPS coordinates of manholes.
  - Location - Location is established based on GPS coordinates of manholes.
  - Year Installed - This data was supplied by the City in the existing GIS data base.
  - Material Type - This data was supplied by the City in the existing GIS data base.

- Manholes, Invert Elevations

  Elevation data was required in order to run the InfoSWMM modeling software, but the available data was on various elevation datums that could not be easily correlated. To address this, the SewerCAD model of the 2007 study was constructed starting with an assumed elevation at the water treatment plant. From that point, individual data segments (original SANSYS data, subdivision sewer systems, etc.) were linked together sequentially going in an upstream direction. This essentially provided datum conversions for each data segment. At some locations, converging segments had different invert elevations for the same pipe at the same manhole; at these locations it was necessary to use professional judgment and make assumptions to correlate elevations sufficiently for modeling. The computed elevations are suitable for flow modeling, but the user is cautioned that they do not correspond to actual elevations. Please refer to the recommendations listed in Section 3.4.3 regarding improving the existing data base in the future.

In this study, the existing model was expanded to include all manholes entered into the City's current (2014) data base. Record drawings provided invert elevations, and each record drawing plan set (or subset) was converted to the datum provided by its point(s) of connection to the existing (2007) model data.

- Manholes, Other Data
  - Horizontal Location - In the previous study, manhole locations were supplied by the City in the existing GIS data base. Where data was found to be missing, the City provided coordinates for the manholes. The City's expanded GIS data base used in this study was adequate to cover the expanded model, without the need to request additional manhole coordinates.
  - Rim Elevations - In the previous study, rim elevations were arbitrarily assigned an elevation five feet above the elevation of the invert of the outlet pipe. It was determined that this was adequate for modeling purposes and budget was not available to add actual depths to the data set. Rim elevations for the more recently constructed manholes (after 2007) were added from the record drawings reviewed in this study.
- Lift Stations – Lift station data was taken from a combination of record drawings, operation and maintenance manuals and other data available in the City files. A summary description of the lift stations is included in Chapter 4.

3.4 DATA BASE RECOMMENDATIONS

3.4.1 Introduction

The inventory data base should be continually updated and improved. The following recommendations should be implemented as staff time and budget allow.

3.4.2 Data Base Maintenance

As the wastewater collection system expands, it is important to routinely update the inventory data base. The data base provided with the plan does not include the most recently constructed portions of the collection system and it will need to be updated routinely. A process should be established to ensure that the GPS data is collected and that the data from the record drawings is processed and placed in the data base on a routine basis. Standard procedures should be developed regarding data acquisition and reporting in order to maintain an accurate data base.

3.4.3 Data Base Improvements

The modeling data is based on available information which was compiled at the level of detail and accuracy needed to evaluate and model the collection system. It is anticipated that the data base will be expanded and refined as the City's time and budgets allow. While the efforts for facility planning purposes focused on improving the data base for flow modeling, a collection system data base can be used for a number of operation and maintenance activities. Future improvements to the data base should address improving overall accuracy and usefulness for all purposes. The following recommended improvements should be implemented:

- Collect survey grade elevation data for both pipe inverts and manhole rims based on a single datum.
- Field verify pipe size, configuration and material information.
- Continue to collect condition assessment data for both pipes and manholes.
- Add in field flow monitoring data to data base when available. This can be used to check or calibrate the flow model.

The recommended data base improvements can be phased in over time as staff time allows. Care needs to be taken to maintain the model as the data is revised or expanded. A program should be developed so that both physical and condition information is collected whenever a pipe or manhole is inspected.

Bozeman Wastewater Collection Facilities Plan Update

Chapter 4

Existing System Evaluation

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Model Calibration Hydrographs

4.1 INTRODUCTION

The City of Bozeman (Bozeman) last evaluated the City's entire wastewater collection system in 2007 for the Bozeman Wastewater Facility Plan (2007 Facility Plan). At the time, the City was served by a wastewater collection system made up of approximately 150 miles of gravity sewer mains, approximately 3,300 manholes and six lift stations and associated forcemains that convey wastewater to the wastewater treatment plant (WWTP). The City has experienced significant growth since the 2007 Facility Plan was completed and has added approximately 55 miles of gravity sewer mains (37-percent increase over 2007 system), 850 manholes (26-percent increase) and six additional lift stations and associated forcemains (100-percent increase) to the collection system. The most significant change was with the construction of the Davis-Fowler Interceptor that provided relief to the Baxter Interceptor and diverted flows to the 27th Ave/Cattail Creek Interceptor. This chapter is intended update the basis of the 2007 Facility Plan and to provide an overview of the existing collection system, hydraulic model, and any existing system capacity deficiencies.

4.2 WASTEWATER SYSTEM DESCRIPTION

A major objective of this study is to evaluate the existing wastewater collection system, determine the performance and capacity of existing facilities and their suitability for incorporation into a long-range program to convey future wastewater flows. Figure 4-1 shows an overview of the existing wastewater system.

The existing collection system is currently comprised of 210 miles of pipe varying in diameter from 4-inches for forcemains to 30-inches for the largest system interceptor. The system is also served by approximately 4,100 manholes and 11 lift stations. All of these facilities drain to the WWTP located in the north side of the system.

4.2.1 Gravity System

To better understand the modeling and evaluation process completed as part of this update, the major components that comprise the gravity portion of the collection system are defined. The three basic elements are:

- Collectors and Laterals
- Trunk Sewers
- Interceptor Sewers

4.2.1.1 Collectors and Laterals

The smallest unit of the system, collectors and laterals are designed to collect wastewater from point sources (residential, commercial and industrial sources) for transport to trunk sewers. These pipes are generally 8-inches in diameter and in some instances smaller, and their location within the system varies widely depending on the configuration of housing developments, commercial sites and industrial networks. Collectors and laterals comprise over 150 miles of pipe, or approximately 75-percent of the total footage of pipe in Bozeman's wastewater collection system.

The importance of collectors and laterals in a system-wide study is limited to the point where they connect into trunk sewers and the dry weather and possible wet weather flows they contribute to these sewers. The importance is limited since the confidence in model flows and related system capacity in the upstream portions of the City due to high variability in the timing and related volume of the actual system flows. For example, at the upstream reaches of a collection system, water use and related sanitary flows generated in the morning is dependent on when individuals and groups of people get up and start to get ready for their day. The timing and volume of flow can vary widely at the top of a gravity system. Once these flows reach the downstream areas of the system, it shows a much more defined diurnal pattern. Therefore, model results are inherently more accurate downstream as the variability is averaged out with the more flow that is introduced into the system. Collectors and laterals are rarely evaluated in a system-wide study unless there are known capacity issues in those areas of the system.

4.2.1.2 Trunk Sewer

The trunk sewer is a system element that collects wastewater from the point of discharge of a collector system for conveyance to the interceptor system. They are primarily designed to serve a smaller sewershed comprised of a number of point sources that topographically drain to a single point. This element may also be called an outfall sewer. Trunk sewers generally range in size from 10-inches to 18-inches in diameter, which comprises 40 miles of pipe, or approximately 15-percent of the total length of Bozeman's collection system.

4.2.1.3 Interceptor Sewer

The interceptor sewer is the largest unit of the collection system that intercepts and accumulates wastewater flows generated from a larger drainage basin, a collection of smaller drainage areas, for transport to the WWTP. Interceptors are designed to intercept flows from upstream trunk or other interceptor sewers. If the sewer is designed to intercept flows from a number of upstream interceptor sewers, it is defined as a combined interceptor sewer. For the Bozeman collection system, there are eight (8) major interceptors, comprising 15 miles of pipe, or approximately 10-percent of the total system length. Table 4-1 summarizes interceptor data for the existing system.

Table 4-1 - Summary of Interceptors

Interceptor	Size	Capacity (MGD)[1]	Year Installed	Drainage
Rouse	30 24	14 8	2004	Eastern
North Frontage Road	30 Parallel 20 20	5 7 8	1969	Eastern, Northern
19th Avenue/ 11th Avenue	24	10	1969/1992	Central
Baxter	24 21 20	4 4 3	1980/1981 1999 1981	Western
27th Avenue/ Cattail Creek	27 24	10 9	2002	Western
Evergreen	21	4	1969	Eastern, Northern
Davis-Fowler	18 24	5 13	2007	Western, Northern
WWTP	30	19	1969	All

[1]Capacity of the interceptor was determined based on the minimum slope pipe within the interceptor at full flow

Rouse Interceptor

The Rouse Interceptor is a 24-inch and 30-inch interceptor, originally installed in the 1960s that has been rehabilitated within the past 10 years. It collects flow from the eastern portion of Bozeman, including Burrup Lift Station, and discharges into the North Frontage Road Interceptor. The interceptor extends north along Rouse from Babcock to north of Interstate 90. The 30-inch interceptor ranges in capacity from approximately 14 to 38 million gallons per day (MGD), and capacity in the 24-inch interceptor ranges from approximately 8 to 21 MGD.

North Frontage Road Interceptor (WPCA 125/EPA)
The North Frontage Road Interceptor parallels North Frontage Road and Interstate 90. It collects flow from the Rouse Interceptor as well as residential, golf course restrooms, industrial and commercial development along North Frontage Road, including Gallatin Park. It varies between parallel 20-inch asbestos cement interceptors and a single 30-inch asbestos cement interceptor. The 30-inch line was installed as part of the North 7th Avenue Sewer Relocation in 1996. The parallel interceptors were installed in the late 1960s and 1980. The tie-in with the upstream 19th Avenue/11th Avenue Interceptor is a single 20-inch pipe with a capacity of approximately 8 MGD. Capacities upstream vary between approximately 5 and 13 MGD.

19th Avenue/11th Avenue Interceptor (WPCA 125)
The 19th Avenue/11th Avenue Interceptor collects flow from the central portion of Bozeman, and jogs between 11th Avenue and 19th Avenue, where it crosses the Interstate to discharge into the WWTP Interceptor. It is a 24-inch interceptor that was installed in the late 1960's with a capacity of approximately 10 MGD. Approximately 5,000 linear feet of the northern portion of the interceptor was replaced in 1992 as part of the I-90 & 19th Sewer Relocation. The Baxter Interceptor discharges into this interceptor at Baxter and 19th Avenue. Land uses vary between residential, and commercial, and include Montana State University.

Baxter Interceptor (Valley West/SID 621)
This 24-, 21-, and 20-inch Interceptor collects flow from the western portion of Bozeman including Laurel Glen and Norton Ranch Subdivisions and with a capacity of approximately 3 to 9 MGD. It was installed in 1981 and 1999 as part of Sewer Improvement District (SID) 621 and Valley West projects. Land uses in this area include mostly residential, with some commercial. The interceptor discharges into the 19th Avenue/11th Avenue Interceptor at Baxter and 19th Avenue.

27th Avenue/Cattail Creek Interceptor
The 27th Avenue/Cattail Creek Interceptor collects flow from the northwestern portion of Bozeman including the 27th Avenue area, Baxter Meadows Subdivision, and other commercial and residential flows. The material of this interceptor is PVC, with diameters ranging from 24 to 27-inch, and with a capacity of approximately 9 to 13 MGD. This interceptor ties in with the 19th Avenue/11th Avenue Interceptor south of the Interstate into the Wastewater Treatment Plant Interceptor.

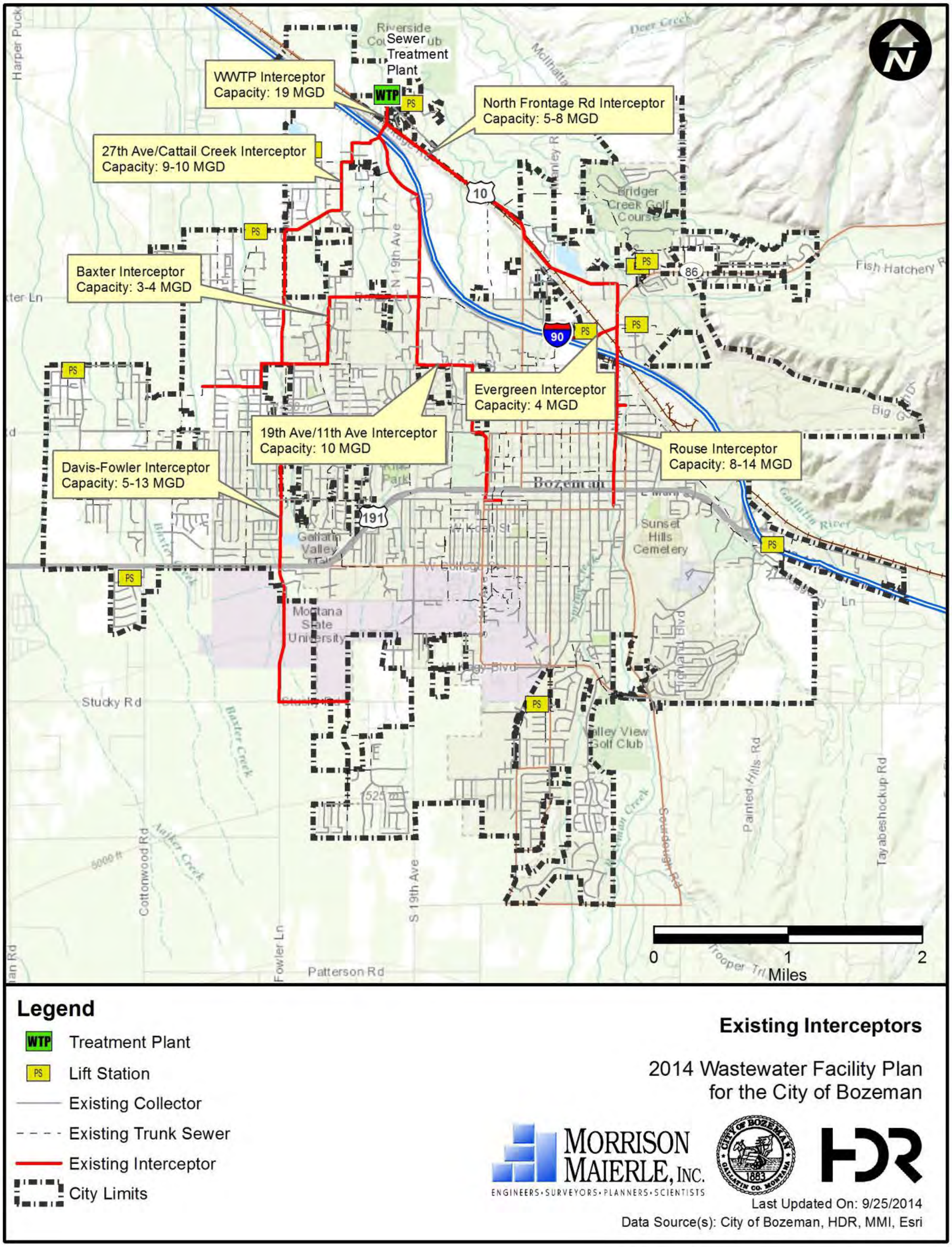
Evergreen Interceptor
The Evergreen Interceptor is a 21-inch clay pipe installed in the late 1960s with a capacity of approximately 4 to 6 MGD. It collects flow from commercial development along the North 7th Avenue business corridor, as well as the Evergreen Business Park and includes an Interstate crossing. It discharges into the Rouse Interceptor near Bryant north of the Interstate.

Davis-Fowler Interceptor
The Davis-Fowler Interceptor varies in size from 18 to 24 inches and runs along Fowler and Davis Avenue from the Meadow Creek Subdivision the 27th Avenue/Cattail Creek Interceptor. Most of

the interceptor was installed in 2006 and 2007 and is made of PVC. A segment connecting Fowler Avenue and Davis Avenue was installed in 1939 and is made of asbestos cement. The capacity of the interceptor varies between approximately 5 to 16 MGD. The interceptor discharges to the 27th Avenue/Cattail Creek Interceptor at Cattail Street.

Wastewater Treatment Plant Interceptor
This is the main interceptor discharging to the wastewater treatment plant. It collects flows from the 27th Avenue/Cattail Creek, 19th Avenue/11th Avenue, and North Frontage Road Interceptors. This interceptor includes an Interstate crossing and has a capacity of approximately 19 to 24 MGD. The interceptor was installed in the late 1960s and is asbestos cement.

Figure 4-1 - Existing Interceptors



4.2.2 Lift Stations

There are eleven existing lift stations within the City's wastewater system. Of the eleven lift stations, seven of the lift stations are owned and operated by the City. Three of the City's lift stations have been installed since the 2007 Facility Plan and all of the new lift stations are located on the West side of the City. Table 4-2 provides a summary of the existing lift stations in the collection system and Figure 4-2 shows the location of the lift stations.

Table 4-2 – Existing Lift Station Summary

Lift Station	Ownership	Forcemain Size (inches)	Approximate Firm Capacity (gpm)	Standby Power	Interceptor Drainage	Year Installed
Baxter Meadows	City	12	690	Yes	Cattail Interceptor	2002
Bridger Center	City	4	100	No	Rouse Interceptor	2004
Burrup	City	6	450	Yes	Rouse Interceptor	1984
Cattail Lake	City	6	225	Yes	Cattail Interceptor	2007
Laurel Glen	City	8	450	Yes	Valley West Interceptor	2003
Loyal Gardens	City	6	364	Yes	Davis-Fowler Interceptor	2007
Norton Ranch	City	4/6	121	Yes	Valley West Interceptor	2010
Overbrook	Private	4	Unknown	Unknown	11th Avenue Interceptor	Unknown
Sebena	Private (not in service)	8	none	n/a	Rouse Interceptor	Unknown
Walker	Private	8	Unknown	No	WWTP Interceptor	1992
Links	Private	4	160	Yes	N. Frontage Interceptor	2008
Cardinal Distribution	Private	4	Unknown	Unknown	N. Frontage Interceptor	Unknown

Baxter Meadows Lift Station

The Baxter Meadows Lift Station serves a commercial and residential development west of 27th Avenue and north of Baxter. The lift station consists of three (3) identical 13 horsepower pumps. The peak hourly flow can be handled by any two pumps, while most flows are handled with a single pump. Firm pump capacity is 690 gpm with a maximum total dynamic head (TDH) of 36 feet (32 feet if only one pump is operating). The lift station discharges via 1,140 linear feet of 12-inch PVC forcemain to the 24-inch 27th Avenue/Cattail Creek Interceptor.

Bridger Center Lift Station

The Bridger Center Lift Station serves the Bridger Center Drive industrial area and discharges via a 4-inch forcemain to a 12-inch collector sewer that discharges into the Rouse Interceptor. This lift station was private, but was transferred to the City in 2004. The lift station consists of two submersible 1.5 horsepower pumps with an approximate firm capacity of 100 gpm. The lift station operates with one pump until water levels rise to 3 feet above the wet well floor (4,674.50 feet). The elevation of the high water alarm is 4,678 feet.

Burrup Lift Station

The Burrup Lift Station serves a large area along Frontage Road, north of the Interstate at the Main Street Interchange, and consisting of the Shawnee Industrial Park and other businesses. The lift station discharges via a 6-inch forcemain to a 10-inch collector along Haggerty on the south side of the Interstate. The 10-inch collector feeds into the Rouse Interceptor. The Burrup Lift Station is the oldest lift station in the system (built around 1984). The station consists of two (2) five (5) horsepower vacuum prime sewage pumps. The station has a firm capacity of 450 gpm at 26 feet TDH.

Cattail Lake Lift Station

The Cattail Lake Lift Station was built in 2007 to serve the Cattail Lake Subdivision. The lift station discharges via a 6-inch PVC forcemain to an 8-inch collector along Blackbird Dr. The pump station consists of 2 pumps with a capacity of 225 gpm each.

Laurel Glen Lift Station

The Laurel Glen Lift Station serves the Laurel Glen subdivision. The lift station discharges via an 8-inch PVC forcemain of approximately 5,300 linear feet to the Valley West Interceptor at Durston and Cottonwood Road. The lift station consists of two submersible Flygt pumps with a firm capacity of 465 gpm with a TDH of 75 feet.

Loyal Gardens Lift Station

The Loyal Gardens Lift Station was built in 2007 to serve the Loyal Gardens subdivision. The lift station discharges via a 6-inch ductile iron forcemain to a 12-inch trunk sewer along Yellowstone Avenue. The pump station consists of 2 pumps with a firm capacity of approximately 364 gpm.

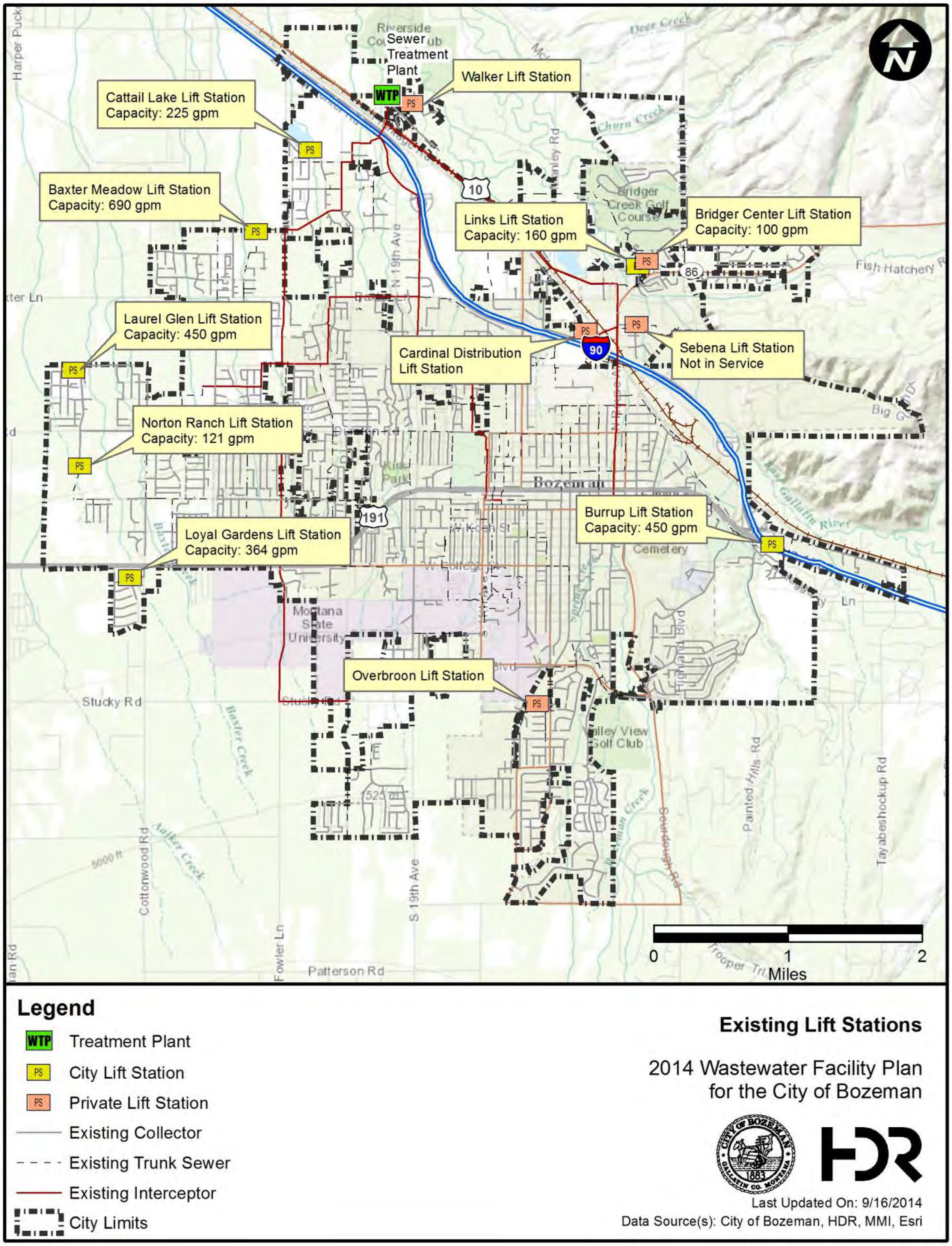
Norton Ranch Lift Station

The Norton Ranch Lift Station was built in 2010 to server the Norton Ranch Subdivision. The lift station discharged via a 4-inch PVC forcemain to a 10-inch trunk sewer along Durston Road. The pump station has an existing firm capacity of 121.5 gpm with plans to expand the station to 750 gpm to accommodate expected full build-out flows.

Private Lift Stations

Private lift stations include Overbrook Lift Station, Walker Lift Station, Sebena Lift Station (not in service), Cardinal Distribution Lift Station and Link Lift Station. Data regarding these stations was limited and the data is not needed for this study do to the small size of the lift stations.

Figure 4-2 - Existing Lift Stations with Firm Capacity



4.2.3 Previous Studies

The 2007 Facility Plan provided recommendations to the existing collection system that focused on the condition of pipes and manholes in the collection system. The recommendations included fixing specific issues identified through CCTV studies as well as recommending a program to CCTV the entire collection system. Since the previous study the City has invested in rehabilitating its existing infrastructure and the City has fixed many condition-related structural and infiltration issues identified in the previous report.

The report specifically made the following conclusions about the City infrastructure at the time:

- Aging infrastructure is a general problem for the entire system that has been addressed through a few large rehabilitation projects, and many small spot repairs.
- City staff indicates that many crawlspace sump pumps are connected to the sewer system and that this has been a practice for over 20 years. There is a City ordinance on record to prevent crawl space pumps from discharging to the sanitary sewer, but this has not been enforced. *Note – Since the time of the 2007 report the City has made progress correcting this situation.*
- The Rouse Interceptor discharges into the North Frontage Road Interceptor. A large capacity differential, i.e. bottleneck, occurs at this location. While this is currently not a problem, since these pipes are not at capacity, growth will likely create a problem as flows increase.

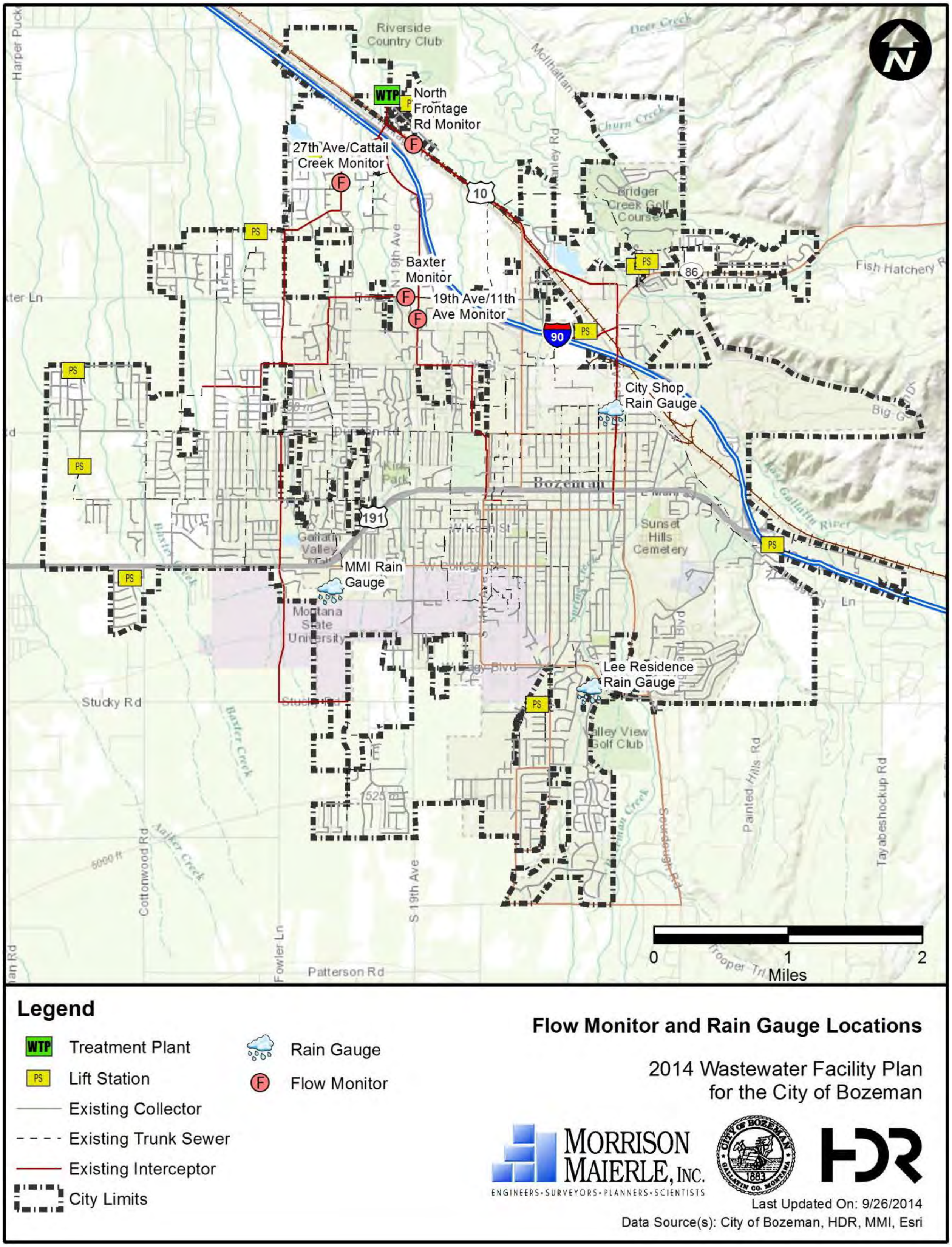
4.3 FLOW MONITORING EVALUATION

4.3.1 Flow Monitor Locations

Flow monitors were placed within the 27th Avenue/Cattail Creek, Baxter and 19th Avenue/11th Avenue Interceptors. The flow monitors recorded flow, depth and velocity at these locations from April 8, 2014 to June 10, 2014. A fourth monitor located within the North Frontage Road Interceptor recorded flow, depth and velocity from April 29, 2014 to June 10, 2014. All four of the flow monitors were placed near the discharge of the interceptors in the northern portions of the collection system near the wastewater treatment plant. The flow monitor locations are shown in Figure 4-3.

In addition to the temporary flow monitors placed in the interceptors within the system, the flow was also monitored at the influent of the wastewater treatment plant. The monitored flow into the treatment plant was provided from January 1, 2012 to May 21, 2014.

Figure 4-3 – Flow Monitor and Rain Gauge Locations



4.3.2 Flow Monitor Equipment

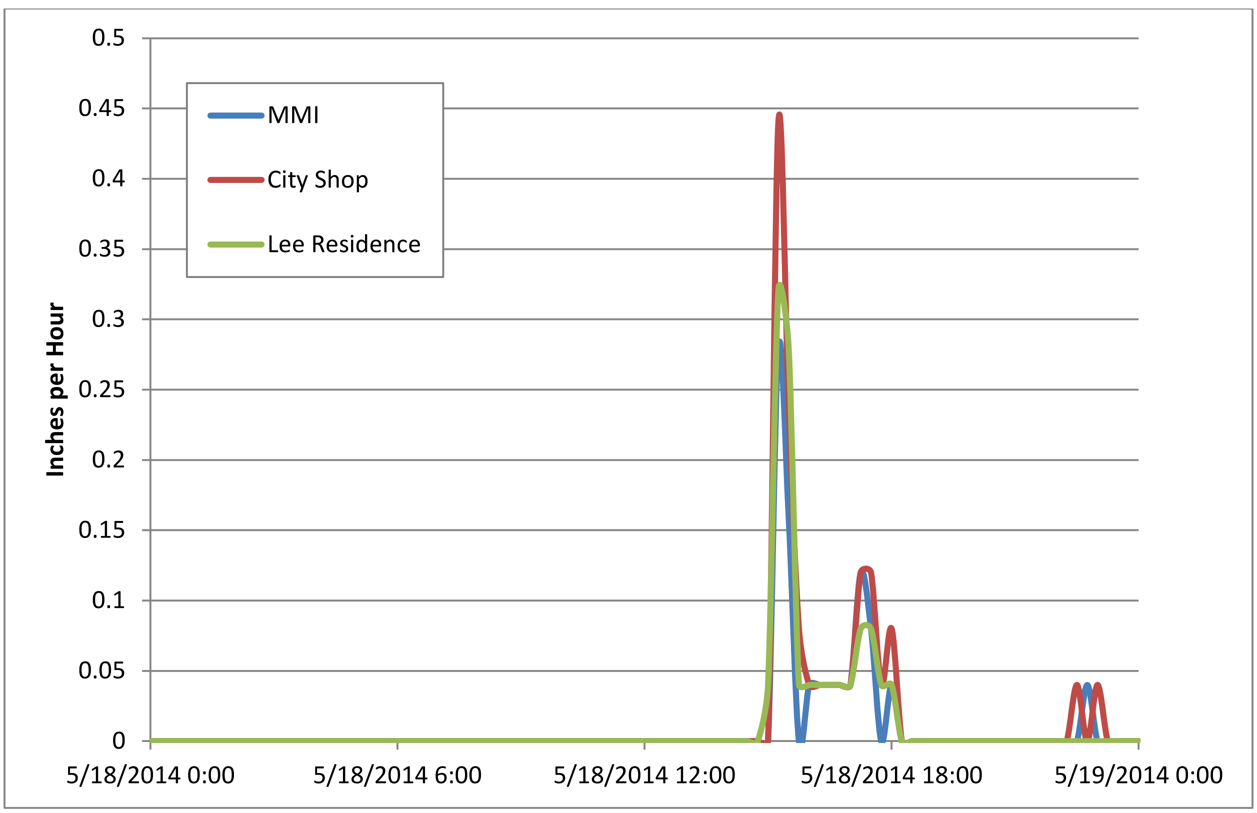
A description of the flow monitoring and rainfall equipment and data collection methods is provided in Appendix A.

4.3.3 Rainfall Analysis

During the flow monitoring time, three (3) rainfall monitors were used to collect rainfall data at the different locations within the City. This provided some spatial variability to the rainfall data collected and allowed for different rainfall profiles to be applied to different parts of the system. All of the rainfall events that occurred during the monitoring period were small events with both a short duration and low total depth. The largest event occurred on 4/28/2014 with 0.58 inches of rainfall recorded at the Lee Residence and 0.31 inches of rainfall recorded at the City Shop rain gauge over a period of 1 hour. The storm volume was lower than the 2-year recurrence interval for a 6-hour storm at the City. The storm was concentrated around the Lee Residence rain gauge and no rain was recorded at the MMI rain gauge. Due to the concentrated area and short duration of the storm, this storm was not used in the wet weather calibration or analysis.

A smaller storm occurred on 5/18/2014 that produced rainfall for 6 hours over all three rain gauges. The MMI, City Shop and Lee Residence rain gauges recorded 0.22, 0.33 and 0.28 inches of rainfall respectively during the storm event. The hyetograph recorded at each rain gauge for this storm is shown in Figure 4-4. The 5/18/2014 rainfall event was used as the basis for the wet weather calibration due to the duration and widespread spatial extents of the event.

Figure 4-4 – May 18, 2014 Rainfall Hyetograph



4.3.4 Flow Monitoring Results

The flow monitors were reviewed in order to calculate an average dry weather flow at each monitoring location as well as the response due to rainfall events in the sewer. The average dry weather flow was calculated at each flow monitor using EPA SSOAP. The software separates the flow monitoring time series into wet weather and dry weather days and then calculates an average dry weather patterns representing a typical weekend day and a typical weekday for each monitor. Table 4-3 summarizes the dry-weather results of the EPA SSOAP analysis for each flow monitor.

Table 4-3 – Dry Weather Flow Monitoring Summary

Monitor Location	Weekday Average Flow (gpm)	Weekday Peak Flow (gpm)	Weekend Average Flow (gpm)	Weekend Peak Flow (gpm)
Baxter Interceptor	518	736	503	709
North Frontage Road Interceptor	1,008	1,281	977	1,259
27th Avenue/Cattail Creek Interceptor	468	632	435	576
19th Avenue/11th Avenue Interceptor	1,054	1,433	826	1,182
WWTP	4,479	5,305	3,989	4,904

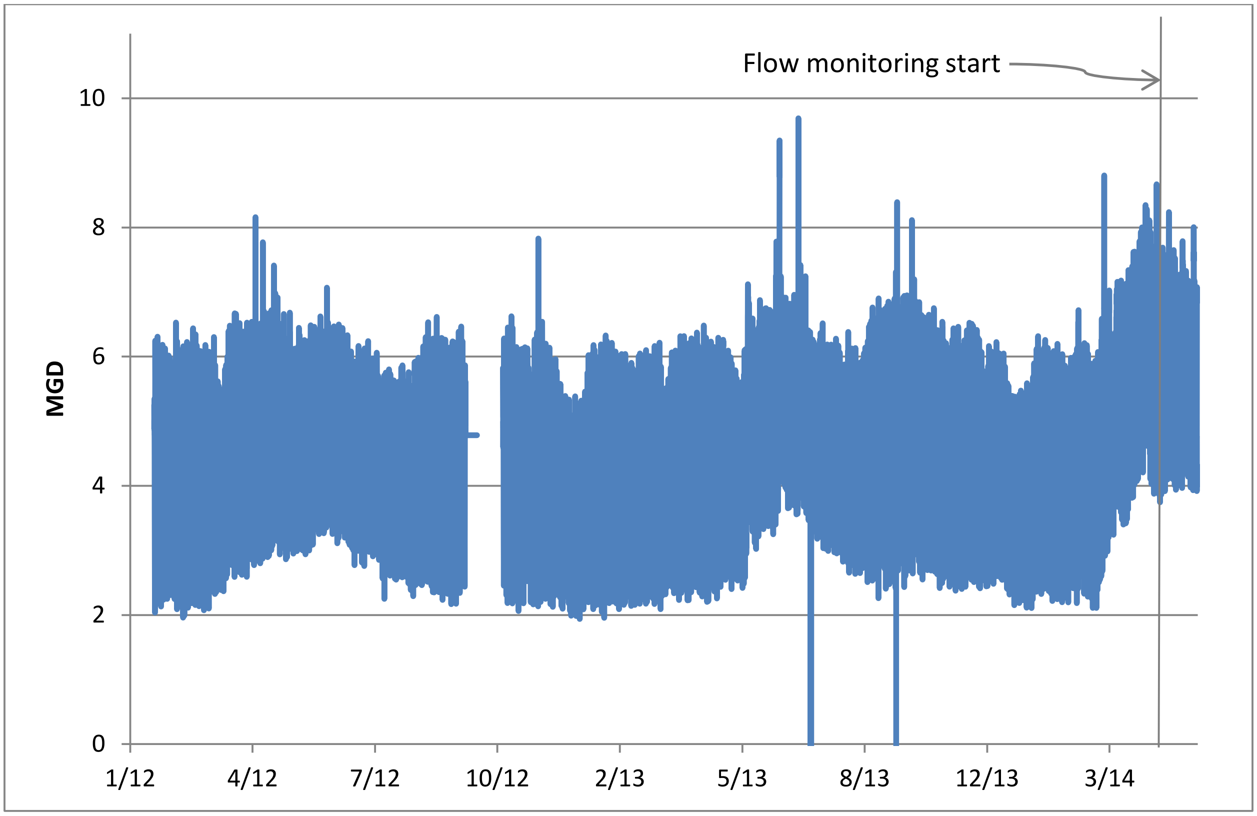
When the flows from the monitors were added together, the total flow was much less than the flow to the treatment plant. By comparing the monitoring results to the monitoring efforts from the 2007 Facility Plan, the flow in the North Frontage Road Interceptor was found to be much lower in the latest monitoring than what was found in the 2007 Facility Plan.

The monitor in the North Frontage Road Interceptor was installed 21 days after the other monitors due to equipment errors with the original monitor. The monitor also stopped recording earlier than the other monitors, so only one month of results was available. The monitor used records depth and velocity and then calculates flow from these parameters using calibration factors. There was uncertainty in the calibration factor used to calculate the flow from the depth and velocity.

The flow monitor at the influent of the wastewater treatment plant showed a large variation in seasonal flows with the base flow peaking in June and July and decreasing during the winter. This is most likely due to increased base flow from higher soil moisture content during the spring and early summer months. During the monitoring period the base inflow remained within a small range, but was higher than the previous two years. The influent flow to the treatment plant since January 2012 is shown in Figure 4-5.

The flow at each of the monitors including the treatment plant for the entire monitoring period is shown in Appendix B.

Figure 4-5 – Wastewater Treatment Plant Influent Flow



4.4 MODEL DESCRIPTION

To evaluate the capacity of the existing sewer collection system and to establish a tool for planning future expansion, an update to the hydraulic model of the sewer network was completed. The model was established using Innovyze's InfoSWMM version 12, service pack 1, update 3 sewer modeling software. The model was updated from the City's GIS data, the model data from the 2007 Facility Plan and as-built drawings of the collection system.

4.4.1 Model Development and Set-Up

System inventory was obtained by MMI/HDR from the City in GIS for import into InfoSWMM. As discussed in Chapter 3, City GIS information was compiled with as-recorded drawings and system databases for import into the collection system model. The data was imported from manhole, pipe and lift station shapefiles provided.

Once all of the available data was imported into the model, steps had to be taken to build the connectivity and topology to convert the data into a functioning model. Pump station information had to be manually entered into the model including wet well size and pump curves. Pipes had to be connected spatially to the nearest manhole and the direction of the pipes had to be verified to ensure they were starting upstream and ending downstream.

The invert elevations used in the model came from multiple sources including the inverts used in the 2007 Facility Plan for older pipes and as-built drawings for pipes installed after 2007. The City uses a unique vertical datum and not all of the inverts were on a consistent vertical datum. The datum from the as-built drawings had to be modified in order for inverts in the existing model to match the inverts in the drawing. The model had to be checked for large adverse and positive slopes that indicated a shift in the datum for the inverts. Where these issues were found, the inverts were modified to attempt to bring the entire system into a consistent vertical datum.

4.4.1.1 Roughness Parameters

The collection system model uses Manning's Equation to represent pipe roughness for open channel flow.

$$Q = \frac{1.49}{n} A R^{\frac{2}{3}} \sqrt{S}$$

where:
1.49 = Imperial unit conversion factor, ft^1/3/s
Q = Flow, cfs
n = Manning's roughness coefficient, unitless
A = Area, sq ft
R = Hydraulic radius, ft
S = Slope, ft/ft

Typically, the Manning's roughness coefficient increases with increasing pipe roughness. However, to remain consistent with the City's design standards a Manning's roughness coefficient of 0.013 was assigned to all pipes regardless of material and age. This approach for assigning roughness coefficients is used as a typical industry standard. Forcemain roughness was assigned with Hazen-Williams roughness coefficients (C value) of 120.

4.4.1.2 Sewer Flow Components

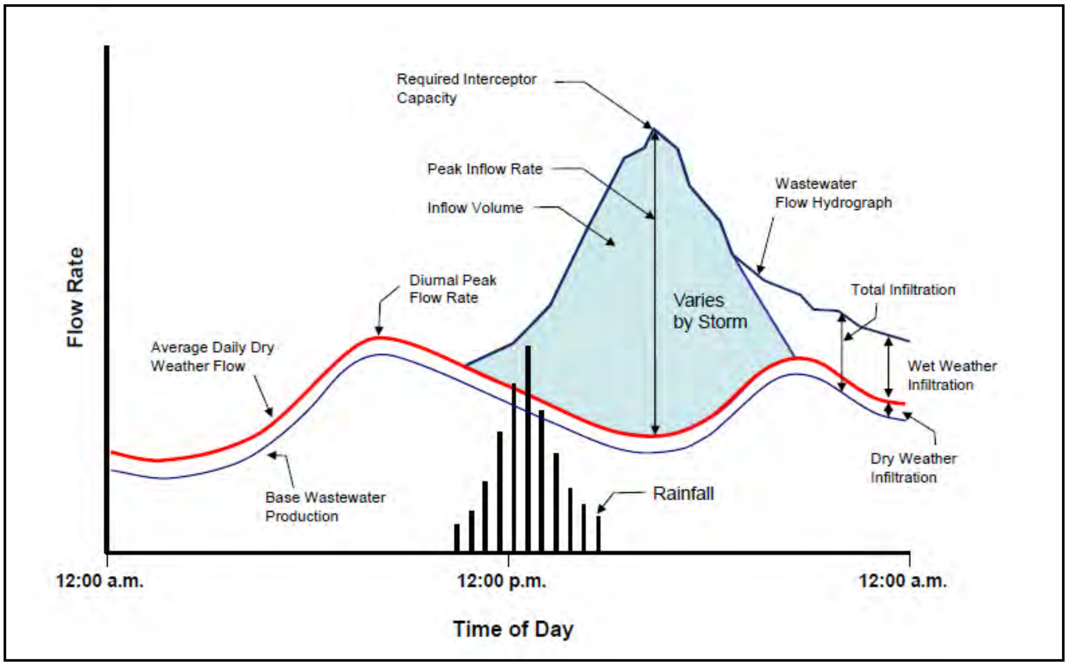
Flow hydrographs represent the fluctuation of wastewater flow over time including daily and seasonally and consist of dry- and wet-weather flow components. These components, described in more detail below, are illustrated on Figure 4-6.

Average daily dry-weather flow is comprised of base wastewater production and dry weather ground water infiltration. Base wastewater production or sanitary flow is typically directly-related to indoor water use for residential customers. Indoor water use volumes exclude outdoor water use of which the majority is for irrigation use. Therefore, indoor water use relates most closely to winter season water meter use when outdoor water uses are at a minimum. Dry-weather infiltration is groundwater that seeps into a collection system through defective pipes, pipe joints, and manhole structures below the manhole corbel and chimney during dry weather conditions. The rate of infiltration depends on the depth of groundwater above the defects, the size of the defects, the type of the soil, pipe bedding and the percentage of the collection system that is submerged.

Variation in groundwater levels and the associated infiltration is both seasonal and weather-dependent as demonstrated in Figure 4-5. Therefore, average daily dry-weather flow is the expected average wastewater flow on a day with no precipitation events and no residual influence of previous precipitation events. Average daily dry-weather flow can vary seasonally as groundwater levels change (causing fluctuations in the dry weather infiltration), depending on the snowmelt, or as the water consumption and sanitary flow change due to redevelopment or industry changes. Daily fluctuations in average daily dry-weather flow are attributable to variations in domestic and commercial wastewater production. These daily fluctuations in wastewater flows over the course of the day are represented by diurnal patterns. Finally, weekly average daily dry-weather flow variation is typical due to the weekday versus weekend water use variations.

Wet-weather flows are comprised of rainfall-derived infiltration and inflow (RDII) combined with DWF. Wet-weather infiltration is the additional rainfall-derived infiltration that occurs due to rainfall induced higher groundwater conditions and is typically seen in the hours or days following significant rain events. Inflow is rainfall-derived water that enters a collection system from sources such as private laterals, downspouts, manhole defects, foundation piping, and cross-connections with storm drains. Inflow is directly influenced by the intensity and duration of a storm event and, therefore, is not a fixed quantity. RDII is typically modeled as a peak hour wet-weather flow which is the RDII to the average daily dry-weather flow. The ratio of peak hour wet-weather flow to average daily dry-weather flow for a selected or typical rainfall event is the wet-weather peaking factor.

Figure 4-6 – Typical Components of a Sewer Flow Hydrograph



4.4.1.3 Dry Weather Flow Allocation

The City provided a shapefile with the location of water meters and billing records that included the monthly consumption from January 2012 through June 2014 for each meter. The water consumption from November through May excluding February from 2012 to 2014 was averaged for each meter to calculate the average indoor water use. February was removed due to spikes in consumption seen during the month. The average indoor water use from each meter was then assigned to the upstream manhole of the pipe that was closest to the meter. This method was used to give spatial variability to the loads based on actual consumption records.

The sanitary flow allocated based on water consumption was less than the average flow at the monitors. This is typical and the remaining flow was assumed to come from dry-weather infiltration into the system and was assigned by contributing area to each meter. The dry-weather infiltration flow was added by calculating the total length of contributing pipe and the total meter infiltration flow per foot of pipe. The flow per foot of pipe was multiplied by the pipe length for each pipe segment and added to the upstream manhole for the pipe segment.

After adding the flow from water consumption data and dry-weather infiltration, diurnal patterns were applied to the Average daily dry-weather flow to model variations in flow throughout the day. The monitors contributing manholes were given different patterns to account for the difference in land use and water use characteristics within each area.

4.4.1.4 Wet Weather Flow Allocation

Previous modeling efforts for the City of Bozeman allocated flows by increasing the base flow to account for extra inflow to match the peak inflow seen at the treatment plant. In this update to the previous modeling effort, the model was updated to calculate runoff entering the system from recorded storms.

The method used requires calculating a drainage area for each manhole and then applying rainfall to each drainage area. The drainage area for each manhole was calculated by taking the following steps:

1. Clip the drainage basins by the existing developed land use areas
2. Create Theissen polygons around each manhole within the previously clipped basins
3. Assign the contributing area to each from the Theissen polygons to each manhole

This method produced some very large drainage areas that are unrealistic for a purely sanitary sewer system. To account for this, all drainage areas greater than 5 acres were reduced to 5 acres.

4.4.1.5 Model Calibration

The model was calibrated.by comparing the model results to the monitor data under three different scenarios. The volume and peak flow as well as shape of the hydrograph was used for comparison between the model and monitor results. When the flows from the four monitors were added together, the total flow was much less than the flow recorded at the treatment plant. The monitored flows were compared to the 2007 Facility Plan flows and this showed that the flows monitored in the North Frontage Road Interceptor were reduced significantly from the 2007 Facility Plan.

Due to uncertainty in the monitor results from the North Frontage Road monitor, the peak flow and volume wasn't used at that location to calibrate the model. Instead, the general shape of the hydrograph was used for this monitor because there was high confidence in the changes in level recorded even if there was low confidence in the flow recorded.

The first scenario compared the dry weather average flow model run to the observed average dry weather flow from the flow monitoring data at all of the flow monitoring locations and at the treatment plant. The diurnal patterns applied to each contributing area were modified until the flow in the model matched the flow from the flow meters. Figures comparing the model results to the recorded flows are provided in Appendix C. Table 4-4 provides a summary of the dry weather flow calibration results.

Table 4-4 – Average Weekday Dry Weather Flow Calibration

Monitor Location	Monitor Volume (ft³)	Model Volume (ft³)	Percent Difference	Monitor Peak Flow (gpm)	Model Peak Flow (gpm)	Percent Difference
Baxter Interceptor	97,963	109,147	10%	736	770	4%
North Frontage Road Interceptor	190,117	384,003	50%	1,281	2,604	51%
27th Avenue/Cattail Creek Interceptor	88,133	86,396	-2%	632	617	-2%
19th Avenue/11th Avenue Interceptor	199,552	202,761	2%	1,433	1,422	-1%
WWTP	789,333	791,703	0%	5,305	5,700	7%

After the dry weather scenario was calibrated the existing wet weather scenario was run. The wet weather scenario was created by starting with the calibrated dry weather flow scenario and applying the 5/18/2014 rainfall event described in the rainfall analysis to all of the manhole drainage areas. With three separate rain gauges collecting rainfall data for the event, the rain gauge closest to each manhole was assigned to the manhole.

The storm used for the calibration occurred over a weekend. Because the dry weather flow was calibrated against typical weekday flows, the model was calibrated to a second dry weather scenario representing typical weekend flows before applying the wet weather event. The weekend flow was calibrated in the same way that the weekday flow was calibrated by adjusting patterns to match model flows with monitor flows. In InfoSWMM, the weekend pattern is applied on weekend days by multiplying the weekend pattern by the weekday pattern, so the weekday pattern stays the same. Based on the date the model is simulating, the model will apply the appropriate patters. Figures comparing the model results to the average weekend dry weather flows are provided in Appendix C. Table 4-5 provides a summary of the weekend dry weather flow calibration results.

Table 4-5 – Average Weekend Dry Weather Flow Calibration

Monitor Location	Monitor Volume (ft³)	Model Volume (ft³)	Percent Difference	Monitor Peak Flow (gpm)	Model Peak Flow (gpm)	Percent Difference
Baxter Interceptor	95,350	104,864	9%	709	665	-7%
North Frontage Road Interceptor	184,121	385,159	52%	1,259	2,540	50%
27th Avenue/Cattail Creek Interceptor	82,118	82,948	1%	576	520	-11%
19th Avenue/11th Avenue Interceptor	156,278	174,396	10%	1,182	1,196	1%
WWTP	701,583	757,194	7%	4,904	5,207	6%

The wet weather scenario was calibrated by modifying unit hydrographs assigned to the manholes that converted the rainfall to runoff. The unit hydrograph used for this study was an RTK Hydrograph where the unit hydrograph is created by adding three triangular hydrographs (. Each triangular hydrograph is created by providing the percentage of rainfall entering the system from the rainfall (R), the time for the flow to peak (T) and the time for the flow to recede (K*T). The RDII volumes of the three unit hydrographs are designated as R1, R2, and R3. A high R1 value indicates that the RDII is primarily inflow driven. If more of the total R-value is allocated to R2 and R3, this indicates that the RDII is primarily infiltration driven.

The system manholes were split into seven different areas based on which monitor the drainage area drains to and which rain gauge the manhole is closest to. The seven areas were all assigned different hydrographs. The scenario was calibrated by modifying the unit hydrographs until a reasonable match was seen between the modeled flow and the monitored flow.

The monitors showed the peak flow occurring before rainfall was recorded at the rain gauges. All of the rain gauges were located in the southern portion of the system and all of the monitors were in the northern portion of the system. The peak shown in the monitors are most likely due to intense rainfall in the northern portion of the system that was not recorded by any of the rain gauges. This limited the ability to match the runoff experienced by the system in the model and more effort was put into matching the peak flow and volume of runoff produced by the storm. Figures comparing the model results to the recorded flows are provided in Appendix A. Table 4-6 provides a summary of the wet weather flow scenario results used for the calibration.

Table 4-6 – Wet Weather Flow Calibration

Monitor Location	Monitor Volume (ft^3)	Model Volume (ft^3)	Percent Difference	Monitor Peak Flow (gpm)	Model Peak Flow (gpm)	Percent Difference
Baxter Interceptor	83,571	87,026	4%	699	618	-13%
North Frontage Road Interceptor	190,684	400,597	52%	1,614	2,686	40%
27th Avenue/Cattail Creek Interceptor	111,134	109,546	-1%	826	792	-4%
19th Avenue/11th Avenue Interceptor	193,120	181,270	-7%	1,419	1,484	4%
WWTP	748,248	789,572	5%	5,559	5,801	4%

4.5 *EXISTING SYSTEM EVALUATION*

The existing system model was calibrated with the goal of matching the peak flows and total volume at each of the monitoring locations within 10% for dry weather flows and wet weather flows. The calibration results were generally within 10% of the monitor results and the model is a good indicator for the system performance for the purpose of this study. The model was used to stress the system to locate and address capacity constraints within the existing system.

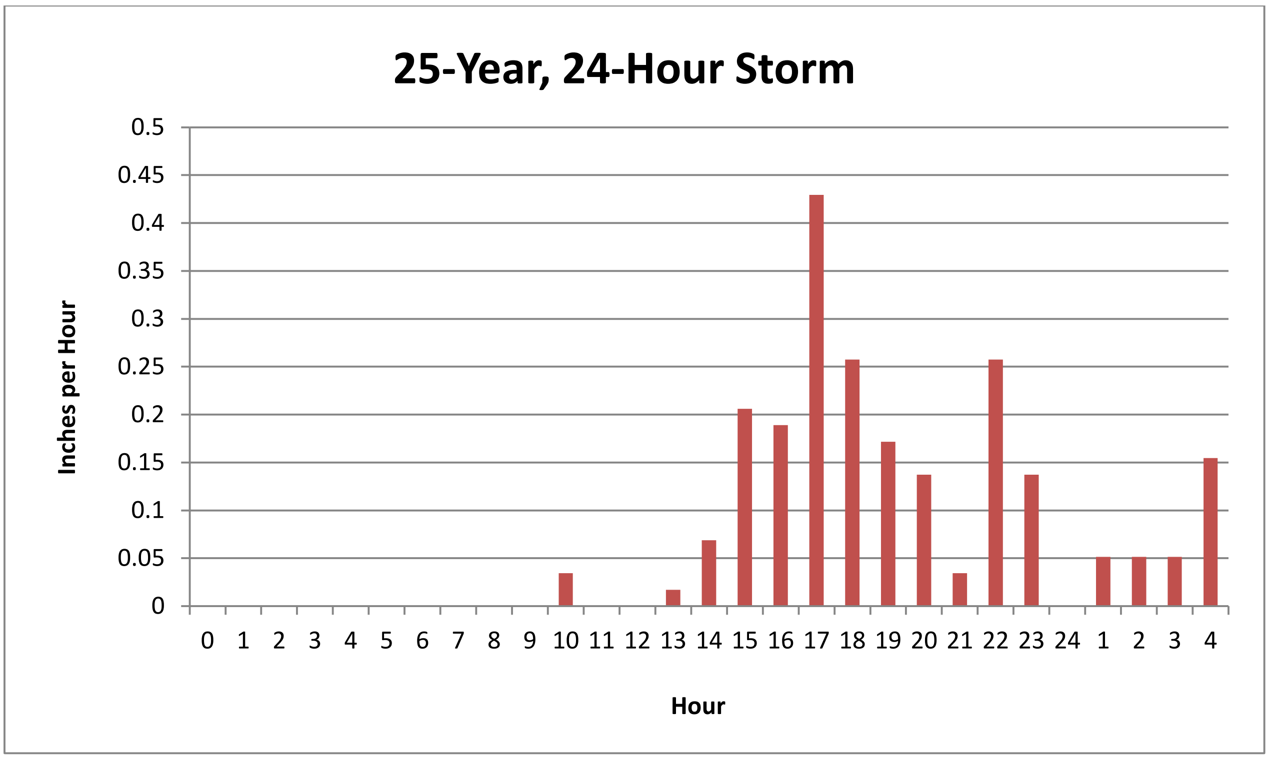
The storm used to calibrate the hydraulic model was not large enough to produce significant runoff and could not be used to check the capacity of the system. To check the capacity of the existing collection system a 25-year, 24-hour rainfall event was applied to the model. The 25-year event was chosen to be consistent with the City's stormwater design standards and provide the same level of service as the stormwater system. This event produced much higher rainfall and was used as the basis for the system evaluation.

4.5.1 System Evaluation Procedure

No significant rainfall event occurred during the monitoring period but a large rainfall event occurred during the previous plan evaluation period. On June 10, 2004, a rainfall event produced 1.31 inches of rainfall over the City. The City does not have a standard design storm recurrence interval that it uses to design the sanitary sewer system, but the City does have a standard for storm sewers. The City uses a 25-year return period to size storm sewers, so this return period was chosen to evaluate the capacity of the sanitary sewer.

According to National Oceanographic and Atmospheric Administration (NOAA) climatic information from the Western Regional Climatic Center, a 25-year, 24-hour rainfall event for the Bozeman area is 2.25 inches. To convert the 1.31 inch storm that occurred on June 10, 2004 into a 25-year storm, the rainfall hyetograph was scaled up to produce 2.25 inches of rainfall to match the 25-year rainfall depth for the City. The most intense 6 hours of the storm totals 1.39 inches of rainfall that is slightly above the 6-hour, 25-year rainfall depth of 1.37 inches for the City. The rainfall was shifted from the original rainfall hyetograph to occur during the afternoon peak in the typical diurnal pattern. The resulting hyetograph, shown in Figure 4-7, was applied to the entire system to evaluate the capacity of the system. The model results from this storm event represent the peak wet-weather design flows.

Figure 4-7 – 25-Year, 24 Hour Hyetograph



The collection system was evaluated to find deficiencies based on a ratio of depth of flow to pipe diameter (d/D) in excess of 0.75 based on the City's design criteria. This indicates capacity issues in the collection system.

4.5.2 Existing System Evaluation Results

The system did not exhibit capacity deficiencies at average dry-weather flows or at peak wet-weather design flows. With the 25-year storm applied to the model, the peak flow at the treatment plant was 13.9 MGD. This flow is higher than any flow recorded at the treatment plant and higher than the peak flow of 9.7 MGD seen since January 2012. These results are also consistent with the City not observing any wet weather related sanitary sewer overflows (SSOs) since the 2007 Facility Plan.

The highest confidence in the model results was in the interceptors in the vicinity of the monitoring locations and confidence is reduced upstream in the trunk and collector sewers. The capacity of the interceptors and the capacity modeled depend on the slope of the interceptor. There is high uncertainty in the elevations used for inverts in the model due to various vertical datum values used over time in new development areas within the City. As information is collected for invert elevations within the collection system, the model should be reevaluated for capacity.

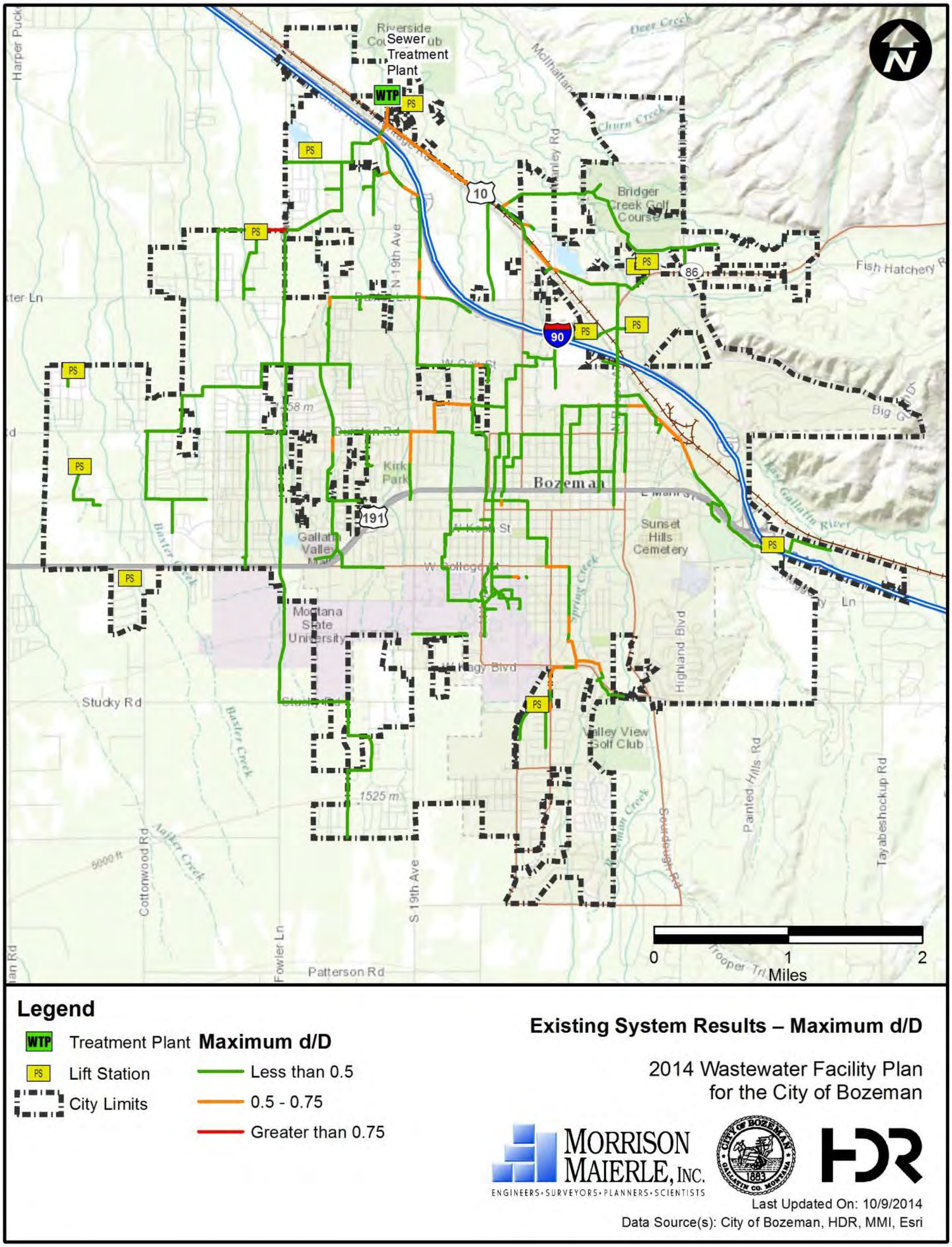
The previous study showed the North Frontage Road Interceptor to be near capacity. This study shows the interceptor to be the closest to capacity. Table 4-7 compares the capacity of the interceptor to the peak wet weather design flow for each interceptor. Figure 4-8 shows maximum depth divided by the diameter for all of the trunk and interceptors sewers from the 25-year storm simulation.

Table 4-7 – Existing Interceptor Results

Interceptor	Size	Capacity (MGD)	Peak Design Wet Weather Flow (MGD)
Rouse	30 24	14 8	3.7
North Frontage Road	30 Parallel 20 20	5 7 8	4.6
19th Avenue/11th Avenue	24	10	7.2
Baxter	24 21 20	4 4 3	2.4
27th Avenue/Cattail Creek	27 24	10 9	2.0
Evergreen	21	4	0.7
Davis-Fowler	18 24	5 13	1.3
WWTP	30	19	13.9

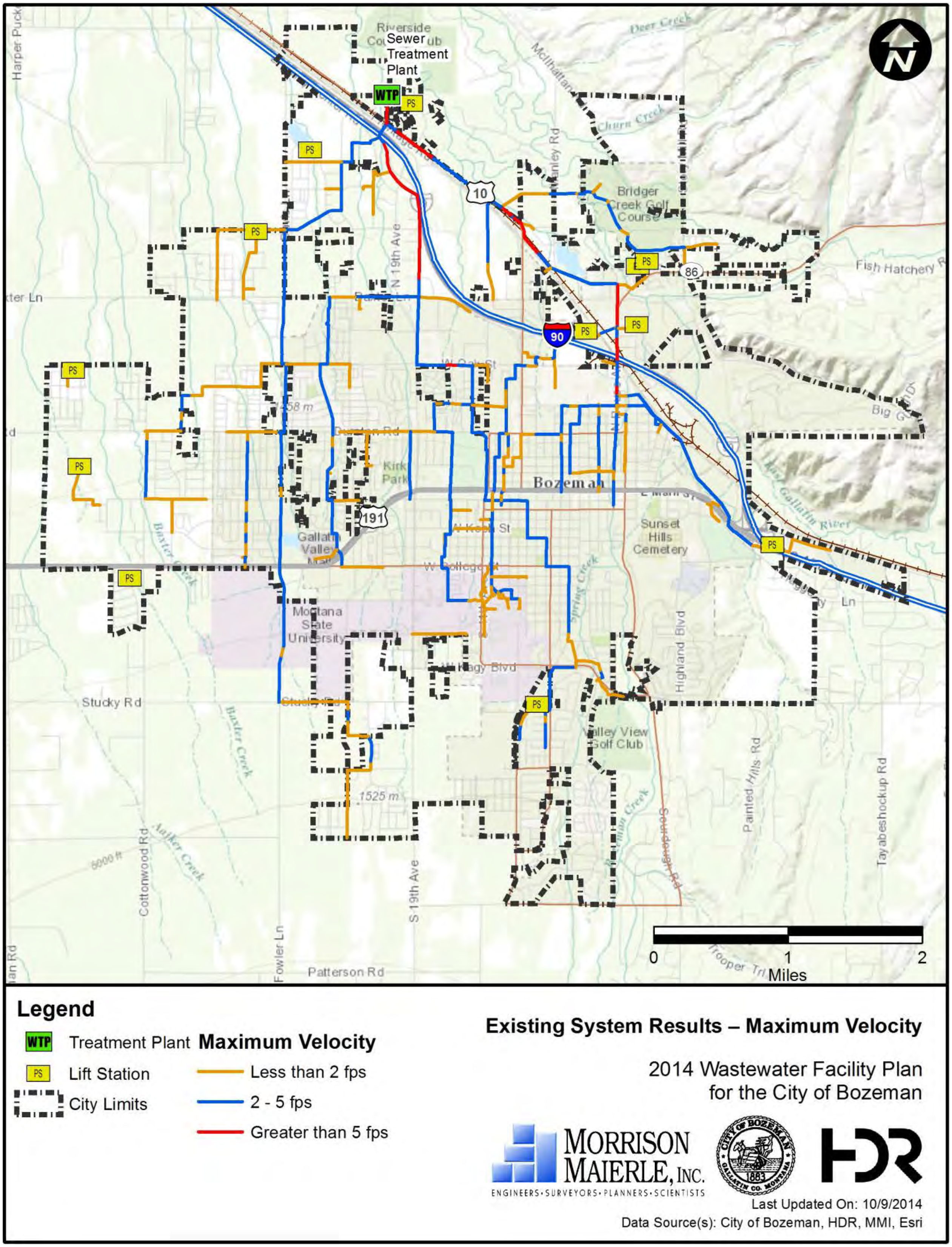
One consideration that should be made is that the model assumes the diameter of each pipe is accurate based on GIS data provided by the City. The internal diameter may be significantly smaller than the reported diameter and this may cause capacity constraints. This is especially true in large diameter PVC and HDPE pipes as well as pipes that have been lined. If there are pipes with reduced internal diameters, this information should be updated in the model to see if the reduced capacity will cause a bottleneck in the system and possible overflow.

Figure 4-8 – Existing System Results – Maximum d/D



The velocity in the system was not used in the evaluation of the existing system as the low and high velocity limits create maintenance problems that are not a part of this study. The velocity calculated from the modeling effort can be used in the City's maintenance program to find areas of the system that should be regularly cleaned due to low velocity and checked for structural problems due to high velocity. Figure 4-9 shows the maximum velocity in the system during peak dry weather flow conditions for all trunk and interceptor sewers. Pipes with velocity greater than 5 fps should be examined for excessive scouring and pipes with less than 2 fps should be examined for settling that could cause reduced capacity.

Figure 4-9 – Existing System Results – Maximum Velocity



4.5.3 Existing Lift Station Evaluation

The lift stations were evaluated by comparing the peak inflow design wet weather inflow to the lift station to the capacity of the lift station. All of the lift stations had enough capacity to pump the peak wet weather design flow and no deficiencies were identified. A comparison of the average dry weather flow, the peak design wet weather flow and lift station capacity is shown in Table 4-8.

Table 4-8 – Existing Lift Station Results

Lift Station	Capacity (gpm)	Average DWF (gpm)	Peak WWF (gpm)
Baxter Meadows	690	69.7	246.3
Bridger Center	100	0	0
Burrup	450	36.8	86.2
Cattail Lake	225	16.8	156.5
Laurel Glen	450	62.6	156.5
Loyal Gardens	364	13.9	40.7
Norton Ranch	121	9.5	74.3
Overbrook	Unknown	1.3	6.1
Walker	Unknown	7.3	11.3
Links	160	2.4	4.2
Cardinal Distribution	Unknown	0	0

The lift stations are evaluated further in Chapter 5 relative to their current dedicated service area. In addition, the lift stations will be evaluated further to compare the costs of creating larger regional lift stations, as proposed in the 2007 Facility Plan, to smaller local lift stations similar to the lift stations that have been built since the 2007 Facility Plan.

4.5.4 Existing I/I Reduction Recommendations

The City has performed extensive rehabilitation of pipes in the oldest portions of the system in order to reduce inflow and infiltration (I/I) into the system. The City should continue with the recommendations from the previous master plan to reduce I/I into the system. Specifically, the City should take the following steps:

- Enforce existing ordnances to prevent crawl space sump pumps from discharging to the sewer system.
- Rehab or replace old clay pipe in the older parts of the City
- Continue program to CCTV and record approximately 20-percent of the existing collection system each year to identify specific problem areas associated with the aging pipe.
- Establish a dedicated program to inspect approximately 20-percent of their existing manholes each year to identify specific problem areas associated with the aging manholes.

Each pipe or manhole with a deficiency can require a different solution from a point repair or lining to replacement of the pipe. When a pipe is inspected, repaired, lined or replaced, this information should be tracked within GIS and include information such as the new internal diameter if lining, new material if replaced and condition if inspected.

4.6 *CONCLUSIONS*

The existing collection system was evaluated for capacity by creating a model of the collection system. The model was calibrated against flow monitoring data collected during the spring of 2014. The model was calibrated to typical dry weather flows experienced during the monitoring period as well as wet-weather flows seen on 5/18/2014. A 25-year rainfall event was applied to the calibrated model to calculate a design flow in the system to the compare to the capacity of the pipes in the system. This analysis showed that the existing system has enough capacity to convey the existing design flows in the system. The analysis also showed that all of the lift stations had enough capacity to pump the existing flows to the lift station.

Appendix A – Flow Monitoring Memorandum

MORRISON MAIERLE, INC.
An Employee-Owned Company

memo

TO: **James Nickelson**

FROM: **Brian Wainright**

RE: **City of Bozeman wastewater flow monitoring**

DATE: **September 25, 2014**

INTRODUCTION

This memo is intended to outline the wastewater flow monitoring activities performed between April 8th and June 10th 2014. The project entailed the installation and monitoring of four open channel flow meters at previously determined sites and the installation and monitoring of three tipping bucket type rain gauges. The flow meters and gauges were periodically downloaded and the data examined. The locations of both the monitored manholes and the rain gauges are presented on Figure 1.

WASTEWATER FLOW MONITORING

Four sites were identified by MMI and HDR staff and each was inspected by city staff to confirm suitable access and conditions for open channel flow measurements. One flow meter would be installed at each location. The city provided one; a Hach Flow-Tote 3, HDR provided an ISCO 2150 Area Velocity Flow Module and two Hach Flow-Dar sensors coupled with FL-900 data loggers were rented directly from Hach. The Flow-Tote 3 was ultimately replaced with an additional FlowDar meter and data logger due to communications failure within the Flow-Tote 3. Each unit was programmed to record measurements on one minute intervals. A summary of each monitoring site is presented below in Table 1 along with a listing of meter type by location.

Table 1. Wastewater flow monitoring locations and equipment.

Manhole	Nearest	Montana State Plane - International Feet			Flow Meter	Data Date
ID	Cross streets	Easting	Northing	Invert Elevation	Utilized	Range
I0105	Baxter and Commerce	1569053.823	532790.3198	4692.25	Flo-Dar	4/8 - 6/10
H0204	East of 19th (Schnee's)	1569506.837	531918.8818	4698.7	Flo-Dar	4/8 - 6/10
I5014	Frontage and Campbell	1569474.557	538797.7261	4630.71	Flo-Dar*	4/23 - 6/10
J0002	Catron and 27th	1566591.148	537335.6686	4647.64	ISCO 2150	4/8-5/8, 5/12-5/21 & 6/1-6/10

*Installed on April 23, 2014 as replacement for Flow-Tote 3.

The three Hach Flow-Dar units utilized ultrasonic depth measurements coupled with radar velocity measurements. These units were equipped with cellular antennas and were programmed to transmit data on regular intervals which was subsequently available on the Hach FSData website. The replacement unit cellular connection did not function throughout the monitoring period despite coordination with Hach technical support and numerous efforts to initiate service. It was manually downloaded in the field during site visits and at removal. The

Hach Flow-tote 3 and the ISCO 2150 meters each use absolute pressure to measure depth and radar to measure velocity. Each logs data to a data logger for physical transfer via communications cable to a computer. Each sensor was installed on April 8, 2014 utilizing each manufacturers installation and calibration instructions. In general calibration included physical measurements of both level (depth) and velocity. Followed by a process (varying between the three meter types) to calibrate the readings taken by the device to the measured values. The depth measurements were taken initially as in-stream measurements utilizing at first a staff gauge, then with a tape measure and a piece of lathe. In the second method the latch was paced in the flow with the narrow side facing the flow, this minimized the wave and the technician visually estimated the height of the wave, this was then deducted from the wetted length of lathe. These methods were problematic due to relatively high velocities and creation of a wave on the staff or lathe that masked actual level, but it is believed the levels were within approximately 5-hundredths of a foot.

Velocity measurements were taken utilizing a Global Water Flow Probe Hand-Held Flowmeter. The measurements were taken per the meters' instructions for pipe flow by slowly moving the meter from the bottom of the pipe to the surface, then back down, repeating for several minutes. The meter averages the flow across the pipe and that value was recorded. Flow conditions within the channel were typically somewhat laminar though surface variations were present. Measurements for both velocity and depth were problematic; with velocity being limited by the solids content in the fluid (solids and/or fibrous material stopping the measurement propeller) and depth simply variations in fluid surface and identifying the true "bottom" of the channel. It is estimated that depth measurement accuracy was approximately +/- 0.05 feet and velocity +/- 0.5 fps.

Each site was visited again on April 15th and depth measurements were retaken utilizing a staff gauge and a difference measurement between the bottom of the channel to the top of the man-hole minus the top of the flow to the top of the man-hole measurements. Velocity measurements were taken as previously noted. The devices were each re-calibrated to the more precise flow depth. Previously existing data for each site was automatically recalculated using the new measurement. It is estimated that the difference measurement increased the accuracy of the depth measurements to between +/-0.02 and +/-0.03 feet depending on conditions at the time of measurement. Measurements were taken periodically throughout the monitoring period but no further changes to the calibrations were made.

During the April 15th site visit communications could not be made between the City's Flow-Tote 3 and several different computers. HACH technical support was contacted it was ultimately decided that a failure had occurred within the data loggers communications system. Consultation with James Nickelson and Bob Murray indicated it was appropriate to rent an additional FlowDAR from Hach. That unit was ordered April 17th and installed on April 23rd. It was not shipped with a cellular antenna and was thus installed without remote communications capabilities while an antenna was shipped from Hach. Ultimately remote communications were never achieved and the unit was physically downloaded during site visits. Numerous attempts were made to gain remote access including several conversations with Hach technical support and it was ultimately their belief that the cellular card was malfunctioning or that the cellular provider was not allowing communications due to ongoing contracting issues with Hach.

Site visits were originally planned every week for several weeks to gain information on battery life. It was the original intent to minimize site visits as the monitoring program proceeded. Given the initial issues with the monitors in manhole I5014 weekly visits were required until the second meter was found to be functioning properly. During this time data was analyzed and manuals for each monitor consulted to determine a return frequency to ensure adequate battery life and data retention, specifically for the ISCO 2150 as it was had the shortest life and smallest storage capacity. Each occurring due to limited data storage and programmed parameters to over-write old data. The error occurred as the manual lists the device is capable of logging storing approximately 70000 readings (totaling approximately 50 days of readings), but as it turns out each parameter is considered a reading therefore its data capacity was significantly less. The logger logs level, velocity and input voltage and calculates flow rate and cumulative flow; reducing the available storage to about nine days. The result is the two periods of missing data: From May 8th to May 12th and May 21st to June 1st. Data from the Flow-Tote3 was never recovered; therefore no data exists for that location from April 8th to April 23rd.

RAIN FALL MONITORING

Three rain-gauges were placed at secure sites strategically placed around the city; one at the City Shop, one on the roof of the MMI building and one at Bob Lee's residence (an MMI employee) on the south side of town. The three sites were chosen to be representative of city wide precipitation events and were relatively safe from tampering or vandalism.

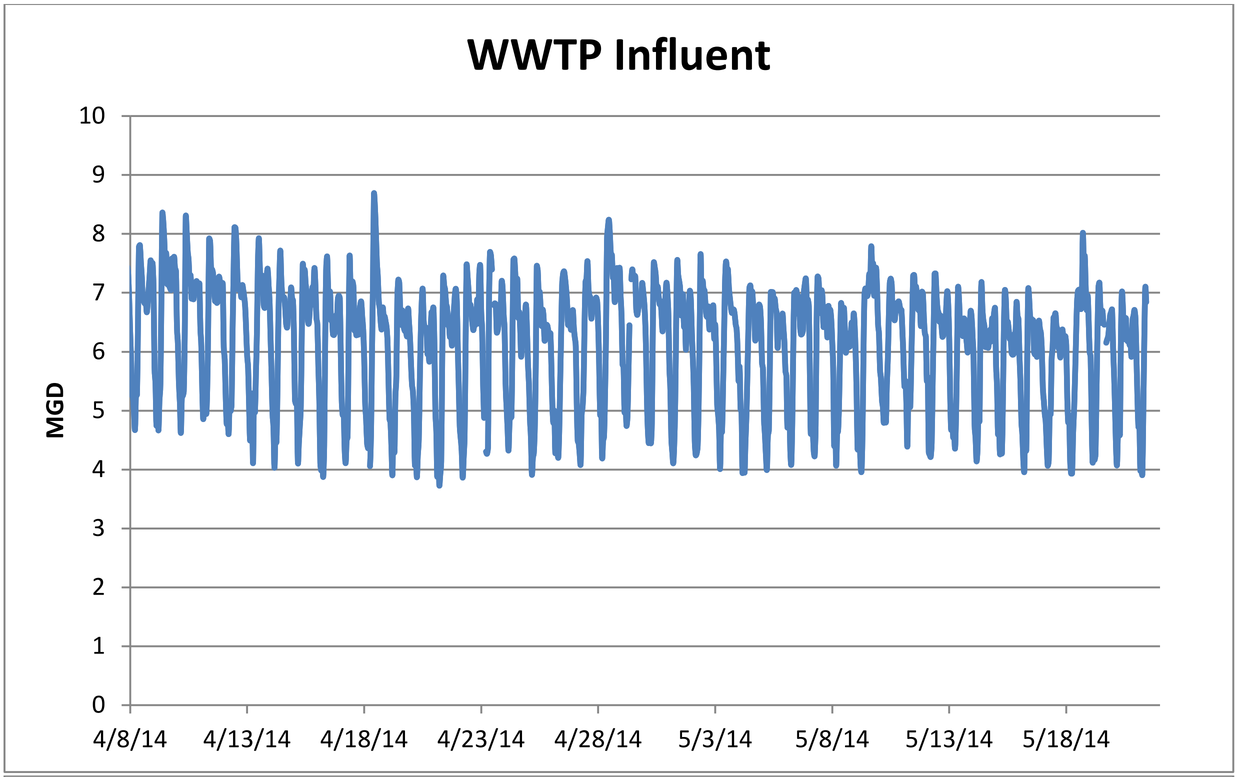
The rain gauges were each Onset Model RG3 tipping bucket rain gauges with Hobo Pendant event loggers. The gauges were placed on April 9th each in an open area, free from obstructions. Each logger was programmed to record data on two minute intervals; however the logger automatically records the date, time and temperature at each bucket tipping event so for rainfall data was recorded on a real time basis and was resampled on a one minute scale. One bucket tip equates to 0.01inches of precipitation, therefore the data was presented as hundredths of an inch for each one minute time period. The data was continuously recorded throughout the monitoring period for each site. As mentioned earlier the data loggers also recorded temperature, however solar radiation shielding was not utilized and the temperature data is therefore not reliable and was not presented in the data summary.

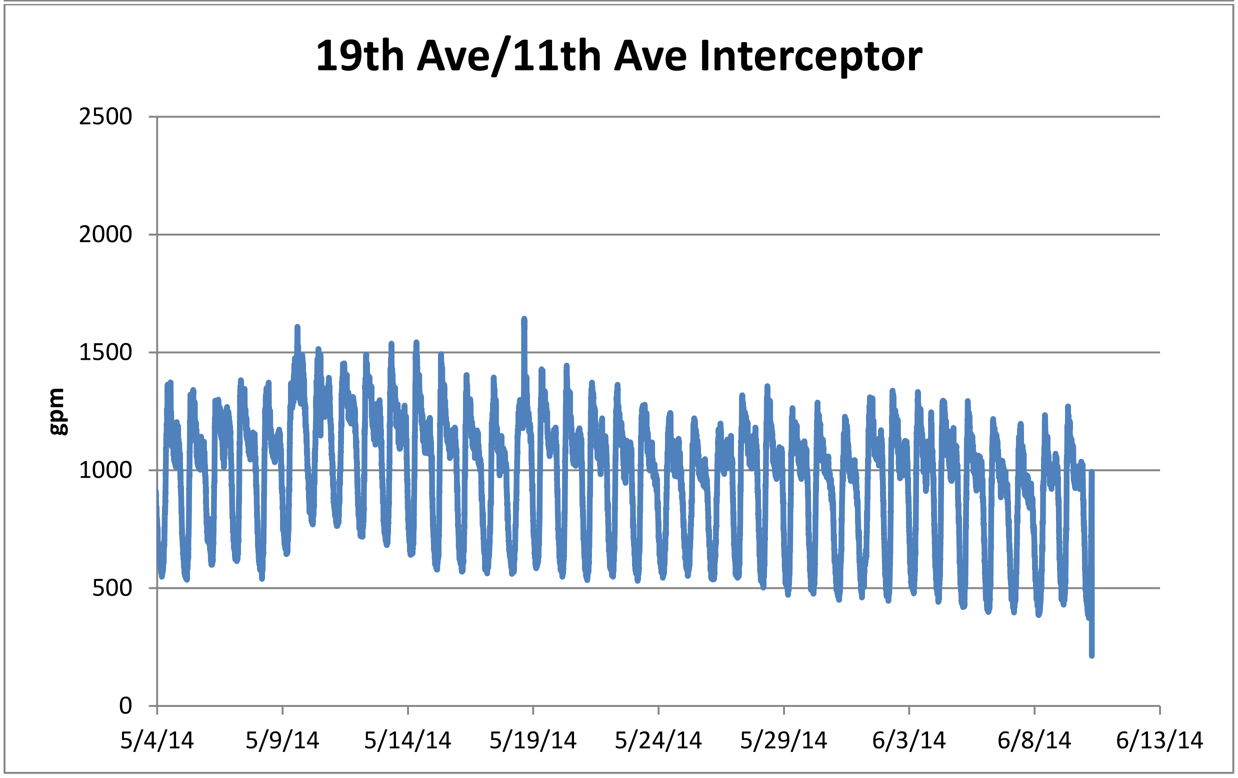
Sincerely,
Morrison-Maierle, Inc.

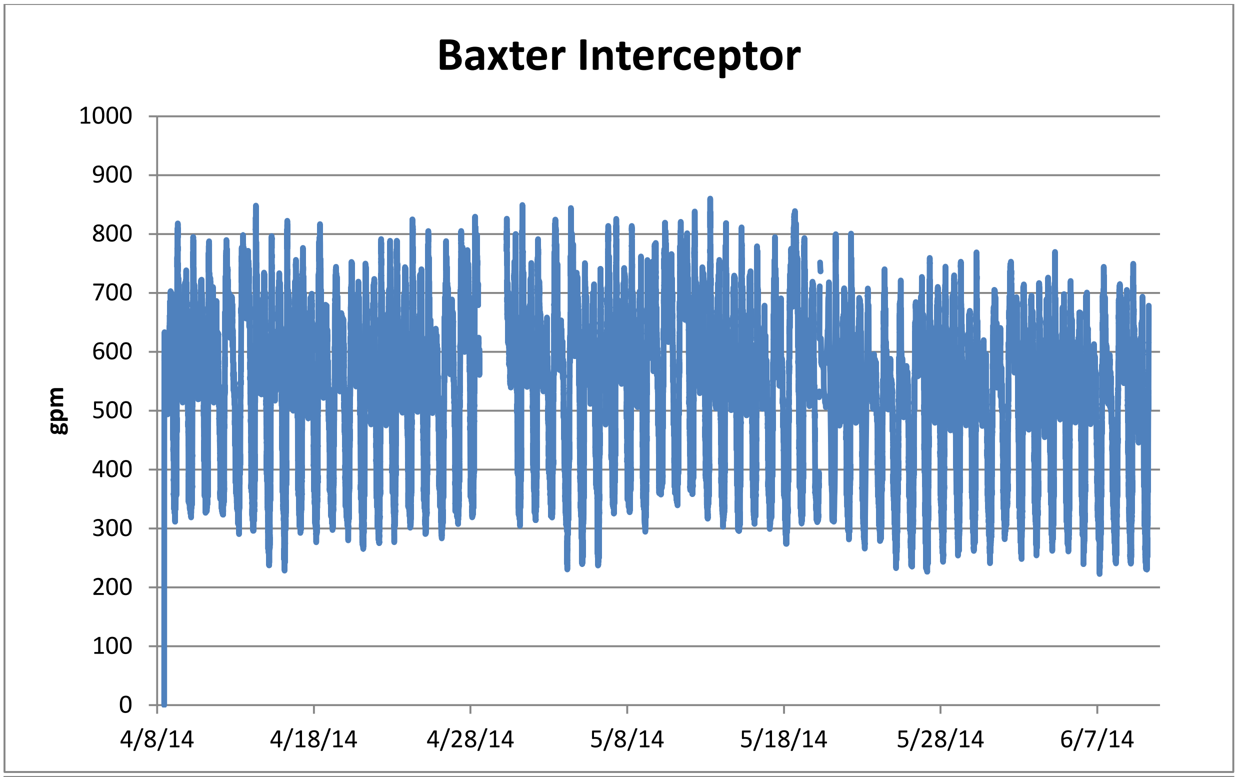
Brian Wainright

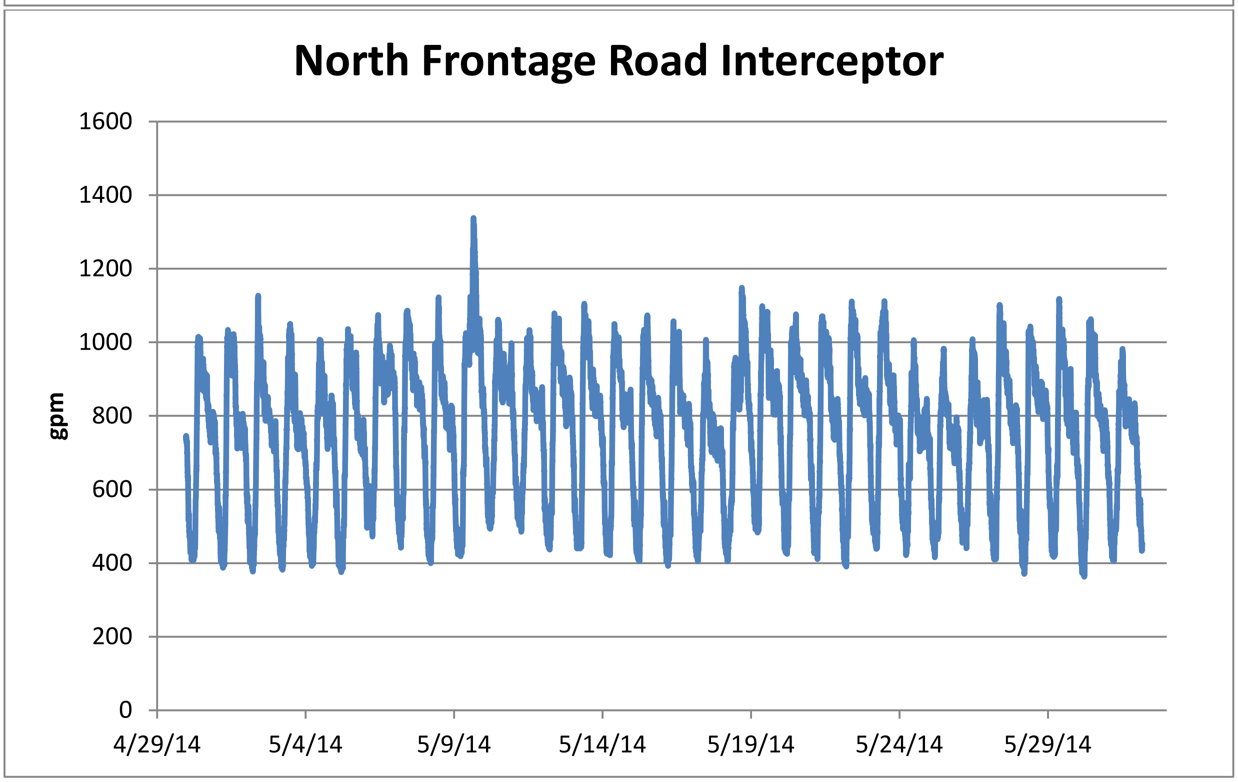
Appendix B – Flow Monitoring Results

The following figures in Appendix C show the flow monitor hydrograph for the entire flow monitoring period.






Baxter Interceptor
gpm
1000
900
800
700
600
500
400
300
200
100
0
4/8/14
4/18/14
4/28/14
5/8/14
5/18/14
5/28/14
6/7/14


North Frontage Road Interceptor
gpm
1600
1400
1200
1000
800
600
400
200
0
4/29/14
5/4/14
5/9/14
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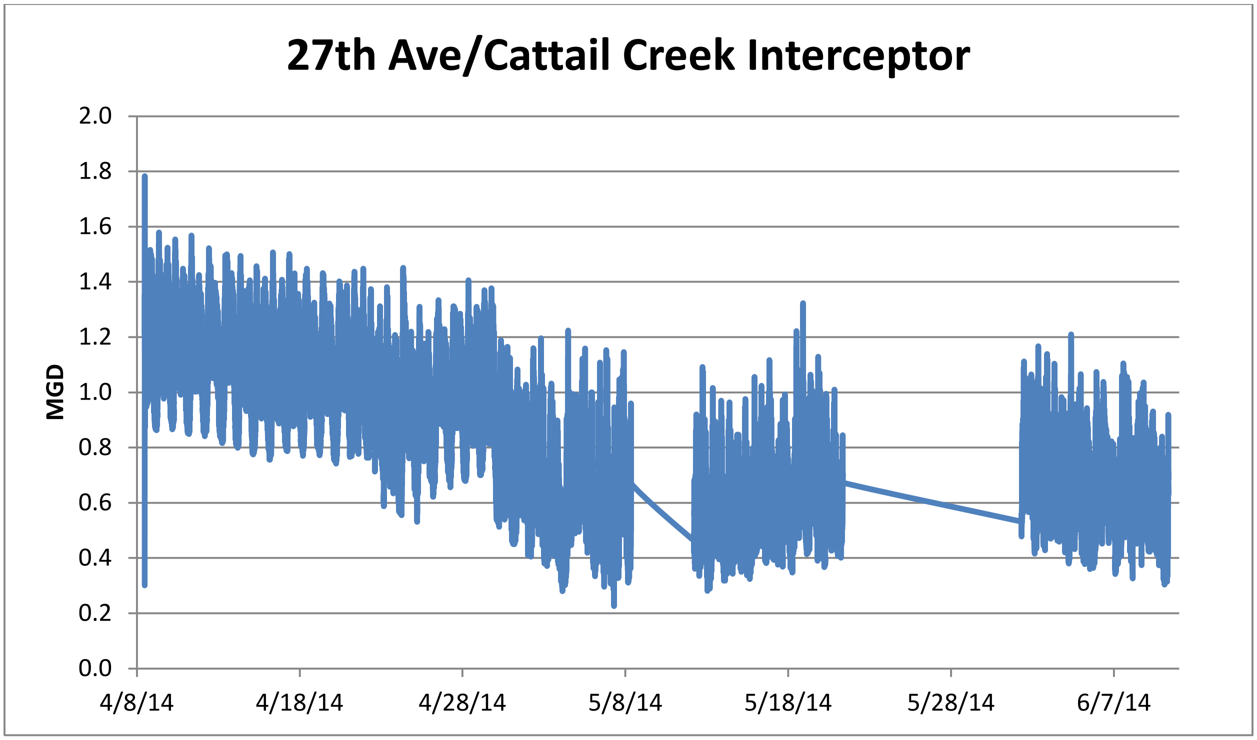
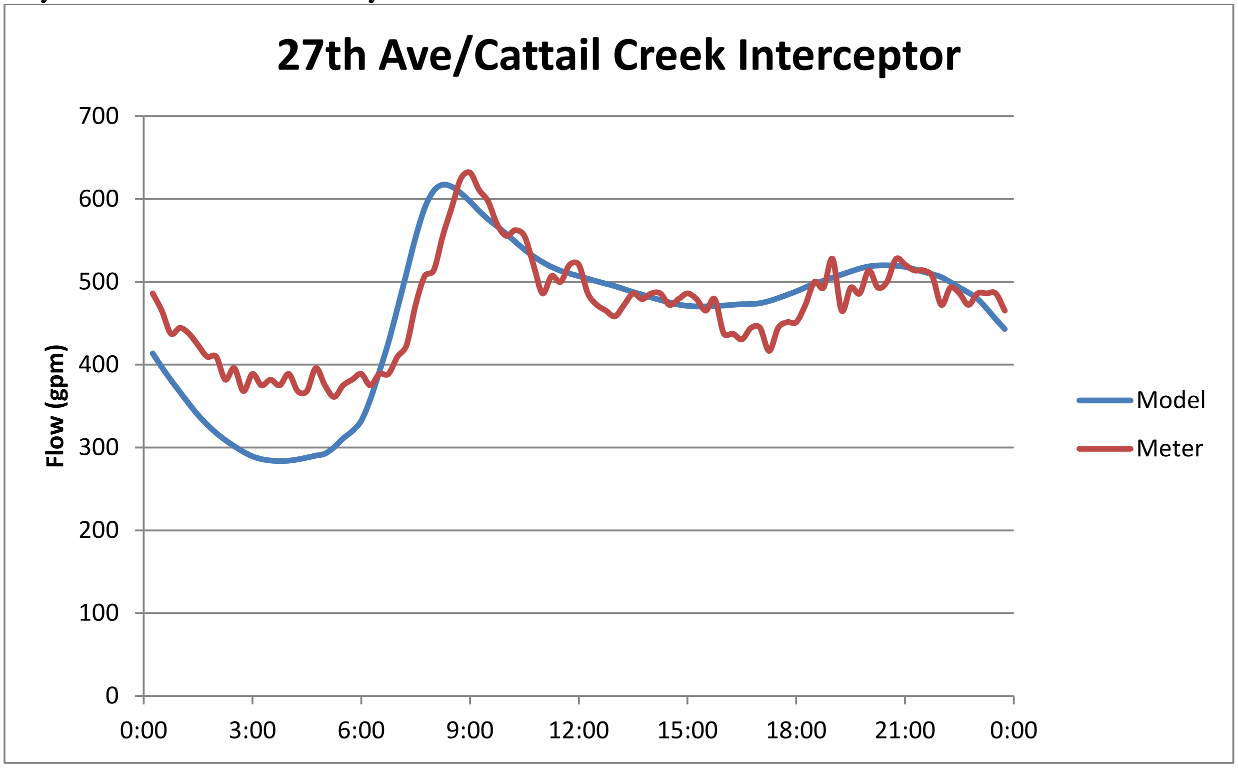
5/19/14
5/24/14
5/29/14

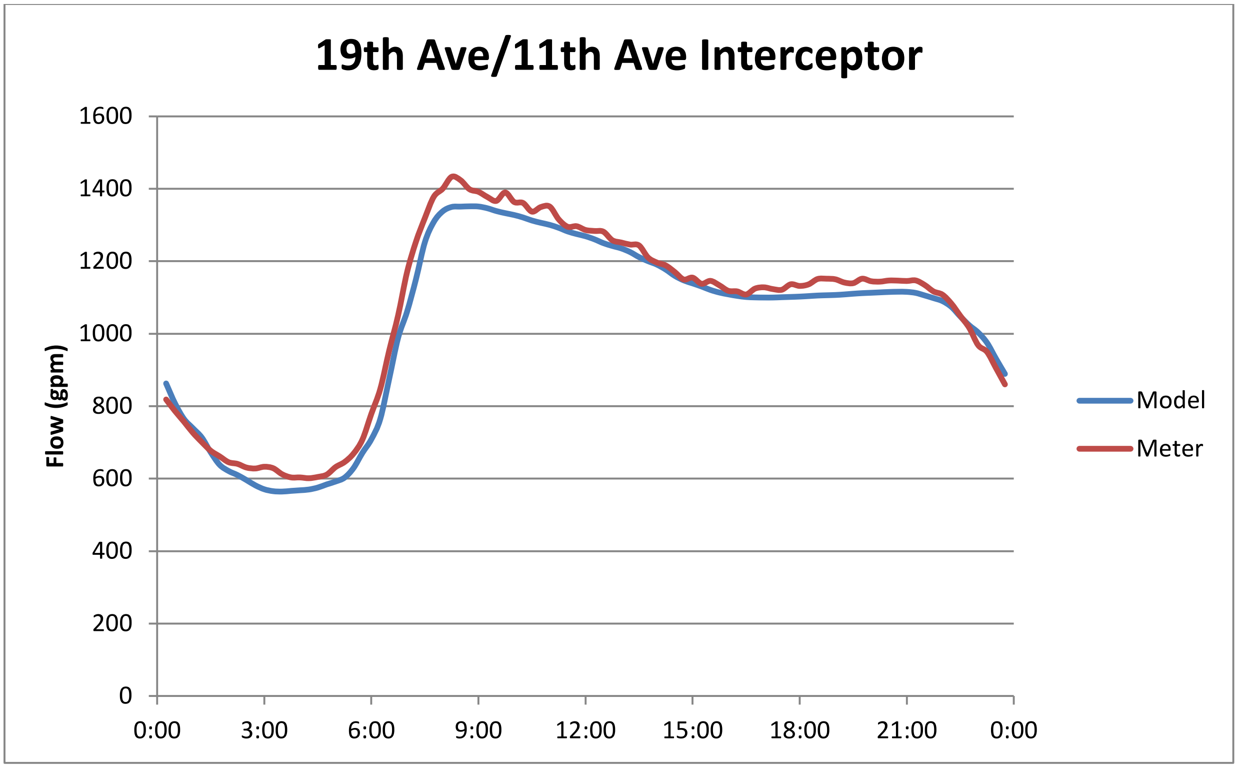

27th Ave/Cattail Creek Interceptor
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
MGD
4/8/14
4/18/14
4/28/14
5/8/14
5/18/14
5/28/14
6/7/14

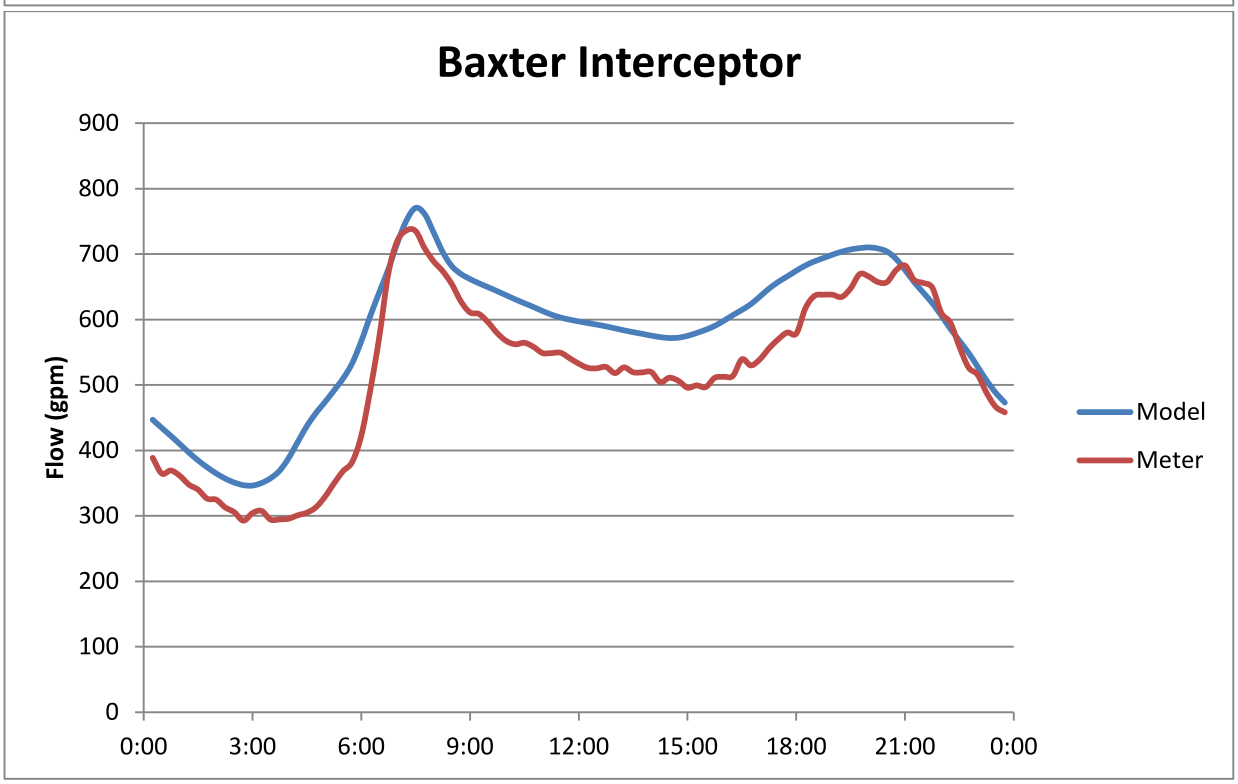
Appendix C – Model Calibration Hydrographs

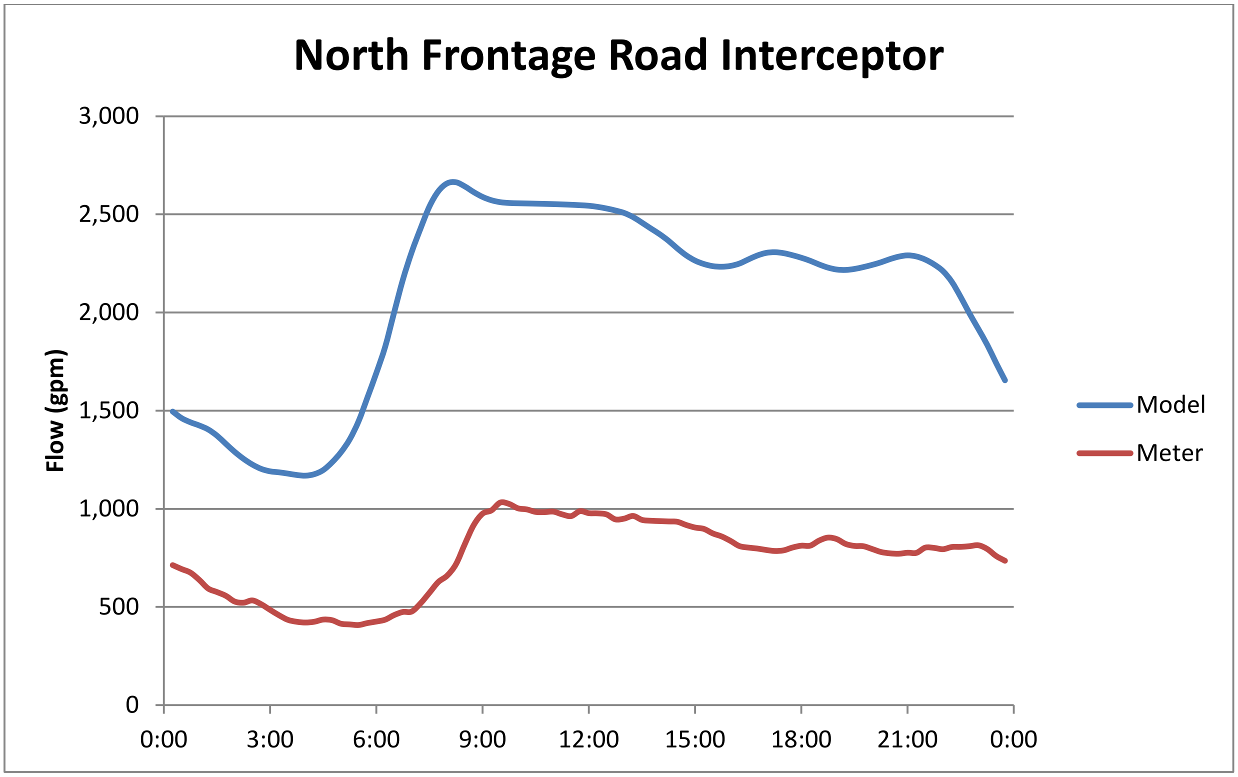
The following figures in Appendix C show the comparison of the flow monitor hydrograph to the modeled hydrograph at each of the monitoring locations.

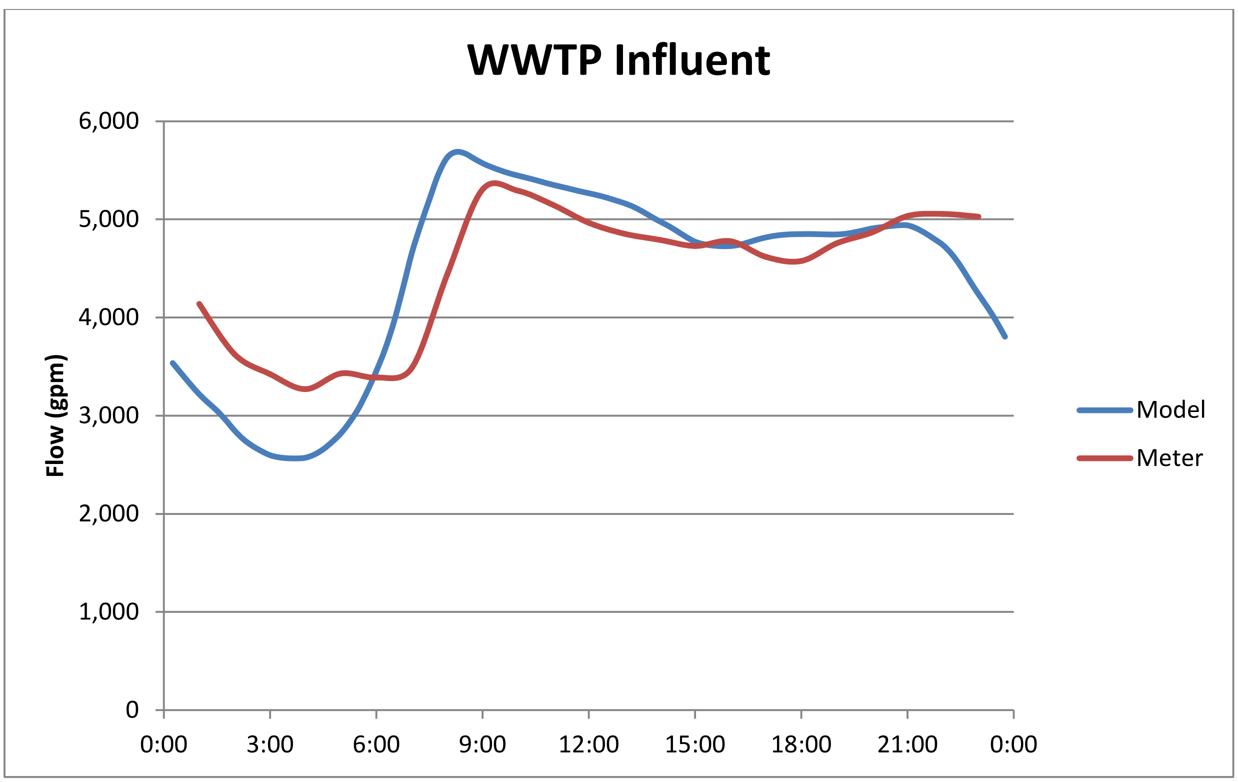
Dry Weather Flow Weekday Calibration



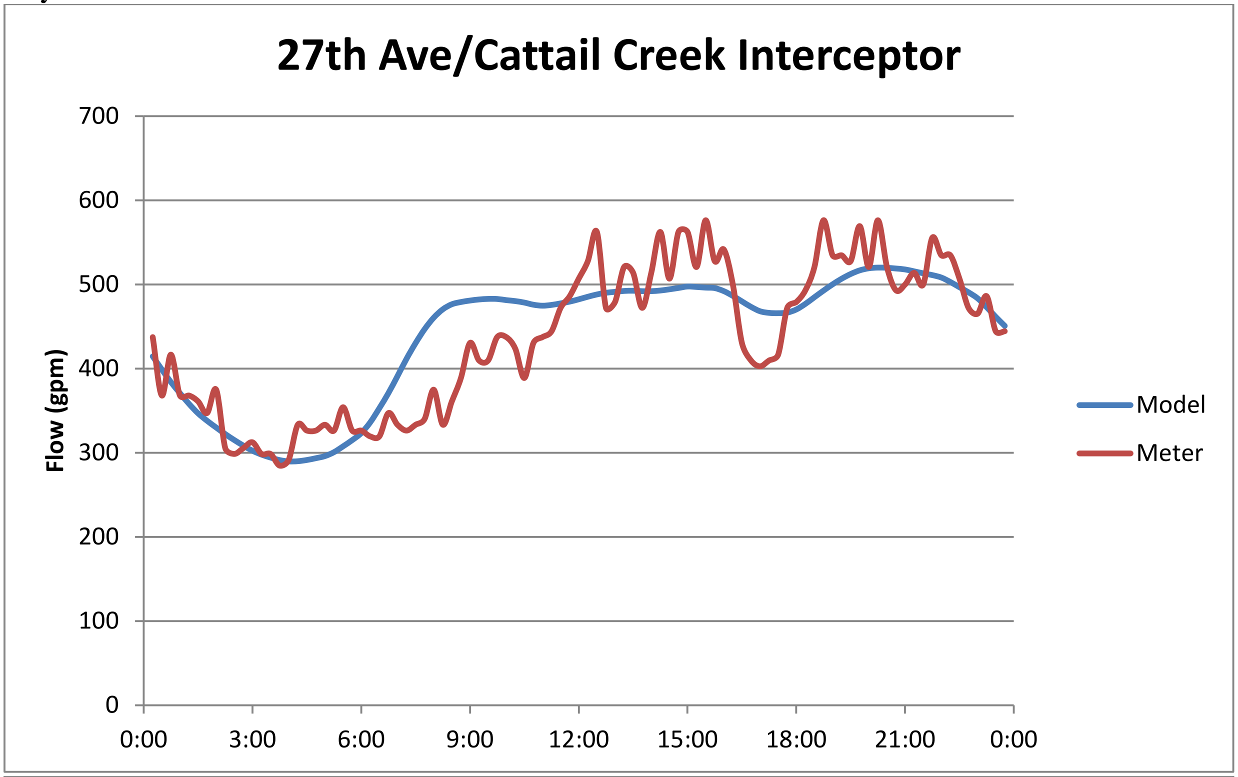

19th Ave/11th Ave Interceptor
Flow (gpm)
1600
1400
1200
1000
800
600
400
200
0
0:00
3:00
6:00
9:00
12:00
15:00
18:00
21:00
0:00
Model
Meter

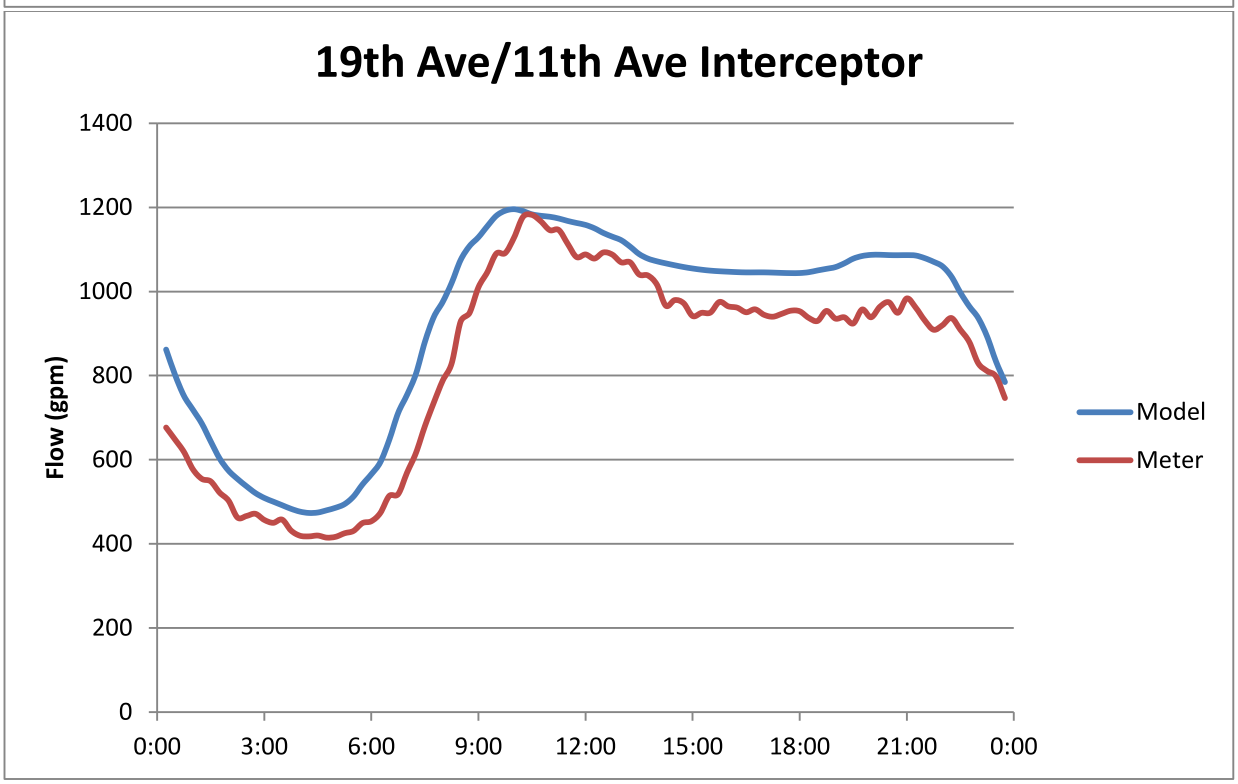

Baxter Interceptor
Flow (gpm)
900
800
700
600
500
400
300
200
100
0
0:00
3:00
6:00
9:00
12:00
15:00
18:00
21:00
0:00
Model
Meter

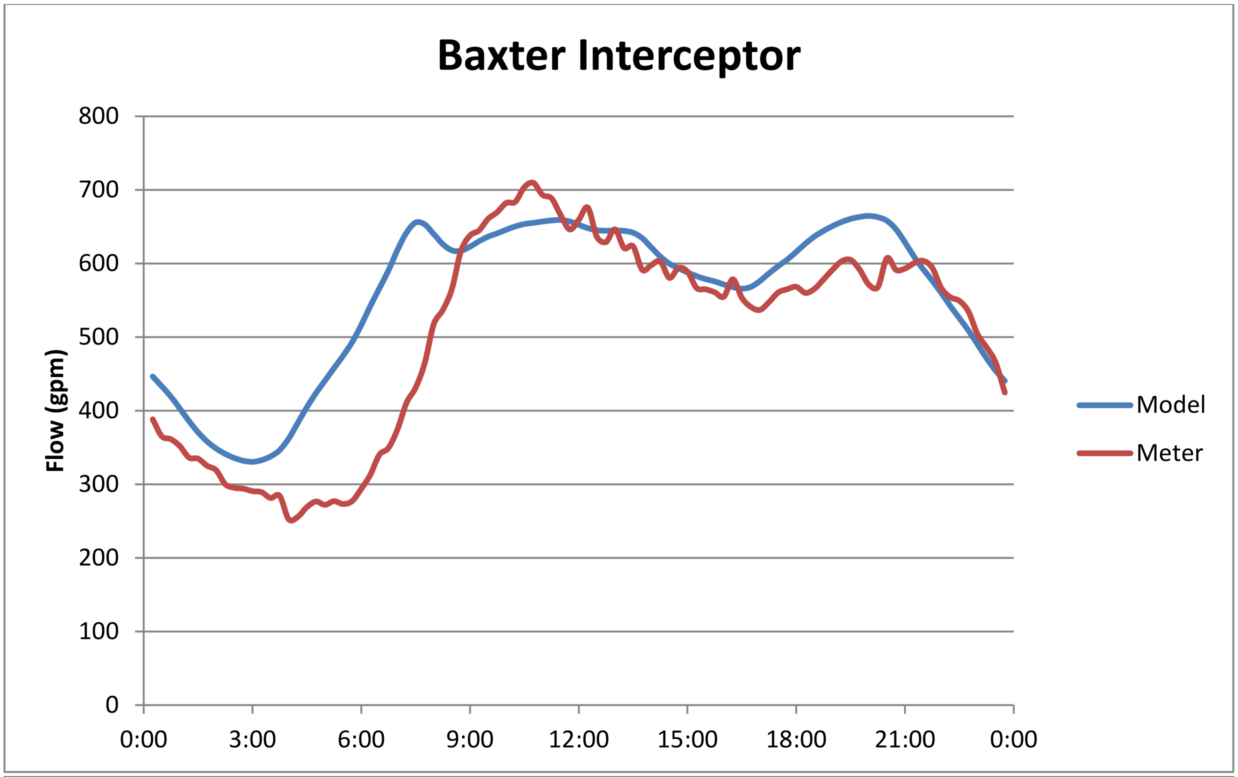

North Frontage Road Interceptor
Flow (gpm)
3,000
2,500
2,000
1,500
1,000
500
0
0:00
3:00
6:00
9:00
12:00
15:00
18:00
21:00
0:00
Model
Meter

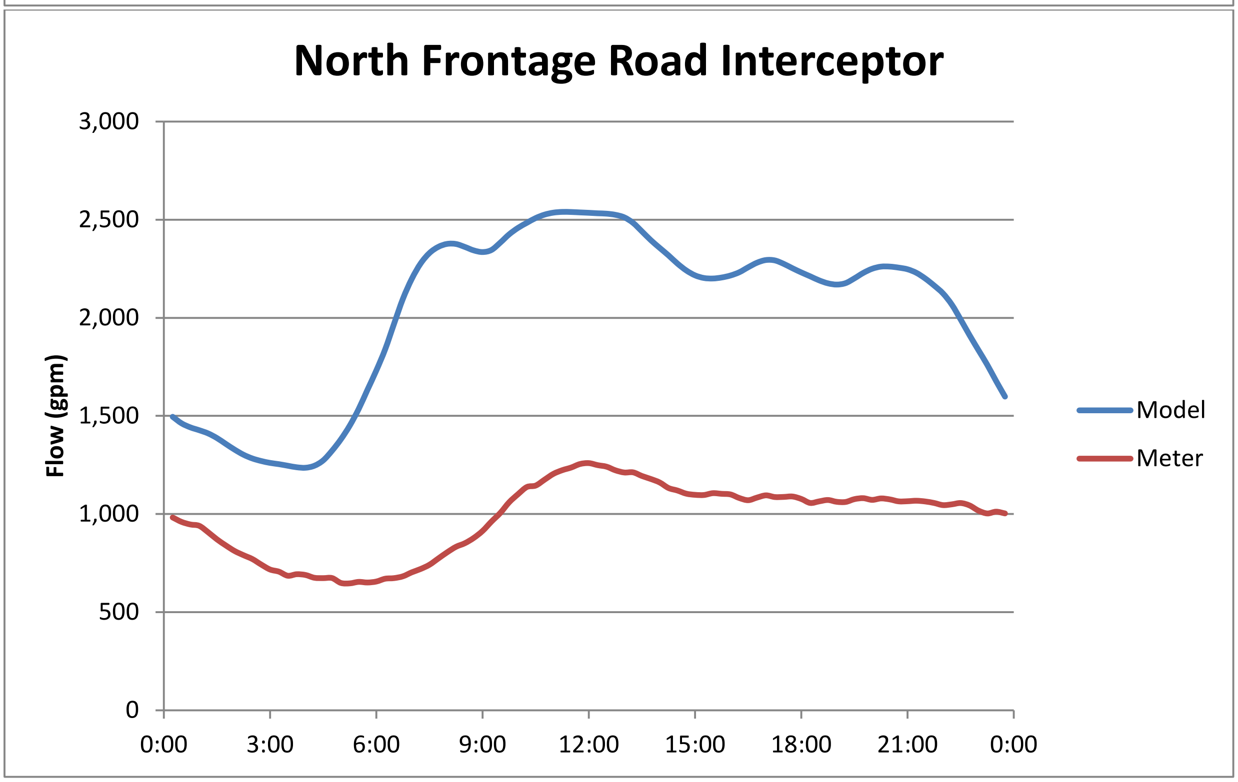

WWTP Influent
Flow (gpm)
6,000
5,000
4,000
3,000
2,000
1,000
0
0:00
3:00
6:00
9:00
12:00
15:00
18:00
21:00
0:00
Model
Meter

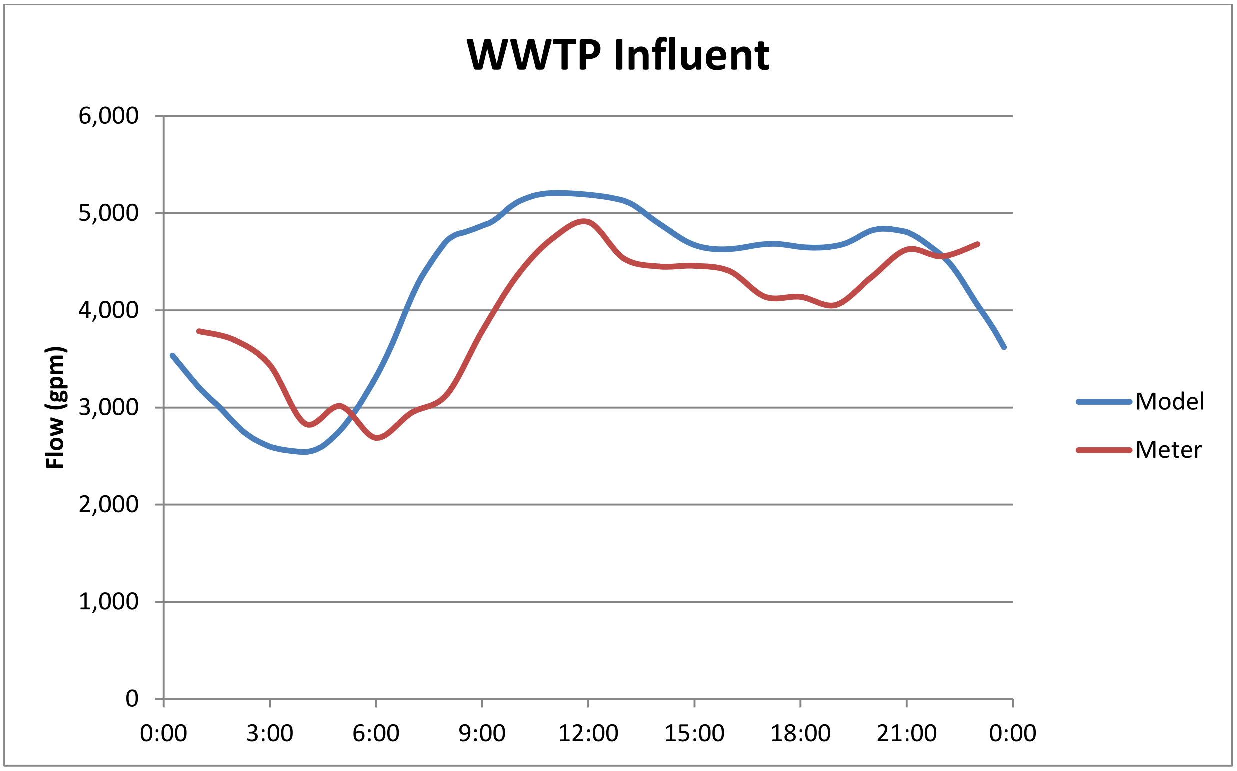
Dry Weather Flow Weekend Calibration





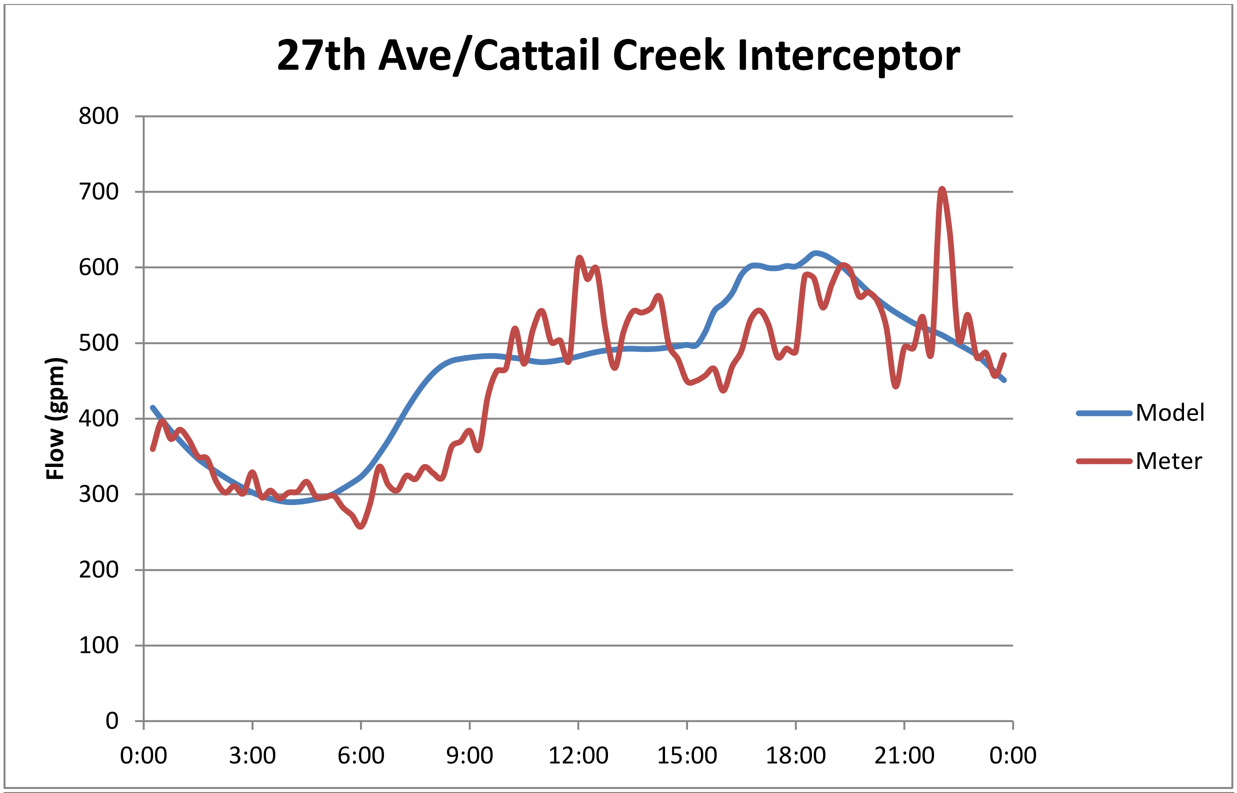

Baxter Interceptor
Flow (gpm)
800
700
600
500
400
300
200
100
0
0:00
3:00
6:00
9:00
12:00
15:00
18:00
21:00
0:00
Model
Meter

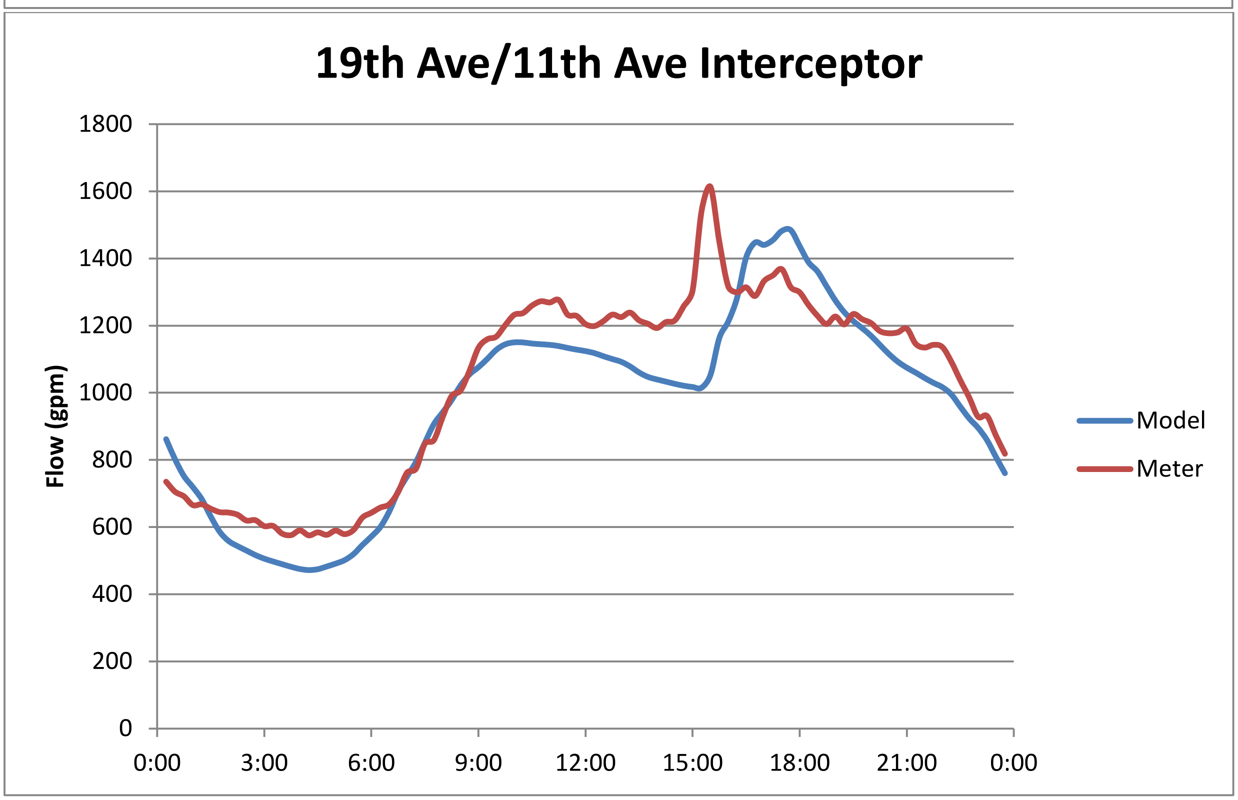

North Frontage Road Interceptor
Flow (gpm)
3,000
2,500
2,000
1,500
1,000
500
0
0:00
3:00
6:00
9:00
12:00
15:00
18:00
21:00
0:00
Model
Meter

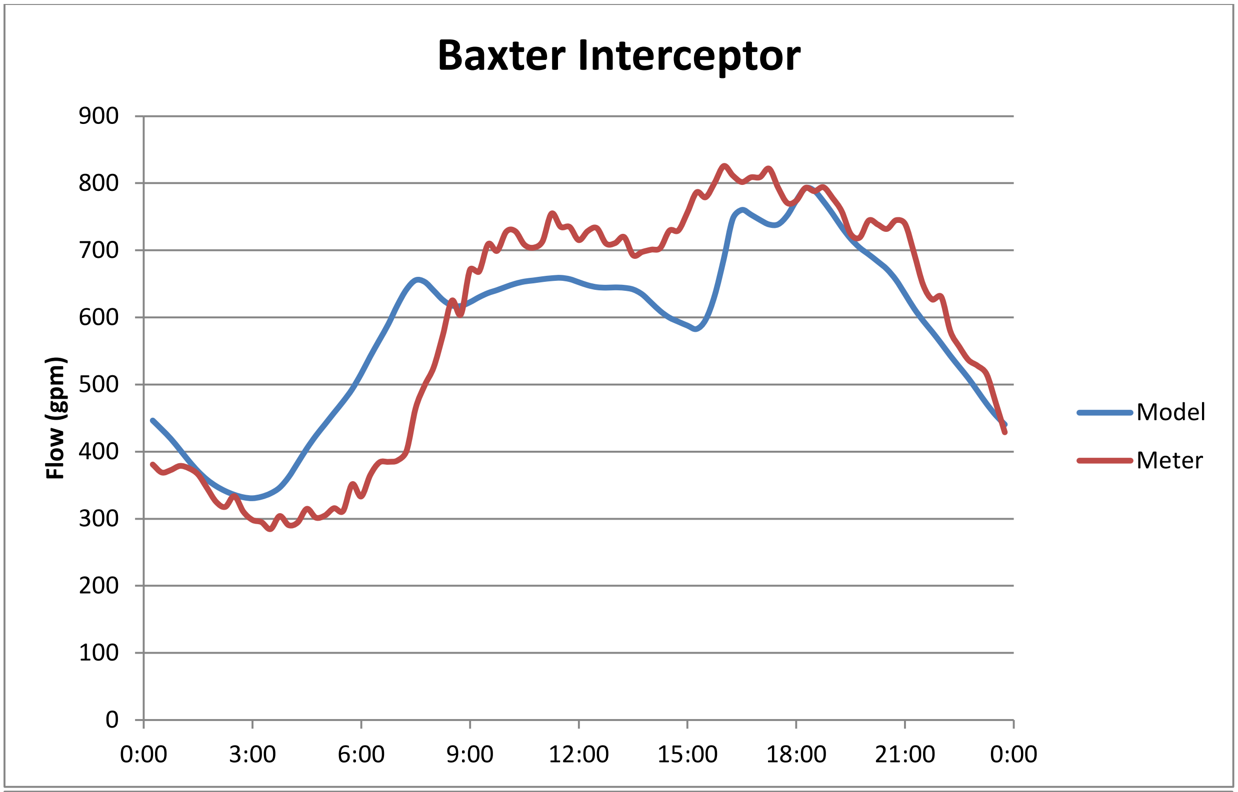


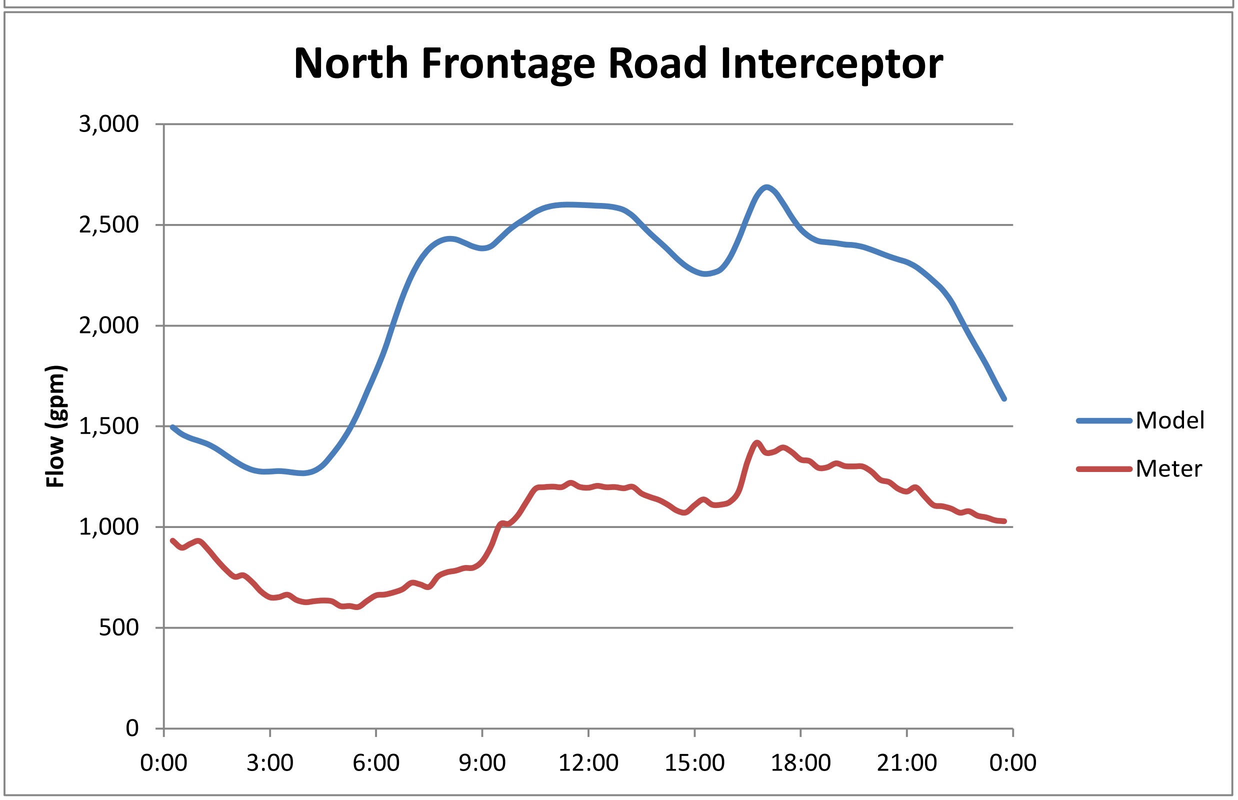
WWTP Influent
Flow (gpm)
6,000
5,000
4,000
3,000
2,000
1,000
0
0:00
3:00
6:00
9:00
12:00
15:00
18:00
21:00
0:00
Model
Meter

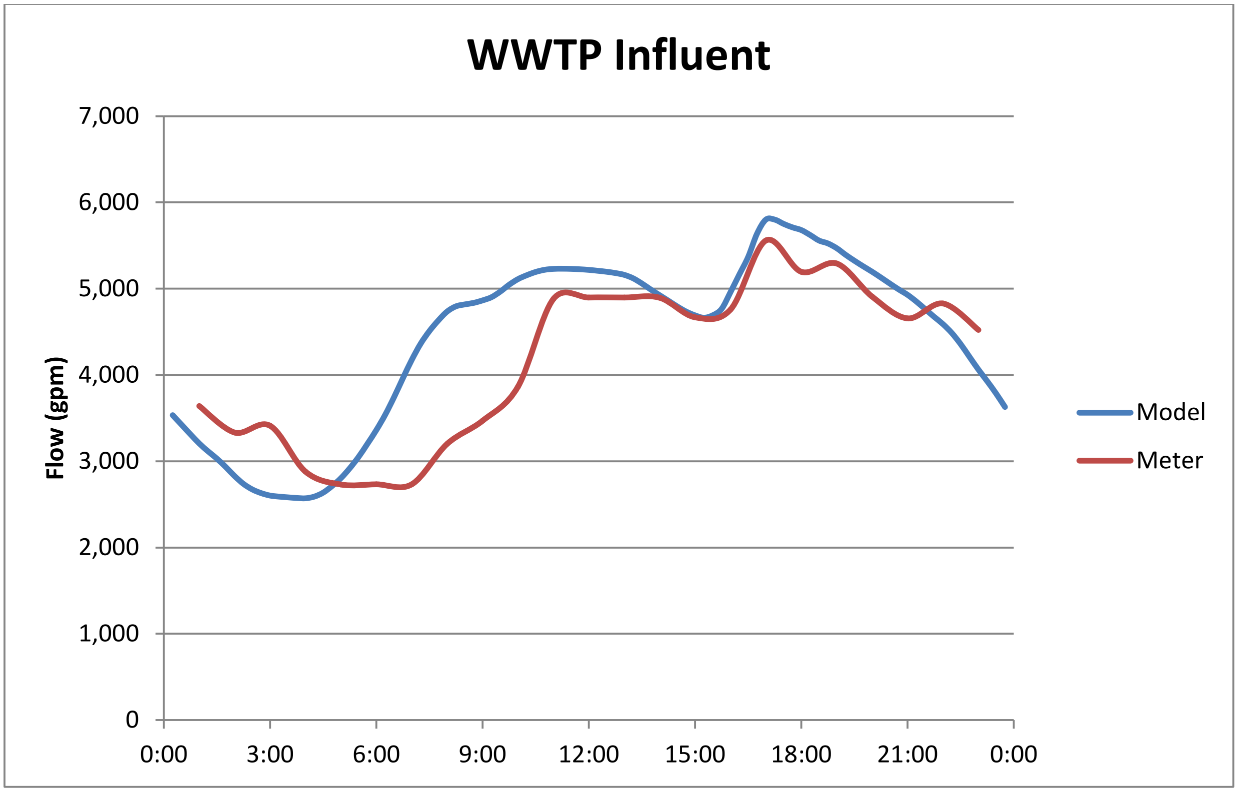
Wet Weather Flow Calibration






Baxter Interceptor
Flow (gpm)
900
800
700
600
500
400
300
200
100
0
0:00
3:00
6:00
9:00
12:00
15:00
18:00
21:00
0:00
Model
Meter


North Frontage Road Interceptor
Flow (gpm)
3,000
2,500
2,000
1,500
1,000
500
0
0:00
3:00
6:00
9:00
12:00
15:00
18:00
21:00
0:00
Model
Meter


WWTP Influent
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
Flow (gpm)
0:00
3:00
6:00
9:00
12:00
15:00
18:00
21:00
0:00
Model
Meter

Bozeman Wastewater Collection Facilities Plan Update

Chapter 5

Future Collection System Evaluation

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5.1 INTRODUCTION

As described in Chapter 2 – Basis of Planning, the City of Bozeman (City) is expected to see significant growth over the next few years that will have an impact on the existing wastewater infrastructure. This chapter discusses wastewater system improvements required to serve population growth within the existing city limit boundaries as well as the boundary used in the Bozeman Community Plan (Community Plan) and evaluate the impact on the existing wastewater infrastructure.

Within the existing city limits 9,400 acres of the total 11,700 acres has been developed. Most of the undeveloped and vacant land within the existing city limits is in the west, north and northeast portions of the City. As this area develops, it will add flows to the interceptors on the east and west side of the system and new collectors will need to be constructed to connect the new development to the existing interceptors. The west side of the system is at a lower elevation than the existing system, so the flows from the development will need to be pumped in order to convey the flow to the reclamation facility.

The Bozeman Community Planning boundary will expand the service area in all directions for the wastewater system to over 42,400 acres. This significant expansion will add load to the existing infrastructure and require an expansion of the wastewater system to provide service to the additional area. The areas of expansion to the north and west are at a lower elevation than the existing infrastructure and will require lift stations to provide service to these areas.

The 2007 Bozeman Wastewater Facility Plan (2007 Facility Plan) recommended collecting all of the loads produced east and southeast of the existing system from the Baxter Creek and Aajker Creek drainage basins and diverting this flow around the city to two proposed lift stations north of the city. Two lift stations, Gooch Hill and Hidden Valley, will then pump the flow to the Water Reclamation Facility (WRF). This study will re-evaluate the options for collecting the future loads and conveying it to the WRF.

5.2 FUTURE SYSTEM EVALUATION

In order to evaluate the future collection system, loads need to be projected out to the future planning periods. Also, facilities need to be planned to service expanded areas in order to ensure that all future developments can be serviced and the loads can be conveyed to the WRF to be treated. This section covers the steps taken to plan the locations of future facilities and calculate the anticipated loads in the planned system.

5.2.1 Future Facilities

To serve continuing development within the city limits and planning area, extensions of service, lift stations and force mains were laid out. The extensions were planned to follow existing roadways where roads existed and followed drainage basins where roads did not exist. All of the extensions were planned to follow the general drainage direction in order to ensure there would be some drop in elevation to allow for the sewer to drain by gravity as much as possible. Where the extensions could not flow by gravity to an existing interceptor or to the reclamation facility, lift stations and force mains were planned to pump the flow toward the reclamation facility. The model was used to size proposed extensions, including lift stations and force mains.

5.2.1.1 Drainage Basins

The drainage basins used to locate proposed extensions were delimited in the 2007 Facility Plan. In general, the drainage through the city goes from the east to the west on the eastern third of the city and then from south to north in the central and western two-thirds of the city. The WRF is located north of the city, so areas south and east in the city can flow by gravity to the reclamation facility. Areas on the western and northwestern portions of the city and the planning area will require lift stations to pump the flow to the WRF. The drainage and sub-drainage areas are shown in Figure 5-1.

5.2.1.2 Extensions and Lift Stations

Once the drainage basins were delineated using topography, future extensions were laid out across the study area. These extensions were assumed to either follow major road corridors, existing development, or drainages, as is the case in the eastern portion of the study area. The western portion of the study area primarily follows road corridors, which also correspond with drainage that generally falls to the north toward the East Gallatin River.

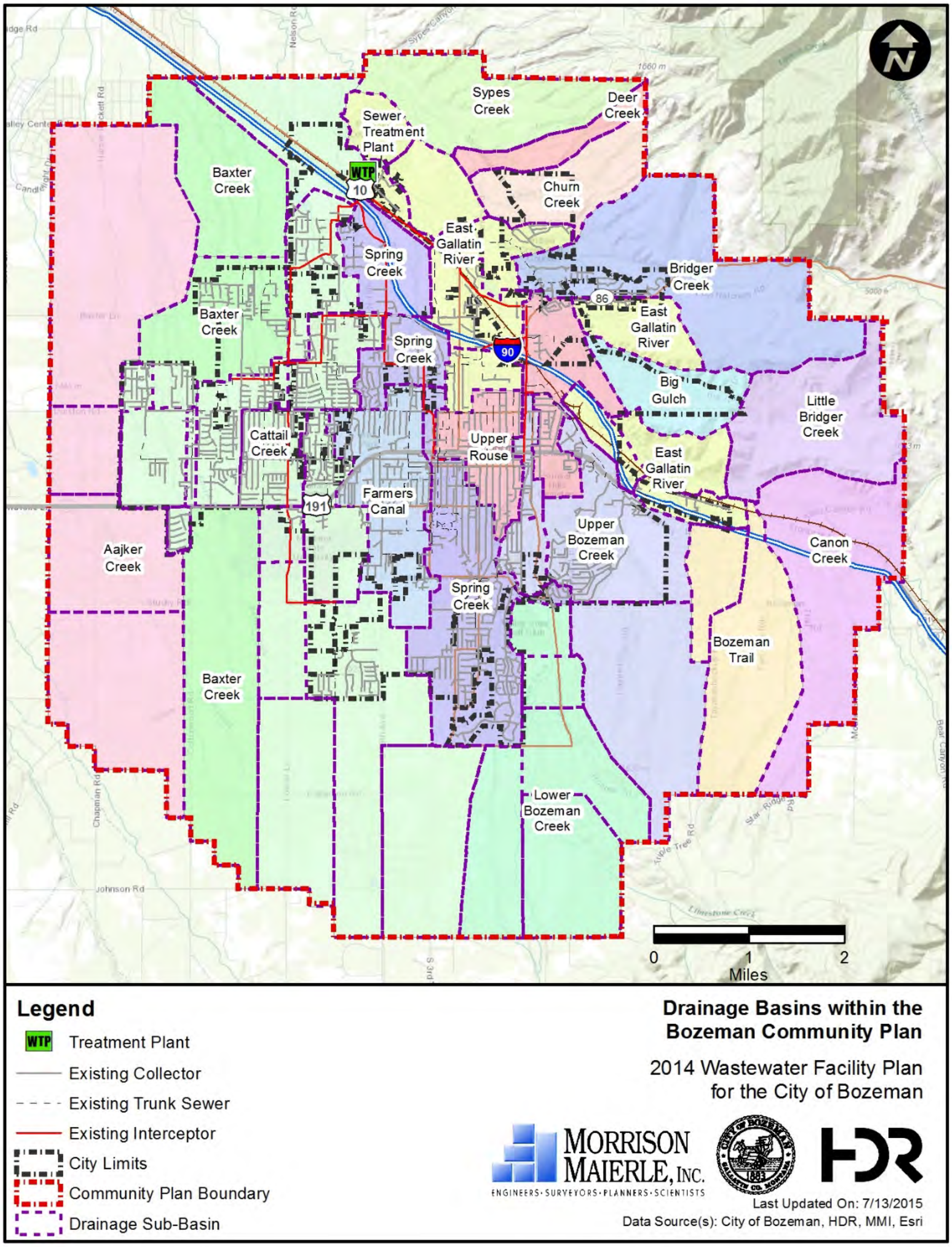
The East Gallatin River flows from the east central edge of the study area along the northern edge of Interstate I-90 and exits on the area in the north central portion of the study area. Due to this natural fall across the study area to the northwest, major extensions generally followed a northerly route on the western and central portions of the study area, and a westerly to northwesterly route on the eastern side.

Proposed extensions in the eastern portion of the study area follow drainages to the northwest toward the interstate and the East Gallatin River. Extensions in the northeastern corner of the study area including Bridger Creek, Churn Creek and south of Deer Creek flow in a southwestern direction to the East Gallatin River.

Based on the location of the WRF, development to the north of St. Andrews will require flow to be routed to McIlhattan Road, and then east to Spring Hill, where it will need to be pumped to the wastewater plant.

The Aajker Creek drainage basin on the west side of the Community Plan extents will also require a lift station. A small ridge is located in the northwestern corner of the study area between Baxter Creek and Aajker Creek. Areas directly west of this ridge will require a lift station. The area north of the Baxter Meadows Lift Station within the Baxter Creek drainage basin will also require a lift station to pump flows to the WRF.

Figure 5-1 – Drainage Basins within the Bozeman Community Plan



5.2.2 Future Loading

Chapter 2 – Basis of Planning identified several areas that could contribute future loading in the Bozeman wastewater collection system. These included:

- Developed areas within the City limits, 9,400 Ac
- Vacant zoned areas within the City limits, 2,346 Ac
- Land within the Growth Policy Boundary that has defined land uses, 42,463 Ac total

These areas were used to determine wastewater flow loading within the future system. All flows were allocated based on the area contributing to a manhole and on a load per acre basis that changed with land use designation. The loading areas were determined by creating Thiessen polygons within drainage areas based on the assumption that the existing and future sewershed areas would generally follow existing drainage areas. Some areas within the planning boundary are within a conservation easement, so they were removed from the boundary and did not contribute any future loads.

5.2.2.1 Developed Areas

The average dry weather flow within the existing developed area was assigned based on existing land use for developed and non-vacant properties within the city limits. There was one developed property, Meadowlark Elementary, outside of the city limits at the intersection of Cottonwood Road and Durston Road that was accounted for by modifying the city limits for this analysis. The occupied and developed land use was intersected with the Thiessen polygons to calculate the contributing land use to each manhole. The load to the manhole was calculated by multiplying the contributing land use by the flow rate by land use developed in Chapter 2 – Basis of Planning and shown in Table 5-1.

Table 5-1 – Wastewater Flow Rate by Land Use Category for Existing Uses[1]

Category	Dwelling Units per Acre	GAL./ACRE/DAY
Commercial/Auto	-	50
Commercial/Retail	-	1,000
Hotel/Motel	-	6,000
Light Manufacturing	-	800
Mixed Use	-	1,000
Restaurant/Bar	-	3,500
Public Facility/Park	-	25
Administrative/Professional	-	1,000
Church	-	360
Duplex/Triplex Residential	8.8	1,232
Mobile Home/Mobile Park	8.8	1,232
Multi-Family Residential	8.8	1,232
Single-Family Residential	6.1	854
School/Educational/Facility	-	400
Golf Course	-	30
Right of Way	-	0
Vacant	-	0
MSU [2]	-	2,220

1. The flow allocation in this table is based on net area.
2. Flow rate is based on MSU property east of South 19th Avenue.

5.2.2.2 Vacant Zoned Areas

Approximately 2,300 acres within the existing City limits is classified as vacant or undeveloped. To account for future flow from these areas, the land use developed in the Community Plan was used. Chapter 2 – Basis of Planning identified per acre loading parameters based on City zoning which were applied to all vacant properties within the City limits and is shown in Table 5-2. There were two areas within the City limits where the land use was modified from the land use in the Community Plan. A large area within the city limits around Big Gulch Drive was changed to Open Space due to terrain and access issues with developing. Also, property south of Bridger Drive and west of Story Mill Road recently purchased and set aside for open space was assumed to remain open space. The vacant and undeveloped areas were added to the model by using Thiessen polygons to calculate the contributing area and intersected with the land use to calculate the contributing area per land use. This load was used to evaluate the collection system for an existing and obligated scenario, if the City were to maintain its current limits.

Table 5-2 – Wastewater Flow Rate for Zoned Undeveloped Areas [1]

Designation	Dwelling Units per Acre	GAL./ACRE/DAY
R-S	6.5	910
R-1	3.9	546
R-2	5.2	728
R-3	6.5	910
R-4	10.4	1,456
R-O	5.2	728
RMH	5.2	728
B-1		1,000
B-2		2,000
B-3		3,000
M-1		960
M-2		960
B-P		960
NEHMU	6.5	910
UMU	10.4	1,456
REMU	10.4	1,456
PLI		1,030

1. The flow allocation in this table is based on gross area.

5.2.2.3 Land within Community Plan Boundary

The Community Plan identified a study area boundary and projected land use for the entire area. Projected loads from this area were developed by creating contributing areas for each manhole within the study area boundary and intersecting the area with the projected land use. The RAE Water and Sanitation District is within the Community Plan boundary and the City does not expect to serve this district. The RAE District was removed from the future loading. Chapter 2 –Basis of Planning outlined per acre loading parameters based on these proposed land uses and are shown in Table 5-3.

Table 5-3 – Wastewater Flow Rate by Land Use Designation[1]

Designation	Dwelling Units per Acre	GAL./ACRE/DAY
Industrial	-	960
Neighborhood Commercial	-	1,200
Community Commercial	-	2,400
Regional Commercial	-	1,600
Business Park	-	960
Public Institutions	-	1,030
Residential	5.5	770
Suburban Residential	1.3	182
Park& Open Space	-	25
Other Public Lands	-	1,030
Golf Course	-	30
MSU	-	2,780
MSU West		1,030
Future Urban		770

1. The flow allocation in this table is based on gross area

5.2.3 Dry Weather Inflow and Infiltration

Inflow and infiltration (I/I) in the future is expected to be similar to existing inflow and infiltration levels. The City has made an effort to reduce inflow and infiltration into the system by rehabilitating and replacing pipe in the existing system. This will help reduce inflow and infiltration from the older pipelines, but the overall system will continue to deteriorate over time. Based on water meter data discussed in Chapter 4 – Existing System Evaluation to calculate the indoor water use, the load entering the wastewater system from base wastewater load is approximately 4.9 MGD and the average flow to the reclamation facility is approximately 6.4 MGD. This extra flow is caused by infiltration and accounts for approximately a 33% increase to the base wastewater load.

The dry weather flow assigned to the model based on the land use for future scenarios was the base wastewater flow. To account for the inflow and infiltration into the system during dry weather days, the total base wastewater load was increased by 33% by assigning the extra flow to each manhole based on the contributing area to the manhole. In order to account for large rural areas that will experience I/I, but not as much as denser populated areas, the area was scaled by the land use flow rate. Areas with higher wastewater flow per acre was assumed to have more service connection and a higher chance of I/I while areas with lower wastewater flow per acre was assumed to have smaller infiltration contributions per acre. This method was used to allocate the inflow and infiltration to all future scenarios. The base wastewater load combined with the dry weather inflow and infiltration was used for the dry weather flow scenarios.

The method used to calculate infiltration is most appropriate for a system wide evaluation and is not intended to replace the City's current design standards. The model is good for predicting flow in the City's interceptors, trunk sewers and areas close to the flow monitoring locations, but is not as good at predicting peak flows in local sewers. The City should continue to size local sewers based on the existing City standards.

5.2.4 Rainfall Derived Inflow and Infiltration

The additional loads caused by rainfall were added to the wastewater system by applying rainfall and a unit hydrograph to an area assigned to each manhole. The sewershed area was calculated by using the same Thiessen polygons within drainage areas that were developed to calculate dry weather flows and the same area scaling factor was applied per land use. The rainfall event used to predict wet-weather loads in the future system was the same 25-year storm event created in Chapter 4 – Existing System Evaluation.

The unit hydrograph parameters used to calculate the inflow was based on the average response calculated in the calibration process discussed in in Chapter 4 – Existing System Evaluation.

The method used to calculate peak flows from inflow and infiltration is different then the method described in the City of Bozeman's Design Standards. The City uses a peaking factor method described in the 10 State's Standards where a peaking factor is applied to the base flow to calculate the peak design flow. The peaking factor gets smaller as base flow increases downstream to account for storage and timing of peak flows reducing the peaking factor. This method is typically applied to steady-state analysis of a collection system and does not apply to extended period simulations where storage and timing of peak flows are accounted for in the simulation.

5.3 FUTURE SYSTEM EVALUATION RESULTS

The analysis of the future collection system was split into two main planning periods in order to help plan the timing of improvements for the City. The first planning period covers the development within the city limits also referred to as the existing and obligated area. This will be used to plan near term improvements for the City, so the City can plan and budget for the required improvements. The second planning period covers development of the entire Community Plan extents. This second period provides a long term outlook for the wastewater system to allow the City to see what the system might look like in order to plan and budget for the required improvements.

Four main scenarios were evaluated for the future system evaluation

- Existing and obligated
- Wet-weather existing and obligated
- Study Area build-out with extensions and
- Wet-weather study area build-out with extensions.

The existing and obligated model runs will be utilized to determine priorities of future system improvements generated from the build-out model runs. The runs will assume that the entire area within the city limits is completely developed. This will show if the existing infrastructure has capacity for the full development of the existing obligated service area and what improvements will be required to serve this area.

The build-out model runs will be used to determine the size of the infrastructure required to serve the entire study area. These scenarios assume the entire study area is completely developed based on land use used in the Community Plan. This will show what improvements and expansions to the wastewater system will be required to serve the entire future service area.

5.3.1 Evaluation Criteria

Similar to the existing system analysis, the future system extensions will be analyzed in accordance with standards established based on the City of Bozeman Design Standards by including:

- Peak-hourly wet-weather depth over diameter no greater than 75%;
- A Manning's friction factor of 0.013 is assigned to all new sewers;
- And minimum diameter of sewer mains is no less than 8 inches.

5.3.2 Existing and Obligated System Analysis

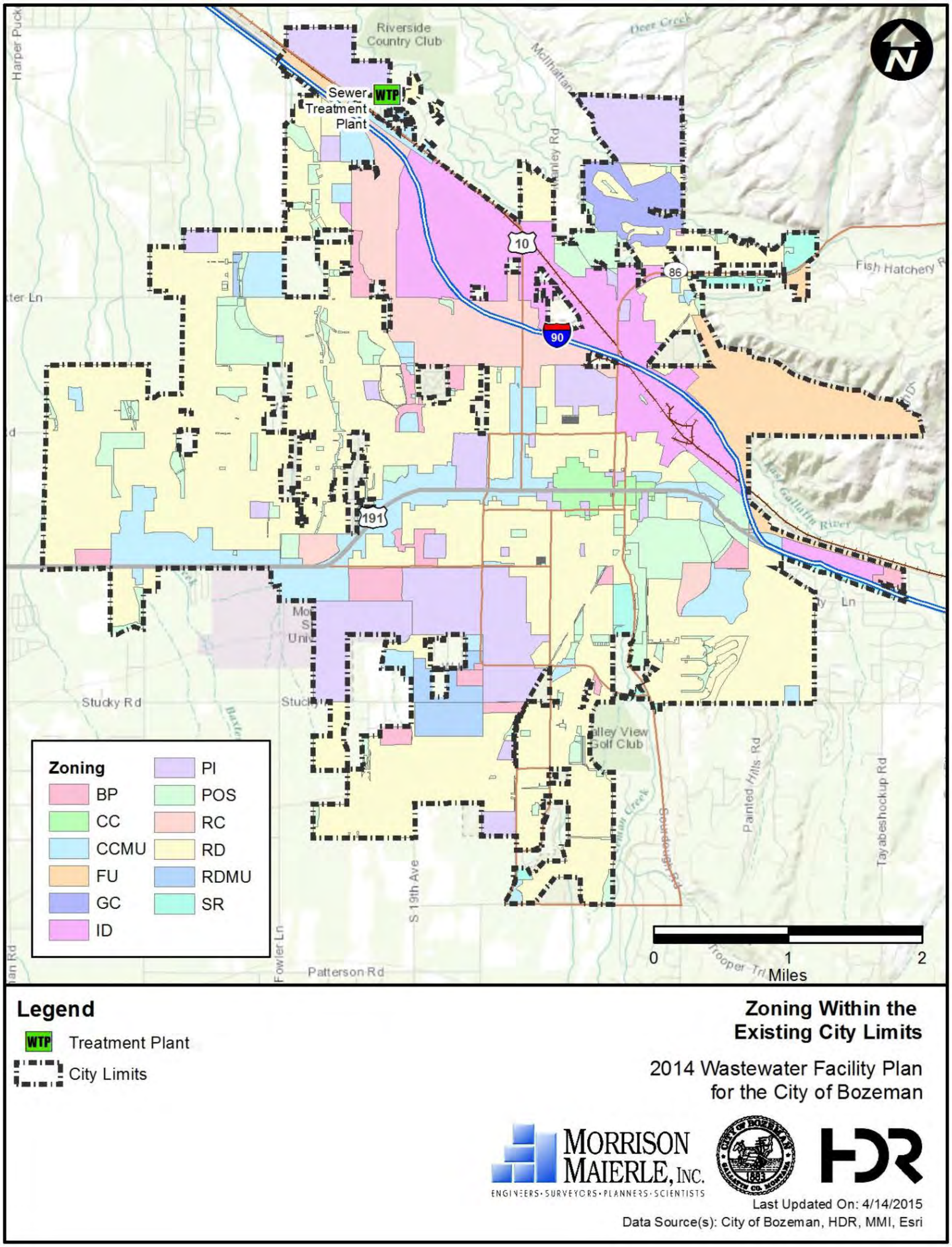
The wastewater collection system was evaluated under the completely developed existing and obligated flows in order to locate areas of the systems that may see near-term capacity constraints. The analysis also looks at what extensions to the existing system will be needed to serve the currently undeveloped areas of the City. All of the facilities were sized based on both the entire area within the Bozeman City Limits developing and the ultimate build out of the Community Plan. The peak flow at the WRF for this analysis reached 18.5 MGD. For comparison, the anticipated average flow at the WRF by the year 2034 is 8.2 MGD based on an anticipated population of 63,964 residents. The land use and zoning used to develop the inflow for this scenario are shown in Figure 5-2

Norton East Ranch Outfall Sewer

The Norton East Ranch Outfall Sewer will divert flows from the outlet of the Norton Lift Station and direct this flow to the Baxter Meadows Lift Station. This diversion of flows will reduce flow to the Baxter Interceptor and provide extra capacity for flow to the interceptor from other sources.

The new interceptor varies in size from 21-inches at the inlet to 27-inches in the middle of the interceptor and ends with a temporary portion at 15-inches. The limiting capacity of the interceptor is in the 15-inch portion where the capacity is 1.6 MGD. For this analysis, the sewer was assumed to be constructed based on construction drawings dated May 5, 2014.

Figure 5-2 – Zoning Within the Existng City Limits



5.3.2.1 Existing and Obligated Gravity System

The development within the Bozeman City Limits will have a significant impact on the east side of the gravity system. Other than the trunk and interceptor sewers along the east side of the collection system, the rest of the collection system has enough capacity to convey the existing and obligated flows. Table 5-4 compares the capacity of the interceptors to the maximum flow in the interceptor produced from the existing and obligated full development. The recommended pipe improvements under the full development of the existing and obligated flows condition is listed in Appendix A and shown in Figure 5-4.

Table 5-4 – Existing and Obligated Interceptor Results

Interceptor	Size	Capacity (MGD)	Existing and Obligated Peak Flow (MGD)
Rouse	30 24	14 8	5
North Frontage Road	30 Parallel 20 20	5 7 8	6.2
19th Avenue/11th Avenue	24	10	7.4
Baxter	24 21 20	4 4 3	2.3
27th Avenue/Cattail Creek	27 24	10 9	4.6
Evergreen	21	4	0.8
Davis-Fowler	18 24	5 13	2.7
Norton East Ranch	21 27 15	7 8 1.6	0.5
WWTP	30	19	18.4

Existing System Deficiencies

There are capacity constraints in the existing system that should be addressed before the east side of the city is fully developed. The southeast undeveloped portion of the city will eventually need to flow to the existing trunk sewer that starts at the intersection of I-90 and Main Street and ends at the Rouse Interceptor. The undeveloped contributing area is predominantly the Bozeman Deaconess Health Services development and is expected to be developed in the near term. When this property develops, the entire trunk sewer will need to be replaced or paralleled in order to collect the future expected flows. The impact on the existing sewer system from the development of the Bozeman Deaconess Health Services development area was analyzed in detail for the City and results are presented in Appendix A.

City staff has noted that there are probable capacity deficiencies associated with the South Church Avenue 8" sewer main. This is based on specific flow monitoring and extrapolations of peak and future flow values within the sewer main. While the existing system model did not confirm that this area has capacity problems, these problems could exist due to high local infiltration not captured in the model or flow monitoring for this study. The model is calibrated to the flow monitoring utilized for this study and will not fully define areas of high infiltration in localized portions of the collection system. It is recommended that the city require applicants to perform a project specific analysis for any significant connections to small sewer mains to identify any local deficiencies.

The expected existing and obligated flows will cause the North Frontage Road Interceptor to exceed capacity in two separate locations. The interceptor has two parallel 20-inch pipes that run along Manley Road. The interceptor has a bottleneck where the two lines are combined and capacity of the interceptor will be exceeded at this location. The final segment of the interceptor before it discharges to the WWTP Interceptor creates a bottleneck in the interceptor and the segment capacity will be exceeded. These portions of the interceptor will need to be replaced or paralleled in order to collect the future expected flows. The results for the entire system showing the maximum d/D is shown in

Figure 5-3.

South University Area Development

The South University Area is currently being planned for development and will generate significant sanitary sewer flows. The local sewer lines adjacent to the property were not sized to serve the development area. Options for serving the development were analyzed and recommendations were made on how to serve the area for the City and are presented in Appendix B.

Existing and Obligated Extensions

The following system extensions shown in Figure 5-4 will be needed in order to serve the undeveloped areas of the City.

5.3.2.2 Existing and Obligated Lift Station Evaluation

Development within the existing city limits will have a small impact on most of the City's lift stations. Both the Norton Ranch and Baxter Meadows lift stations will reach capacity when the Norton Ranch property completely develops. When the Norton East Ranch Outfall Sewer is completed, the flows from the Norton Ranch Lift Station will be diverted to the Baxter Meadows Lift Station. There is capacity in the Baxter Meadows Lift Station to pump all of the extra existing flows, but when the Norton Ranch property is fully developed, the Baxter Meadow Lift Station inflow will reach capacity.

In order to serve areas north of the Cattail Lake Lift Station, another lift station will need to be built at the intersection of Davis Lane and E Valley Center Road in order to pump flow generated by the area to the WRF. Table 5-5 compares the existing capacity of the lift stations to the existing and obligated flows into the lift station.

Table 5-5 – Existing and Obligated Lift Station Flows

Lift Station	Existing Capacity (gpm)	Peak WWF (gpm)
Baxter Meadows	690	710
Bridger Center	100	10
Burrup	450	114
Cattail Lake	225	40
Laurel Glen	450	360
Loyal Gardens	364	80
Norton Ranch	121	350
Overbrook	Unknown	7
Walker	Unknown	20
Links	160	8
Cardinal Distribution	Unknown	10
Davis Lane	N/A	55

Figure 5-3 – Existing and Obligated Results – Maximum d/D

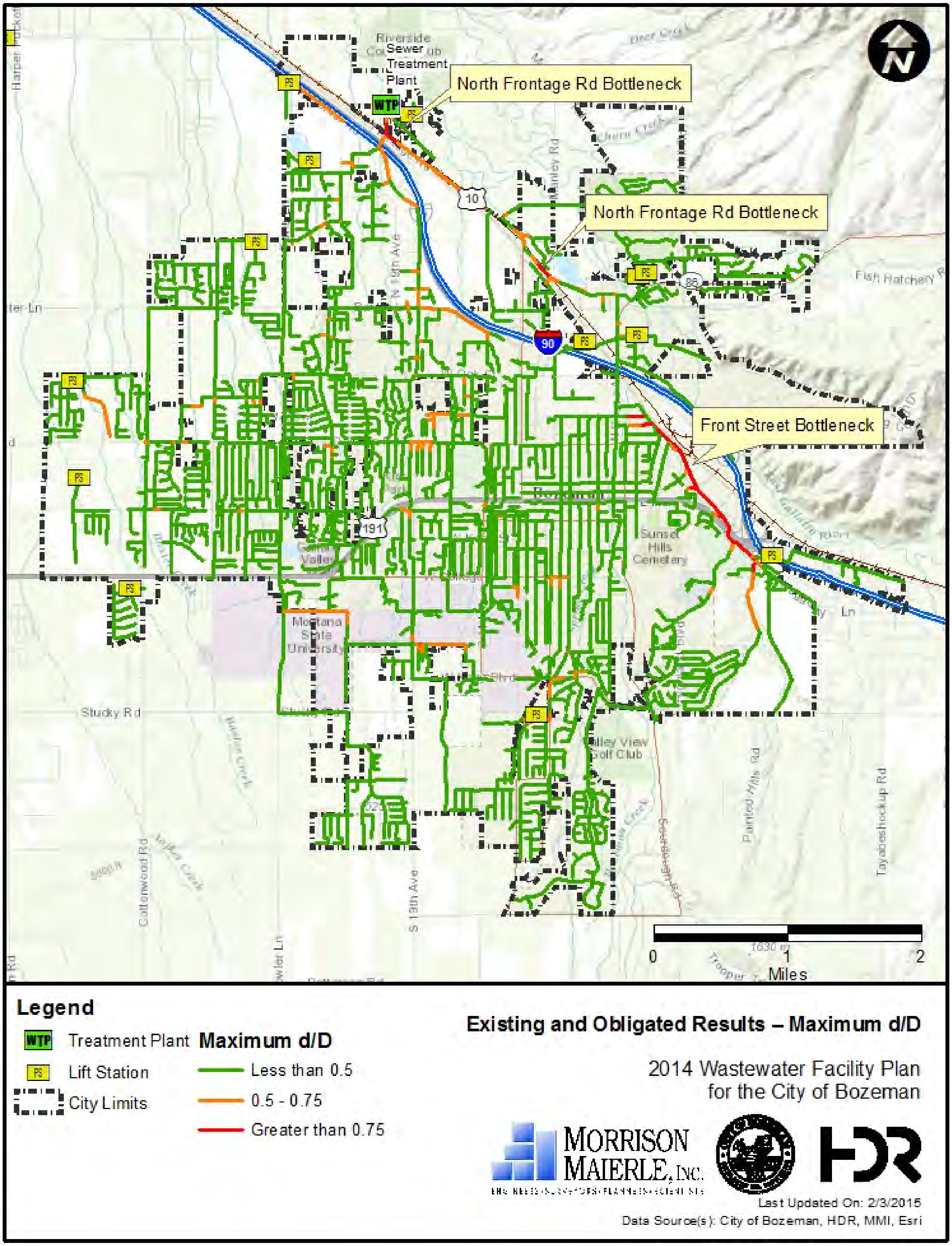
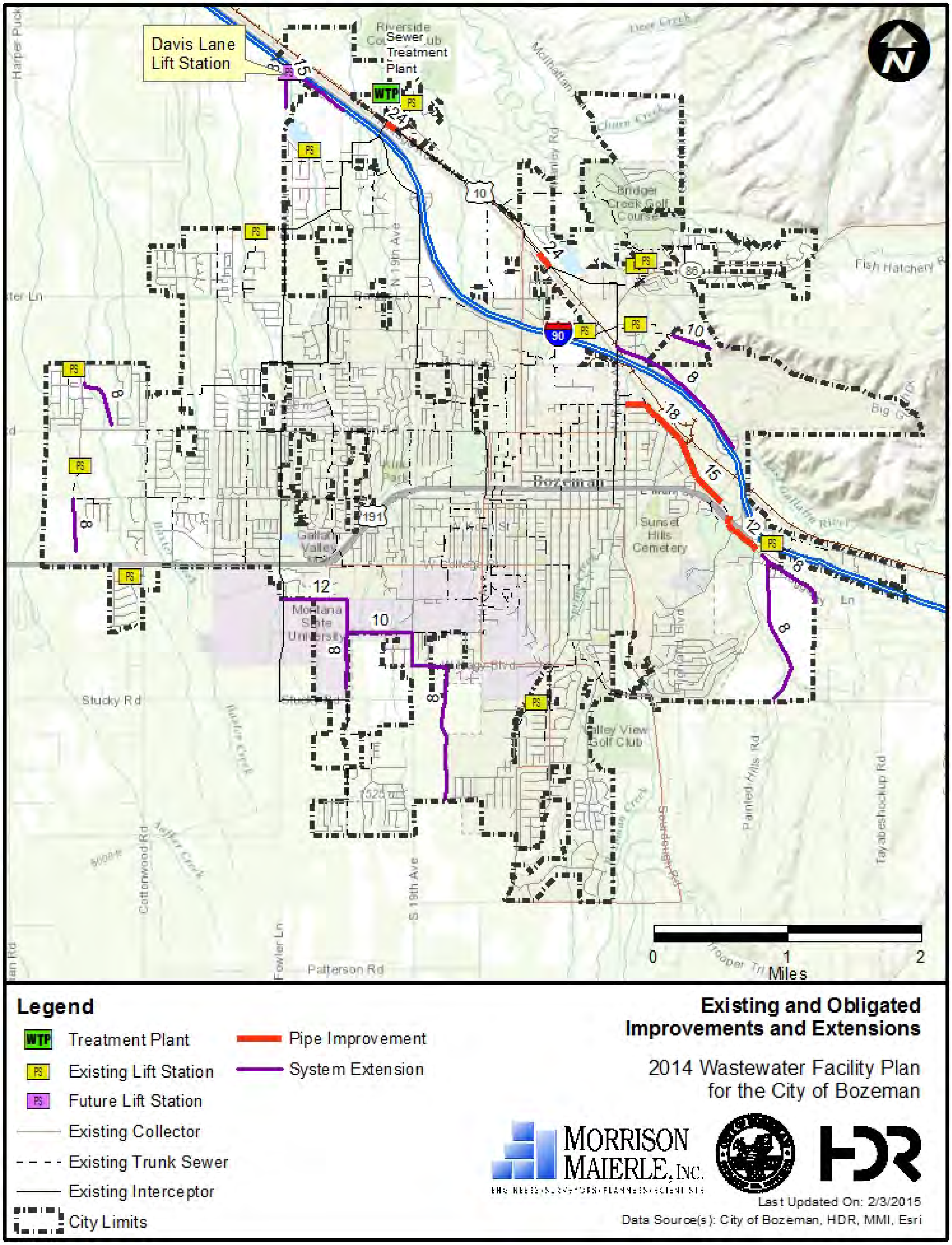


Figure 5-4 – Existing and Obligated System Improvements and Extensions



5.3.3 Bozeman Community Planning Area Analysis

The Community Plan calls for the City to increase the service area from the existing 11,700 acres to 42,500 acres and the peak flow wet weather flow to the reclamation facility is expected to reach 60 MGD. In order to serve this expanded area, new sewer lines and lift stations will need to be constructed and some of the existing pipes will need to be replaced or paralleled. This analysis looks at what improvements long term improvements will be needed in order to serve the possible future city limits based on the Community Plan.

5.3.3.1 Community Plan Gravity System

The flows generated by developing the entire Community Plan will generate significantly more load than what is experienced in the existing system. Much of the new area is south of the existing city and will need to be conveyed through the existing system in order to contribute to the WRF. All of the pipes that will require replacement discussed below are listed in Appendix C and shown in Figure 5-6. The expected peak wet-weather flow and capacity in each of the interceptors is shown in Table 5-6.

Table 5-6 – Community Plan Interceptor Results

Interceptor	Size	Capacity (MGD)	Community Plan Peak Flow (MGD)
Rouse	30 24	14 8	20
North Frontage Road	30 Parallel 20 20	5 7 8	26
19th Avenue/11th Avenue	24	10	10
Baxter	24 21 20	4 4 3	4
27th Avenue/Cattail Creek	27 24	10 9	10
Evergreen	21	4	0.8
Davis-Fowler	18 24	5 13	9
Norton East Ranch	21 27	7 8	6
WWTP	30	19	47

Davis-Fowler Interceptor

Development of the Cattail Creek and Farmers Canal Basin south of the existing city limits will increase flows to the Davis-Fowler Interceptor. The older section of the interceptor constructed in 1980 between Durston Road and West Oak Street will exceed capacity. This section of the interceptor is only 18-inches in diameter and connects the 24 and 21 inch diameter segments of the interceptor constructed in 2006. The capacity of the 18-inch diameter section is approximately 5 MGD and the full build-out peak wet-weather flow is expected exceed 7 MGD. In order to convey the ultimate build-out flow, the interceptor will need to be increased from an 18-inch diameter pipe to a 24-inch diameter pipe.

North Frontage Road Interceptor

The development of the drainage basins in the southeast and east portion of the Community Plan area will cause the North Frontage Road Interceptor to exceed capacity. The bottleneck starts where the interceptor splits into parallel 20-inch pipes at North Rouse Ave and continues until the interceptor discharges to the WWTP Interceptor. The interceptor changes size from the parallel 20-inch diameter pipes to varying between 20-inch and 30-inch diameter pipes as it goes along Frontage Road. The entire length of the interceptor is undersized and will need to be replaced or paralleled in order to convey the anticipated peak wet weather flow of 27 MGD at the end of the interceptor.

Front Street Interceptor

Upstream of the North Frontage Road Interceptor and Rouse Interceptor an existing trunk sewer that runs from the intersection of I-90 and Main Street and eventually ends at the Rouse Interceptor is all undersized to convey the contributing ultimate build-out Community Plan flows. The peak hour wet weather flow in this scenario is expected to reach 4.9 MGD and the entire sewer line will need to be upsized or paralleled to convey this flow.

WWTP Interceptor

The WWTP interceptor receives flow from the entire City and Community Plan Boundary. The capacity of the interceptor is exceeded in the Community Plan build-out scenario. The interceptor is currently 30-inches in diameter and will need to be increased to 48-inches in diameter. The interceptor discharges directly to the reclamation facility and influent flows are monitored at the reclamation facility. When the peak wet weather flow at the reclamation facility reaches 19 MGD, it will indicate that the interceptor is at capacity and should be replaced. The ultimate build-out of the Community Plan area is expected to increase the flow through the WWTP Interceptor to 48.5 MGD under peak wet weather flow conditions.

Norton East Ranch Outfall Sewer Interceptor

The final 1,300 feet of the Norton East Ranch Outfall Sewer was not designed to convey the ultimate build-out flows as the new interceptor changes from 27-inches to 15-inches at Baxter Lane and Flanders Mill Road. The 2007 Facility Plan assumed the 15-inch section of the interceptor would eventually be abandoned and the interceptor would be extended north to a new lift station. The capacity of the 15-inch portion of the interceptor is 1.6 MGD and it will be able to serve the build-out existing and obligated flows. The sewer will reach capacity as the Aajker Creek drainage basin develops and is served by the City. The peak wet weather flow in the interceptor is expected to reach 9.6 MGD if the Community Plan area is fully developed.

Two scenarios were evaluated to determine what should be done with this interceptor. The first scenario evaluated the improvements required if the 15-inch portion of the interceptor was eventually abandoned and the second scenario evaluated the improvements required if a portion of the flow was diverted to the north and the 15-inch sewer remained in service.

Norton East Ranch Full Diversion Scenario

If the Norton East Ranch Interceptor is fully diverted to the north, all of the flow in the Norton East Ranch Interceptor will have to go through a new interceptor and eventually be pumped to the WRF at the proposed Hidden Valley Lift Station. The full diversion will reduce flows to the 27th

Ave/Cattail Creek Interceptor as well as the Baxter Meadows Lift Station and reduce the improvements required for both the interceptor and the lift station. The flow reduced from the 27th Ave/Cattail Creek Interceptor and Baxter Meadows Lift Station Davis Lane Lift Station. The peak wet weather flow in the ultimate build-out scenario is expected to reach 9.6 MGD.

Norton East Ranch Partial Diversion Scenario

If the 15-inch portion of the Norton East Ranch Interceptor is left in place after the interceptor is extended to the south approximately 1.6 MGD can continue through the 15-inch sewer and 8.1 MGD will continue down the diversion. This will cause increased flows in the 27th Ave/Cattail Creek Interceptor and Baxter Meadows Lift Station. In this scenario, approximately 5,700 feet of the interceptor would need to be replaced or paralleled to convey the extra flow. Additionally, 2,700 feet of sewer between the diversion and the Baxter Meadows Lift Station would also need to be replaced or paralleled.

Community Planning Area Extensions

The system extensions shown in Figure 5-6 will be needed in order to serve the undeveloped areas of the City. The size and length of all of the extensions are listed in Appendix B. All of the planning area extensions were sized to carry the ultimate build-out flow of the Community Plan service area.

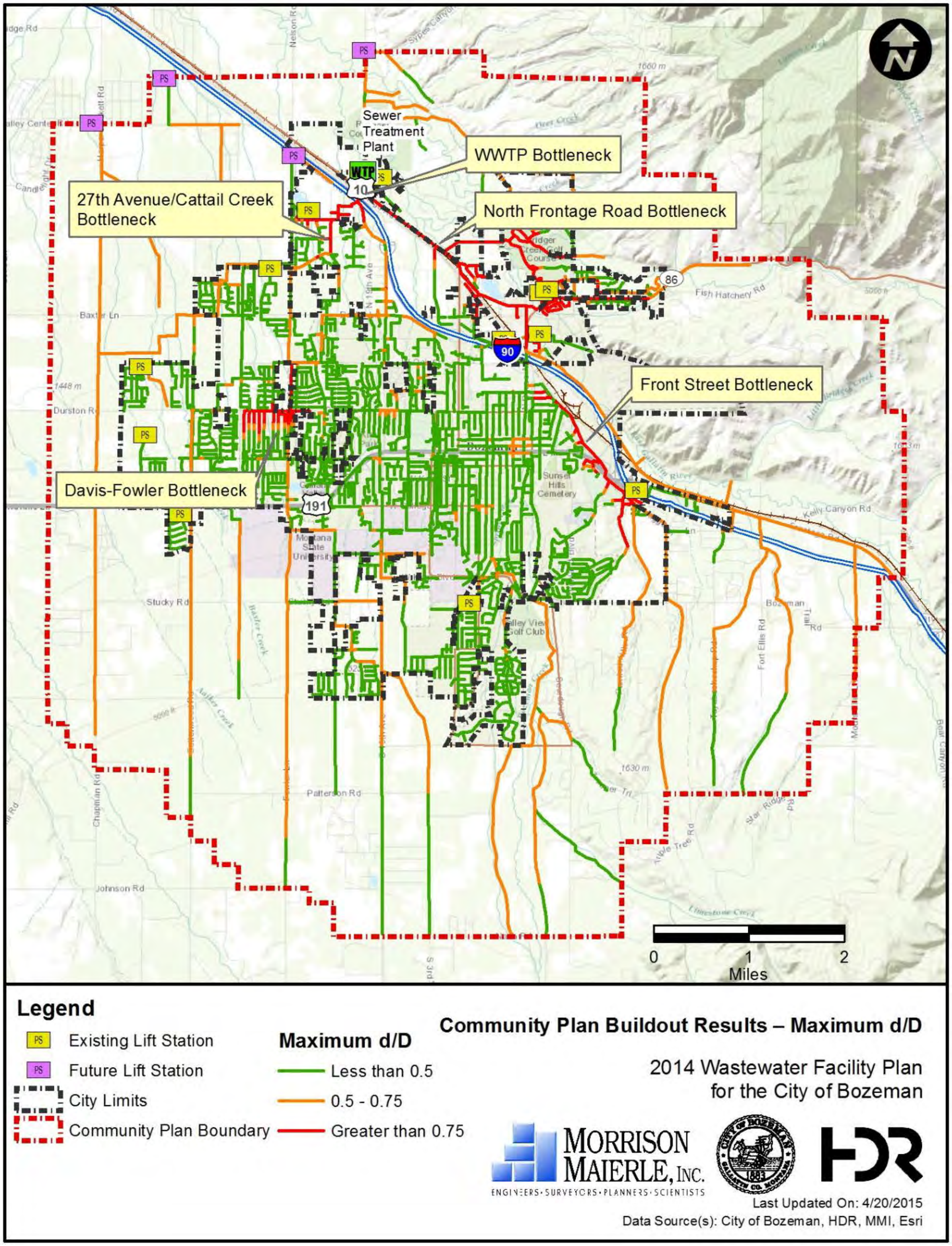
Aajker Creek Basin Extension

The Aajker Creek Basin is the drainage on the western side of the Community Plan boundary. Due to the topography of the City, the northern portion of the City will have to be serviced by a lift station in order for the area to contribute to the reclamation facility. It is possible for the Southern portion of the basin to drain by gravity to the Norton East Ranch Outfall Sewer Interceptor and eventually the existing Baxter Meadows Lift Station. In order to reduce the ultimate flow to the existing infrastructure a diversion can be built to divert some of the flow north to the new lift station required to service the rest of the basin. The diversion will not be needed until approximately 40 to 60 percent of the basin to the south of the diversion is developed. The percentage depends on the final developed land use and timing of development in other basins contributing the Baxter Meadows Lift Station.

5.3.3.2 Community Plan Lift Station Evaluation

The build-out of the Community Plan area will have a small impact on most of the existing lift stations. Most of the impact will be seen on the lift stations on the west side of the city that have contributing areas that haven't fully developed. The Baxter Meadows and Norton Ranch lift stations will see the largest impact from development. The Norton Ranch Lift Station will exceed capacity when the Norton Ranch Subdivision is fully developed and flows to the lift station should be monitored as the subdivision develops. The Baxter Meadow Lift Station will be greatly influenced by the Norton East Ranch Interceptor. It will see the most immediate impact when the interceptor is constructed and will continue to be impacted as flows increase in the interceptor with development in the Aajker Creek Drainage Basin. The projected peak wet weather flows to the lift stations based on the build-out of the Community Plan area are shown in Table 5-7 and the locations are shown in Figure 5-6.

Figure 5-5 – Community Plan Buildout Results – Maximum d/D



In order to serve areas north of the existing system, four lift stations are proposed to pump flows to the WRF from this area.

Gooch Hill Lift Station
The Gooch Hill Lift Station will pump flows from west of the Baxter Creek ridge and collect flows from the northern section of the Aajker Creek and Baxter Creek Drainage Basins.

Hidden Valley Lift Station
The Hidden Valley Lift Station will collect flows from Baxter Creek Drainage Basin north of the Baxter Meadows Subdivision.

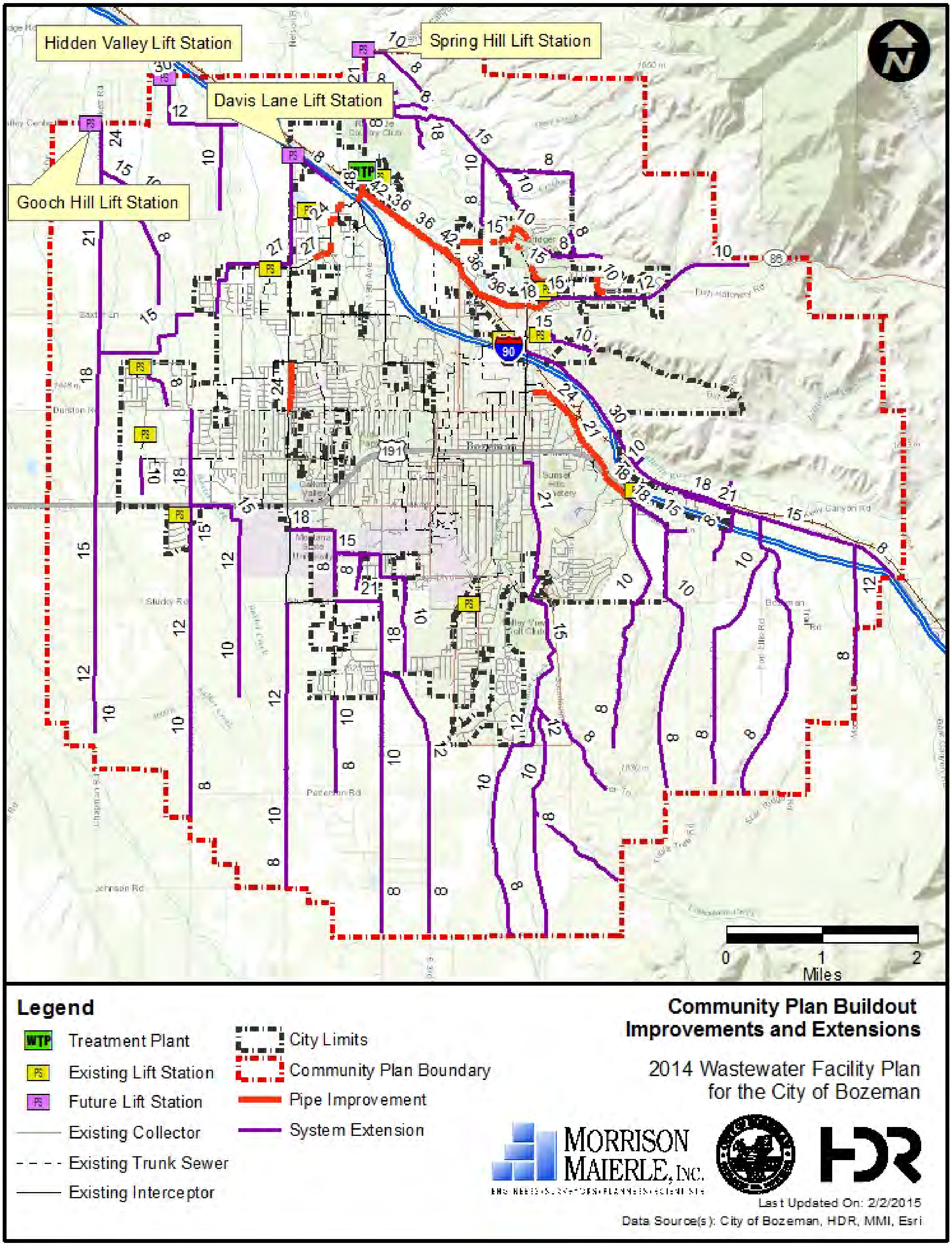
Davis Lane Lift Station
The Davis Lane Lift Station will serve development north of the Cattail Lake Lift Station and eventually the flow diverted from the Norton East Ranch Outfall Sewer. A small lift station will be required initially, but after the diversion is completed, the lift station will need to be expanded in order to pump the additional flow from the diversion.

Spring Hill Lift Station
The Spring Hill Lift Station will be located north of the WRF to collect flows from the McIlhattan area, including Deer Creek. The lift station will pump contributing flows directly south to the WRF.

Table 5-7 – Community Plan Lift Station Flows

Lift Station	Existing Capacity (gpm)	Community Plan Peak WWF (gpm)
Baxter Meadows – Norton East Ranch Full Diversion	690	900
Baxter Meadows – Norton East Ranch Partial Diversion	690	2100
Bridger Center	100	10
Burrup	450	215
Cattail Lake	225	175
Laurel Glen	450	410
Loyal Gardens	364	155
Norton Ranch	121	415
Overbrook	Unknown	15
Walker	Unknown	50
Links	160	10
Cardinal Distribution	Unknown	0
Davis Lane	N/A	4800
Gooch Hill	N/A	4300
Hidden Valley	N/A	1100
Spring Hill	N/A	2900

Figure 5-6 – Community Plan Buildout System Improvements and Extensions



5.4 RECOMMENDED IMPROVEMENTS

Due to uncertainty in development timing and locations within the Community Plan boundary, the timing of improvements is difficult to predict. The City's collection system collects flow from a contributing area and has a limited number of diversions to transfer flow from one contributing area to another. The flow in the system is dependent on the development in the contributing area and improvements should be timed based on development timing.

The following recommendations offer a view of the eventual improvements that will be required to serve the area within the Community Plan boundary. All of the recommendations should be reevaluated regularly as the developments are proposed and the land use assumed is modified. The recommendations and overall improvement plan are also shown in Figure 5-7.

Due to the uncertainty on where development will occur, the timing of when a project needs to be completed to provide capacity is difficult to predict. The recommendations below attempt to split up projects based on when they are expected to be needed. Table 5-8 provides the capacity of the sewer system at key locations identified within the collection system and the contributing basins to help identify potential bottlenecks as the City develops.

Table 5-8 – Available Capacity at Key Locations

Location	Line Capacity at 75% d/D (gpm)	Modeled Peak Flow 2014 (gpm)	Available Peak Flow Capacity 2014 (gpm)	Contributing Basins
Front Street	650	590	60	Upper Bozeman Creek
North Frontage Road	3600	2780	820	Lowe Bozeman Creek and all bains east
27th Avenue/Cattail Creek	3870	1180	2690	Baxter Creek and Cattail Creek
WWTP Interceptor	11880	9600	2280	All basins
Davis-Fowler	3300	620	2680	Baxter Creek and Cattail Creek
Norton East Ranch (15" section)	1000	40	960	Baxter Creek
Bridger Dr to North Rouse Ave	670	50	620	Bridger Creek
Bridger Creek Golf Course to North Frontage Rd	740	320	420	Bridger Creek
Valley West	1620	500	1120	Cattail Creek

5.4.1 Gravity System

All of the proposed gravity main improvements are listed in Appendix C. When the sewer line reaches capacity and needs to be replaced, the pipes should be sized based on the full development of the contributing area. The pipes are sized for the full development of the Community Plan boundary in Appendix C and should be used to size all improvements.

It is recommended that before any interceptor is replaced due to capacity constraints, the condition of the existing interceptor is evaluated. If the interceptor is in good condition, paralleling the interceptor may provide large cost saving due to reduced bypass pumping and smaller pipes used in the parallel than a replacement.

Before replacing or paralleling any of the recommended pipes, the size of the new sewer should be re-evaluated based on proposed horizontal and vertical alignments. All pipes for this analysis were sized based on assumed existing inverts and pipes slopes. If there are alternative alignments that can increase the slope, it may be possible to use a smaller diameter pipe and if the slope is decreased, a larger diameter pipe may be required. This is also true for all proposed system extensions. The pipe slopes were assumed based on the ground slope between manholes. There may be utility conflicts or different horizontal alignments that will change the assumed slope and can change the required pipe size and cost.

5.4.1.1 Near Term Interceptor Improvements

The following improvements are recommended to be completed before the existing and obligated contributing areas are developed.

Front Street Interceptor

The Bozeman Deaconess Health Services development is expected to be developed and this development will have an impact on the Front Street Interceptor. The existing system does not have enough capacity to convey the expected build-out flow and it is recommended to replace this sewer line before the full development of the property. A detailed analysis and recommendations for the Front Street Interceptor are provided in Appendix A.

North Frontage Road Interceptor

Development of the east side of land within the Community Plan Boundary will be collected by the North Frontage Road Interceptor. Portions of the interceptor are already near capacity and will be at capacity when the Existing and Obligated areas are developed. The entire interceptor should be replaced or paralleled before the hills contributing to the interceptor are developed. The size of the interceptor should also be re-evaluated as development is proposed due to the topography of the contributing area. The ultimate flow contributing to the interceptor may be much less than what is being evaluated due to challenges with developing the contributing area.

South University District

The South University District is located between Lincoln Way, University Way, and South 19th Avenue and vacant land west of Spectators. It is recommended that flow from this area should be routed west through the MSU property going west along Lincoln Ave and then south the Garfield Avenue and west to the Davis-Fowler interceptor. The South University District improvement alternatives and recommendations are detailed in Appendix B.

Bridger Drive Extension

To serve future development along Bridger Drive and Story Mill Road, it is recommended to construct 1,500 feet of sewer from the existing 12" sewer along Bridger Drive to Story Mill Road.

5.4.1.2 Long Term Interceptor Improvements

The east side of the collection system is anticipated to collect flows from the hilly areas on the east side of the plan boundary. A majority of the area has a future land use proposed as Future Urban or 5.5 du/Ac. Development of this topography can be challenging and the ultimate density of development may be much less than what was used to size improvements on the east side of the system. As development is proposed, it should be compared to the land use within the Community Plan and proposed flows should be updated before the improvements are constructed.

The following improvements are recommended in order to provide service for the entire area within the Bozeman Community Plan.

Norton East Ranch Outfall Sewer

The existing system does not have enough capacity to collect all of the flows from the full development of the Aajker Creek and Baxter Creek drainage basins. Most of this flow is proposed to be collected by a new extension in the Aajker Creek drainage basin and enter the system at the Norton East Ranch Outfall Interceptor. To reduce the impact on the existing system, when the 15-inch diameter section of the interceptor reaches capacity, a diversion to the Davis Lane Lift Station should be implemented. It is recommended to ultimately divert all of the flow to reduce the impact on the existing system.

Davis-Fowler Interceptor

The interceptor between Durston Road and West Oak Street will eventually exceed capacity as the Baxter Creek drainage basin develops. In order to convey the ultimate build-out flow, the interceptor will need to be increased from an 18-inch diameter pipe to a 24-inch diameter pipe.

27th Avenue/Cattail Creek Interceptor

The interceptor between collects flows from Baxter Creek and Cattail Creek Basins. The interceptor currently has a lot of excess capacity under existing conditions, but also provides service to large undeveloped areas. Small portions of the interceptor will eventually reach capacity as the two basins develop and will need to be paralleled or replaced.

WWTP Interceptor

As the City continues to develop, the WWTP Interceptor from I-90 to the WRF will need to be replaced or paralleled. The 30-inch interceptor will need to be replaced with a 48-inch interceptor.

5.4.2 Lift Stations

All of the lift stations have enough capacity to handle existing flows and it is recommended to perform regular maintenance on all of the lift stations and monitor run times. The pump run times will act as an indicator to show the increase in flow to the lift station.

Taking into consideration the current/short-term flow versus the long-term/build out flows, future pump station design should include phasing options and scheduled upgrades.

- Pump sizes should be upgraded when the pump runtime approaches or exceeds 12 hours per day. Initial wet wells should be sized so that the pump will operate for a reasonable length of time, but not so large that the pump would run every other day, and still be able to accommodate future build out flows. This can be accomplished by the following:

- Plan for the installation of multiple wet wells such as manholes or vaults upstream of the pump as flow increases;

- Install a wet well sized to accommodate build out flows, with a barrier to decrease the volume in the early stages. The barrier can be a steel removable barrier or a concrete wall with an opening in the bottom that can initially be plugged, and the opened at a later time, when the flow increases.

- Installing a force main large enough to handle future build out flows may initially be less expensive, however operation and maintenance costs over the long term would be more expensive, and future adverse water quality issues could be avoided. Consider the following alternatives:

  - Schedule the installation of parallel pipelines. The initial pipeline being smaller with less volume and increased velocity to accommodate current and short term flows. As flow increases install a second larger pipeline to accommodate build out flows;

  - Install both parallel pipelines at the same time, using the smaller line first, then using the second one by itself as necessitated by future flows and, then use both to accommodate build out flows.

These options may cost more up front, however will save money in the long run in operation and maintenance costs. When projected build out flows are much greater than current and short term needs, the construction/installation of parallel force mains and a wet well with a barrier, is easier and cheaper to operate and maintain, when compared to the other options. The redundancy of having a second force main is also a plus. With multiple inline wet wells there is a risk of odor, and the logistics of maintaining such a system is more involved, elevating the operation and maintenance costs. Due to the corrosive nature of the waste and residential area, the multiple inline wet well option is not considered further. It is likely that if the force main is sized for build out flow, water would need to be injected, resulting in more corrosive waste that would be more difficult and costly to treat.

For the purpose of estimating construction costs for the force mains, all force mains were sized to have a velocity of 5 fps under the ultimate peak wet weather flow scenario.

5.4.2.1 Near Term Lift Station Improvements

The existing collections system lift stations have enough capacity to convey existing flows to the lift station, but flows are expected to increase over time to the lift stations. The following improvements to lift stations are recommended to serve development within the city limits.

Norton Ranch Lift Station

As the development of the Norton Ranch Subdivision continues, the existing Norton Ranch Lift Station will reach its capacity and the lift station will need to be upgraded. The timing of this improvement will be driven by the development of the Norton Ranch Subdivision and the improvements will be constructed by the developer on an as required basis.

Davis Lane Lift Station
When development occurs north of the Cattail Lake Lift Station and south of the Davis Lane and E Valley Center Road Intersection, the Davis Lane lift station will be needed. The Davis Lane lift Station will be pumped across I-90 through a new freeway crossing. As a near term improvement, only a small lift station will be needed to serve the localized area near the station. The lift station will need to be expanded to provide service in the long term. An initial small lift station with a 200 gpm capacity is recommended until this occurs. The final sizing of this initial lift station should take into consideration the final design flow and the proposed development at the time the initial lift station is considered.

5.4.2.2 Long Term Lift Station Improvements

Most of the existing lift stations will not see a large change in flows as areas develop outside of the existing city limits. In the long term new lift stations will be required to serve areas within the Community Plan.

Baxter Meadows Lift Station
The Baxter Meadows Lift Station is expected to eventually exceed capacity as the Aajker Creek drainage basin is developed and the Norton East Ranch Outfall Sewer is constructed. The lift station runtimes should be monitored as development occurs within the Aajker Creek drainage basin and Norton Ranch Subdivision.

Davis Lane Lift Station Expansion
When the diversion for the Norton East Ranch Outfall is constructed, the Davis Lane Lift Station will see a significant increase in flows and will need to be upgraded. The extra flow will also add a large amount of flow to the existing I-90 crossing. The crossing peak flow is expected to reach 12 mgd with the build out of the existing and obligated flows and has a capacity of 17 mgd. It is recommended to construct a force main or second gravity main across the I-90 and the flow to the WRF when the diversion is constructed to the lift station. The diversion will also require the pump station to be upgraded in order to pump the additional flow.

The expansion will not have to be sized to pump the entire community plan build-out flow initially, but should be constructed to be expandable. A lift station with a 1 mgd capacity will be enough to pump flows from full build-out within the city limits.

Gooch Hill Lift Station
The Gooch Hill Lift Station will be required with the development of the northern portion of the Aajaker Basin. The lift station should be expanded when the diversion of the southern portion of the Aajaker Creek Basin is constructed.

Hidden Valley Lift Station
The Hidden Valley Lift Station is only required to serve the Baxter Creek Drainage Basin north of the Baxter Creek Subdivision. Once the contributing area is fully developed, no additional major expansions or changes to the lift station are recommended.

Spring Hill Lift Station
The Spring Hill Lift Station is recommended to be constructed when the area north of the WRF from the McIlhattan area, including Deer Creek is developed. After this area is fully developed, no other expansions are recommended for the lift station.

5.4.3 Cost Estimates

In any engineering study that develops a capital improvements program, it is necessary to make estimates of the project costs required to implement the program. To that end, basic cost data must be obtained or developed for each type of construction and system components laid out in sufficient detail to permit determination of approximate project costs.

Inherently, CIP cost estimates vary depending on the phase of the project when they are developed, which determines the level of detail and the expected accuracy of the estimate. The Association for the Advancement of Cost Engineering International (AACE International) Recommended Practices, specifically Document No. 18R-97, outlines typical cost estimate accuracies based on the overall status of the project. The cost estimates for the Transmission and Distribution Systems improvements should be considered Project Definition (Estimate Classification 5) level estimates with an expected accuracy of +100 to -50 percent.

The total project cost necessary to complete a project consists of expenditures for land acquisition, construction costs, all necessary engineering services, contingencies, and such overhead items as legal, administrative and financing services.

The cost of land acquisition is not included in the project costs presented in this report. In most cases, the construction of pipelines will not require purchase of private property or acquisition of easements. Pipeline routes, insofar as possible, follow public streets and roads. Although land or easement acquisition is a significant activity that determines whether a project occurs, the cost is generally a small portion of the overall program cost.

Construction costs cover the material, equipment, labor and services necessary to build the proposed project. Prices used in this study were obtained from a review of previous reports and pertinent sources of construction cost information. Construction costs used in this report are not intended to represent the lowest prices which may be achieved but rather are intended to represent a median of competitive prices submitted by responsible bidders.

Such factors as unexpected construction conditions, the need for unforeseen mechanical and electrical equipment, and variations in final quantities are a few examples of items that can add to planning level estimates of project cost. To cover such contingencies, an allowance of thirty percent (30%) of the construction cost has been included.

Engineering services may include preliminary investigations and reports, site and route surveys, geotechnical and foundation explorations, preparation of design drawings and specifications, engineering services during construction, construction observation, construction surveying, sampling and testing, start-up services, and preparation of operation and maintenance manuals. Overhead charges cover such items as legal fees, financing fees, and administrative costs. The costs presented in this report include an eighteen percent (18%) allowance for engineering services, legal, and administrative costs.

The cost per linear foot shown in Table 5-9 were used to estimate the cost for all pipelines recommended in this report. All estimates provided are based on 2015 prices and do not include escalation.

The estimated lift station cost does not include the cost of long term maintenance and operations and the cost is based on the size required to meet the build-out of the Bozeman Community Plan service area. The estimate also does not include escalation, the cost of the associated force main or intermediate improvements to the lift station that may be required based on the anticipated growth rate and required capacity at the time the lift station is constructed. The estimated cost and capacity for all recommended lift stations is presented in Table 5-10. Forcemain cost estimates are provided in Appendix C.

Table 5-9 – Sanitary Sewer Cost Estimates

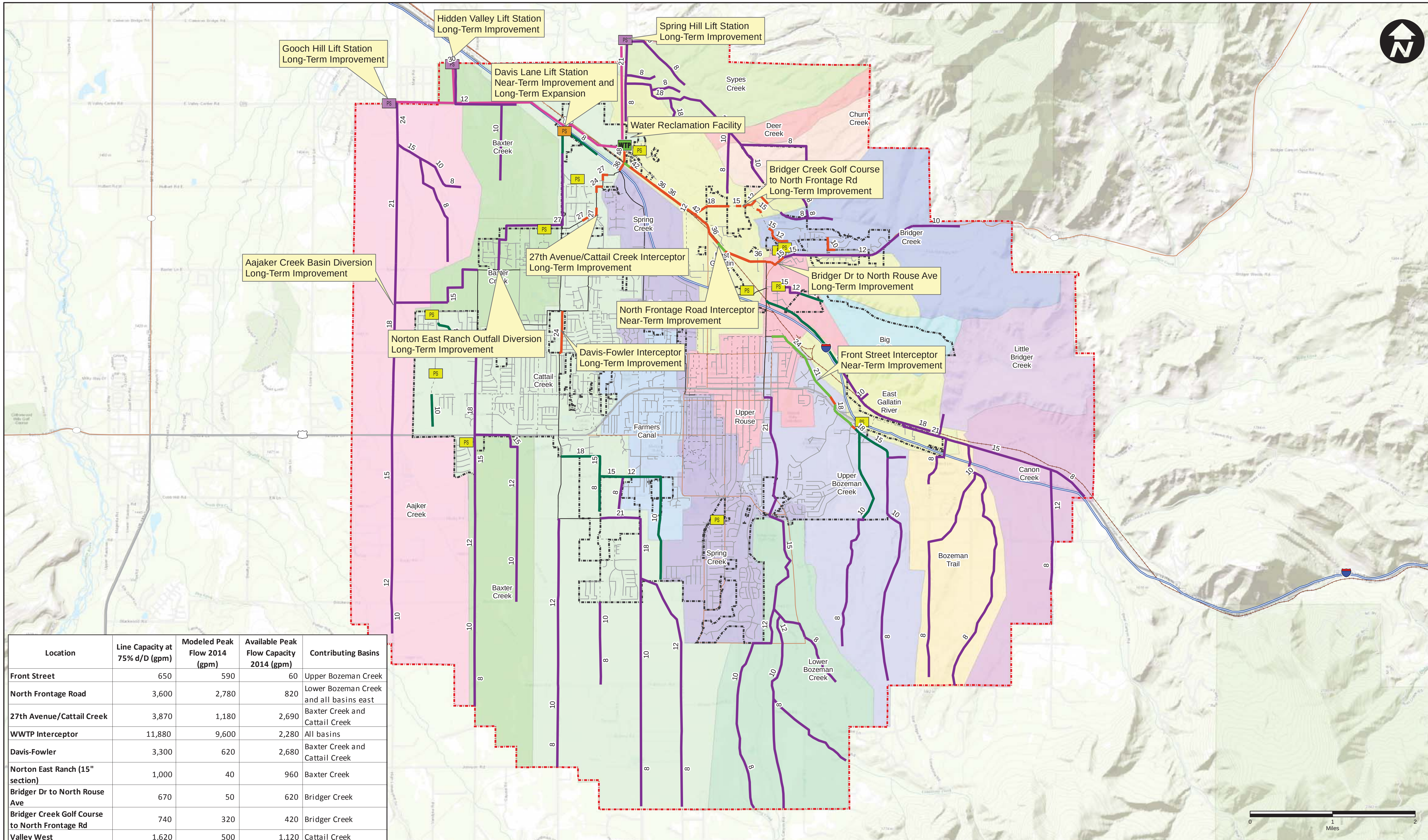
Pipe Diameter	$/lf
8	180
10	190
12	200
15	220
18	240
21	250
24	280
27	290
30	300
36	310
42	340
48	370

Table 5-10 – Estimated Lift Station Cost

Lift Station Name	Capacity (MGD)	Estimated Cost[1]
Davis Lane	6.9	$ 5,300,000
Gooch Hill	6.2	$ 4,900,000
Hidden Valley	1.5	$ 2,800,000
Spring Hill	4.2	$ 3,200,000
Norton Ranch Upgrade	0.6	$ 500,000

[1]Does not include cost of forcemain

The total estimated cost for all improvements is $114,000,000. Of this total cost, $12,000,000 is needed for improvements to the existing system and $86,000,000 is required to serve an expanded service area based on the Bozeman Community Plan.



Location	Line Capacity at 75% d/D (gpm)	Modeled Peak Flow 2014 (gpm)	Available Peak Flow Capacity 2014 (gpm)	Contributing Basins
Front Street	650	590	60	Upper Bozeman Creek
North Frontage Road	3,600	2,780	820	Lower Bozeman Creek and all basins east
27th Avenue/Cattail Creek	3,870	1,180	2,690	Baxter Creek and Cattail Creek
WWTP Interceptor	11,880	9,600	2,280	All basins
Davis-Fowler	3,300	620	2,680	Baxter Creek and Cattail Creek
Norton East Ranch (15" section)	1,000	40	960	Baxter Creek
Bridger Dr to North Rouse Ave	670	50	620	Bridger Creek
Bridger Creek Golf Course to North Frontage Rd	740	320	420	Bridger Creek
Valley West	1,620	500	1,120	Cattail Creek

Legend

- WTP Treatment Plant
- Existing Collector
- Existing Trunk Sewer
- Existing Interceptor
- PS Existing Lift Station
- PS Near-Term Lift Station Improvement
- PS Long-Term Lift Station Improvement
- City Limits
- Community Plan Boundary
- Near-Term Pipe Improvement
- Near-Term Extension
- Long-Term Pipe Improvement
- Long-Term Extension
- Long-Term Force Main

City of Bozeman
Wastewater Improvement Plan

2014 Wastewater Facility Plan
for the City of Bozeman

Morrison Maierle, Inc.

HDR

Last Updated: 5/27/2015

Appendix A – Bozeman Deaconess – Front Street Service Area Memo

MORRISON MAIERLE, INC.

memo

TO: Bob Murray, Jr. P.E.

FROM: James Nickelson, P.E.

DATE: January 7, 2015

JOB NO.: 0417.072

RE: Bozeman Deaconess – Front Street Service Area

CC:

☐ Urgent ☒ For Review ☐ Please Comment ☐ Please Reply ☐ For Your Use

As requested, we have evaluated the needed sewer collection system upgrades to serve the Bozeman Deaconess property, the remainder of The Village property and other undeveloped area that will be eventually served by the Front Street interceptor. The evaluation included a review of projected flows from the Hospital property, pipe size upgrades needed for serving the drainage area within the current city limits and pipe size upgrades needed to serve this drainage area to the limits of the sewer service planning boundary.

Projected Flows from Bozeman Deaconess Hospital
We compared the wastewater flows generated from the sewer model based on land use combined with existing flows from the Hospital and associated developed area to the detailed wastewater flows developed by HKM Engineering in July 2006 in a Master Plan document prepared for the Bozeman Deaconess Health Service Land Bank. The result of this comparison is that the values in the model are very similar to those projected in the master plan. The values in the model, based on the land use categories, provide for adequate wastewater capacity for the property.

Required Upgrades to Serve Drainage Basin – Existing City Limits Only
The required upgrades to serve the area within the existing city limits are shown on the attached figure entitled “Existing and Obligated Improvements and Extensions.” Improvements include upsizing the Front Street sewer line from Rouse to the Hospital property. The pipe sizes shown on the figure are what is required to serve the area within the existing city limits that flows through the Front Street sewer line and does not account for areas in the drainage basin that are outside of the city limits. In addition, there are two small segments of the Frontage Road interceptor that will require upsizing at some point in the future, but have available capacity for the Hospital and Village property development if other land on the east side of the community is not developed in the near term.

Comparison of Front Street Pipe Sizes for Sewer Service Planning Basin

The required upgrades, in the Front Street Area, to serve the drainage basin within the sewer service boundary are shown on the attached figure entitled "Service Area Build-out Improvements and Extensions." The pipe sizes for this scenario range from 24-inches to 21-inches. If only consideration is given to the area within the city limits the pipes sizes range from 18-inches to 15-inches.

Available Capacity for Additional Connections

The available capacity in the Front Street sewer line was evaluated based on the data in the existing system model. The existing system model is based on 2014 water use data so reflects the total connected load in 2014. The limiting section is a 400 foot segment of 12" sewer line located at the east end of Village Downtown Boulevard. The remaining capacity for this segment is 390 dwelling units based on an R-4 zoning density. The remainder of the Front Street sewer has varying limitations; however, the next limiting factor is numerous segments that have a remaining capacity of 1,020 dwelling units based on an R-4 zoning density.

Summary

The Front Street Interceptor will need to be replaced with a larger pipe to provide service to the service area. In addition, consideration should be given in the not too distance future to schedule improvements to two segments of the Frontage Road Interceptor.

In line with the recommendations of the 2007 plan, it is recommended that the Front Street Interceptor be sized to serve the full buildout of the drainage basin within the service area boundary. The pipe sizing completed as part of this analysis shows that the incremental size increase to serve the entire drainage basin compared to the drainage basin within the city limits is reasonable.

Appendix B – South University Area Sewer Service Memo

MORRISON MAIERLE, INC.

memo

TO: Bob Murray, Jr. P.E.

FROM: James Nickelson, P.E.

DATE: December 8, 2014

JOB NO.: 0417.072

RE: South University Area Sewer Service

CC:

☐ Urgent ☒ For Review ☐ Please Comment ☐ Please Reply ☐ For Your Use

As requested, we have explored options to serve the area generally between Lincoln and University Way, and South 19th and the vacant land to the west of Spectators. We reviewed the 2007 Wastewater Facility Plan, capacity of downstream pipes, and the projected flows from planned and future development. I also met with Cordell Pool, who provided much appreciated insight on the REMU project and the various scenarios he has evaluated to provide sewer capacity to the area.

Flows
As part of the Basis of Planning work completed earlier in the project, an average day flow value of 1,456 gallons per acre was assigned to the REMU zoning district. This value was based on the land use description in the zone code. Comparing values that Mr. Pool has developed as part of the master plan work for the project, it was determined that this value is the most accurate that can be developed at the facility planning level at which we are working.

2007 Wastewater Facility Plan Service Plan
The existing wastewater facility plan provides service to this area from the interceptor in Fowler. The pipe route begins at Fowler/Garfield and generally goes to the east and south to South 19th/Lincoln and includes an upsize pipe replacement in Lincoln.

Potential Pipes with Capacity
There is little potential to push additional flow due north from this area through the existing pipe network. This is confirmed based on both the model and discussions with Mr. Pool, who has done an extensive study of this area. The conclusion is that either the Fowler interceptor needs to be utilized or another extension from the North 19th Avenue area would need to be constructed. Neither upsizing an existing pipeline nor installing a new one to the North 19th Avenue area is cost effective.

Options for Serving Area

We developed a number of schematic ideas to serve the area, with the ultimate discharge point flowing into the Fowler interceptor. The ideas ranged from slight variations of the 2007 Plan to various configurations of lift stations and forcemains to provide service.

Modify Drainage Basin

This would direct wastewater flow generated east of South 19th and south of the extension of Stucky Road to the planned interceptor in Stucky Road. This option provides a number of positive changes from the existing plan. It is recommended that this be adopted in order to minimize flows from the service area under consideration.

Option A – Install Lift Station near S 19th/Garfield

This option would convey wastewater from this area and lift it from South 19th/Garfield to Fowler/Garfield with a lift station and forcemain.

From an initial capital cost perspective to serve the area under consideration, this is likely a favorable option.

From an overall wastewater service plan, this is not desirable. It would require a permanent lift station and require both the identified forcemain and future gravity mains in this area to serve both the area under consideration and the property south of Garfield between South 19th and Fowler.

This duplication of infrastructure along with the long-term cost associated with a lift station make this option less than desirable.

Option B – Install Lift Station near S 19th/Lincoln

This option is very similar to Option A. However, the location of the lift station would allow it to be abandoned at the time a gravity sewer line was installed from the west, which would generally follow the route in the 2007 plan. Initial capital costs would be similar to Option A.

Option C – Install Lift Station on Kagy near Willow Way

Installing a lift station on Kagy with a forcemain from this point to South 19th/Stucky would provide service to the South University District. It could be considered a permanent solution to serve this area.

The negative aspect of this option is that the un-sewered area between Lincoln and Kagy would still require additional capacity, which would most likely need to be provided by the Fowler interceptor. In addition, the lift station would require perpetual operation and maintenance.

Option D – Gravity Service through MSU Property

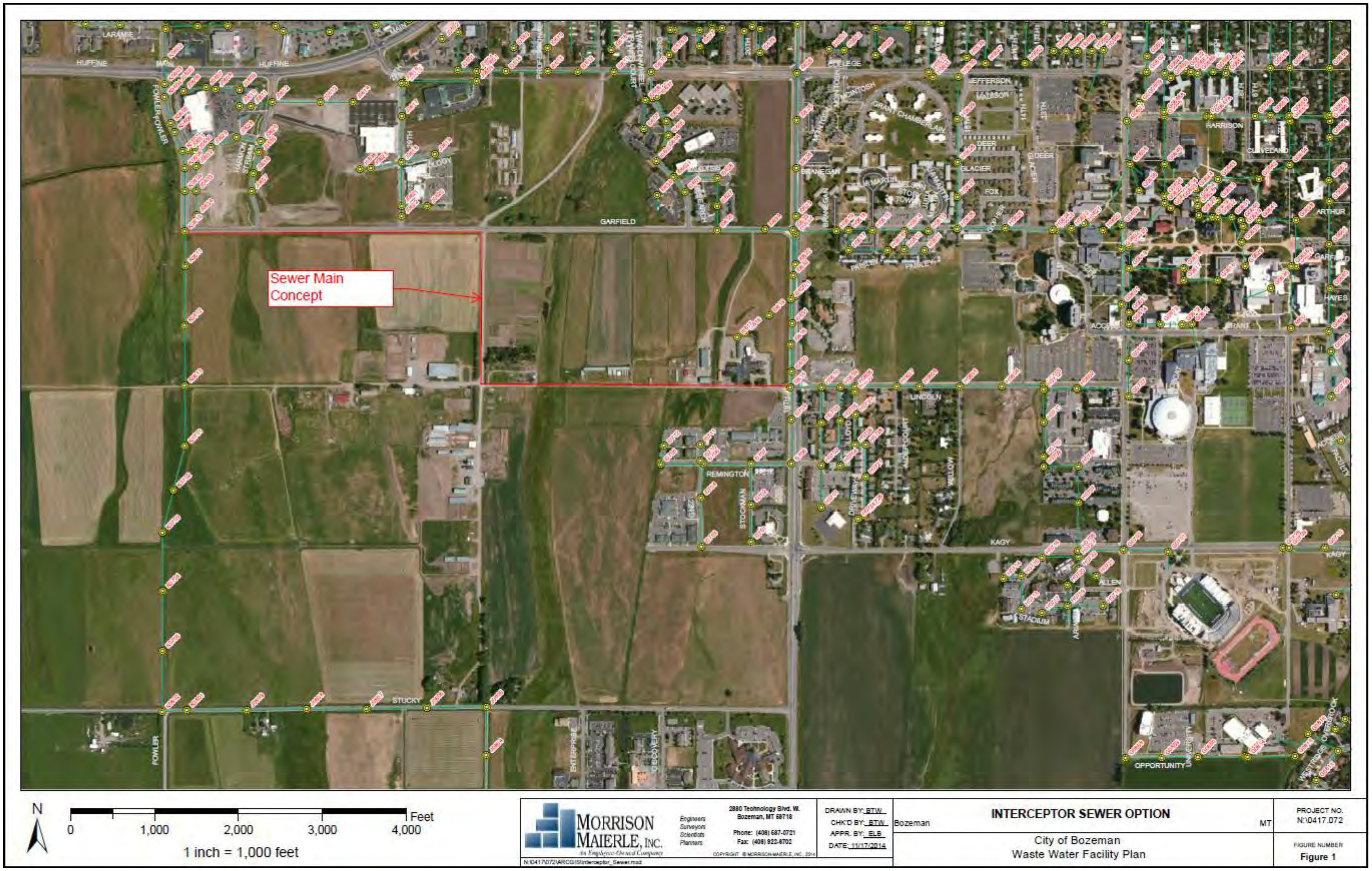
This option generally follows the concept developed in the 2007 plan. The pipe route follows travel routes within the MSU property. Based on very preliminary discussions with the ranch manager, it appears to be a feasible route for a sewer line. The route is shown on the attached exhibit.

Recommendation

Based on a community wide wastewater planning perspective we recommend that Option D be implemented.

We also recommend that the basin modification to the south be implemented. This provides for a more direct connection to the Fowler Interceptor and potentially delays the need for upsizing the sewer main in Lincoln.

As a temporary solution, Option B may have some value if funding for the permanent solution is not available and an entity will fund the lift station and forcemain in order to gain access to the City's sewer system.



Sewer Main Concept
LARAMIE
HUFFINE
MAIN
COLLEGE
GARFIELD
LINCOLN
REMINGTON
STOCKMAN
KAGY
STADIUM
STUCKY
FOWLER
ENTERPRISE
DISCOVERY
OPPORTUNITY
HARRISON
CLEVELAND
ARTHUR
GRANT
N
0
1,000
2,000
3,000
4,000
Feet
1 inch = 1,000 feet
MORRISON MAIERLE, INC.
An Employee-Owned Company
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Scientists
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N:\0417\072\ARCGIS\Interceptor_Sewer.mxd
DRAWN BY: BTW
CHK'D BY: BTW
APPR. BY: ELB
DATE: 11/17/2014
Bozeman
MT
INTERCEPTOR SEWER OPTION
City of Bozeman
Waste Water Facility Plan
PROJECT NO.
N:\0417.072
FIGURE NUMBER
Figure 1

Appendix C – Community Plan Boundary Improvements

Pipe Improvements

Front Street Interceptor

Conduit ID	Peak Flow (MGD)	Length (ft)	Existing Diameter (in)	New Diameter (in)	Estimated Cost
C0507_D0501	3.8	400	10	18	$ 100,000
C0508_C0507	3.5	300	8	18	$ 60,000
C0524_C0508	2.7	300	8	18	$ 70,000
D0415_D0419	4.4	400	12	21	$ 100,000
D0416_D0415	4.4	400	12	18	$ 90,000
D0417_D0416	4.3	400	12	18	$ 90,000
D0418_D0417	4.3	400	12	18	$ 90,000
D0419_D0420	4.5	400	12	21	$ 100,000
D0420_E0453	4.5	100	12	21	$ 30,000
D0421_E0442	4.5	200	14	24	$ 70,000
D0501_D0502	3.8	400	10	18	$ 100,000
D0502_D0508	3.8	100	10	18	$ 10,000
D0508_D0509	4.3	300	12	18	$ 80,000
D0509_D0510	4.3	300	12	18	$ 80,000
D0510_D0511	4.3	300	12	18	$ 70,000
D0511_D0512	4.3	300	12	18	$ 70,000
D0512_D0418	4.3	300	12	18	$ 80,000
E0319_E0318	4.5	400	14	24	$ 110,000
E0320_E0319	4.5	400	14	24	$ 110,000
E0323_E0320	4.5	400	14	24	$ 110,000
E0324_E0323	4.5	400	14	24	$ 110,000
E0325_E0324	4.5	400	14	24	$ 110,000
E0326_E0325	4.5	400	14	24	$ 110,000
E0442_E0443	4.5	200	14	24	$ 50,000
E0443_E0326	4.5	400	14	24	$ 100,000
E0453_D0421	4.5	300	14	24	$ 80,000
				Total Cost:	**$ 2,180,000**

Davis-Fowler Interceptor

Conduit ID	Peak Flow (MGD)	Length (ft)	Existing Diameter (in)	New Diameter (in)	Estimated Cost
J0301_J03118	8.7	100	18	24	$ 20,000
J0302_J0303	8.7	400	18	24	$ 130,000
J0303_J0304	8.7	400	18	24	$ 120,000
J0304_J0305	8.7	500	18	24	$ 130,000
J0305_J0306	8.7	500	18	24	$ 130,000
J0306_J0307	8.8	400	18	24	$ 120,000
J03118_J0302	8.7	400	18	24	$ 110,000
				Total Cost:	**$ 760,000**

WWTP Interceptor

Conduit ID	Peak Flow (MGD)	Length (ft)	Existing Diameter (in)	New Diameter (in)	Estimated Cost
I5011_I5025	45.6	500	30	48	$ 170,000
I5025_I5026	45.6	500	30	48	$ 190,000
I5026_WWTP	64.3	200	30	48	$ 60,000
				Total Cost:	**$ 420,000**

North Frontage Road Interceptor

Conduit ID	Peak Flow (MGD)	Length (ft)	Existing Diameter (in)	New Diameter (in)	Estimated Cost
F0101_G0131	9.5	400	20	36	$ 130,000
F0102_F0101	9.5	400	20	36	$ 140,000
F0103_F0102	9.5	400	20	36	$ 140,000
F0104_F0103	9.5	400	20	36	$ 130,000
F0108_F0104	9.5	500	20	36	$ 150,000
F0109_F0119	9.8	0	20	36	$ 20,000
F0110_F0109	9.8	0	20	36	$ 10,000
F0110_F0116	12.9	0	20	36	$ 10,000
F0111_G0132	13.2	400	20	36	$ 130,000
F0112_F0111	13.2	500	20	36	$ 140,000
F0113_F0112	13.2	500	20	36	$ 140,000
F0114_F0113	13.2	400	20	36	$ 120,000
F0115_F0114	13.2	500	20	36	$ 150,000
F0116_F0115	12.9	500	20	36	$ 160,000
F0118_F0108	9.8	300	20	36	$ 80,000
F0119_F0118	9.8	200	20	36	$ 60,000
G0006_G0007	23.1	400	20	36	$ 140,000
G0007_G0008	23.1	100	30	36	$ 20,000
G0008_G0009	23.1	500	30	36	$ 160,000
G0010_G0011	25.3	500	30	42	$ 170,000
G0011_H0016	25.5	200	30	36	$ 70,000
G0132_G0133	23.1	300	20	36	$ 90,000

G0133_G0134	23.1	400	20	36	$ 130,000
G0134_G0135	23.1	400	20	36	$ 130,000
G0135_G0136	23.1	400	20	36	$ 130,000
G0136_G0137	23.1	300	20	36	$ 90,000
G0137_G0006	23.1	300	20	36	$ 90,000
H0001_H5010	12.7	500	20	36	$ 160,000
H0002_H5011	12.8	500	20	36	$ 160,000
H0003_H0001	12.7	400	20	36	$ 120,000
H0004_H0002	12.8	400	20	36	$ 120,000
H0005_H0004	12.8	400	20	36	$ 120,000
H0006_H0003	12.7	400	20	36	$ 120,000
H0007_H0006	12.7	300	20	36	$ 90,000
H0008_H0005	12.8	300	20	36	$ 90,000
H0009_H0007	12.7	400	20	36	$ 110,000
H0010_H0008	12.8	300	20	36	$ 110,000
H0011_H0009	12.7	400	20	36	$ 110,000
H0012_H0010	12.8	400	20	36	$ 110,000
H0013_H0012	12.8	200	20	36	$ 50,000
H0015_H0013	12.8	100	30	36	$ 20,000
H0016_H0015	12.8	0	30	36	$ 10,000
H0016_H0017	12.7	0	20	36	$ 10,000
H0017_H0011	12.7	300	20	36	$ 80,000
H5008_I5014	25.5	500	20	36	$ 150,000
H5009_H5008	25.5	500	20	36	$ 140,000
H5010_H5009	12.7	0	20	36	$ 10,000
H5011_H5009	12.8	0	20	36	$ 10,000
I5012_I5011	25.6	400	20	42	$ 150,000
I5013_I5012	25.6	500	20	42	$ 170,000
I5014_I5013	25.5	400	20	42	$ 140,000
				Total Cost:	**$ 5,290,000**

27th Avenue/Cattail Creek Interceptor

Conduit ID	Peak Flow (MGD)	Length (ft)	Existing Diameter (in)	New Diameter (in)	Estimated Cost
I5006_I5010	20.1	300	30	36	$ 100,000
I5010_I5011	20.1	200	30	36	$ 70,000
J0001_J5004	9.8	400	24	27	$ 120,000
J0005_J0004	9.6	300	24	27	$ 80,000
J0006_J0005	9.6	300	24	27	$ 80,000
J0013_J0014	9.3	100	24	27	$ 20,000
J0014_J0015	9.3	200	24	27	$ 50,000
J0016_J0013	9.3	400	24	27	$ 110,000
J5001_I5009	10.3	300	24	27	$ 90,000

J5002_J5001	10.2	400	24	27	$	110,000
J5004_J5003	10	400	24	27	$	120,000
				Total Cost:	**$**	**950,000**

Bridger Creek Golf Course to North Frontage Rd

Conduit ID	Peak Flow (MGD)	Length (ft)	Existing Diameter (in)	New Diameter (in)	Estimated Cost	
E0081_F0089	0.7	400	8	10	$	70,000
E0131_E0132	1.3	500	10	12	$	90,000
E0132_E0133	1.3	200	10	12	$	40,000
E0133_E0134	1.3	200	10	12	$	30,000
E0134_E0135	1.3	200	10	12	$	40,000
E0135_E0136	1.3	200	10	12	$	40,000
E0136_E0148	1.3	200	10	12	$	50,000
E0148_E0152	1.3	300	12	15	$	70,000
F0049_F0059	1.3	200	12	15	$	40,000
F0050_G0012	2.1	300	15	18	$	70,000
F0052_F0051	2.1	400	12	15	$	80,000
F0054_F0053	2	200	12	15	$	40,000
F0055_F0054	2	100	12	15	$	30,000
F0056_F0055	2	300	12	15	$	60,000
F0059_F0058	1.3	300	12	15	$	60,000
F0066_F0056	0.7	300	10	12	$	50,000
F0074_F0075	0.7	100	8	10	$	20,000
F0075_F0076	0.7	100	8	10	$	20,000
F0076_F0066	0.7	200	10	12	$	40,000
F0086_F0074	0.7	200	8	10	$	30,000
F0087_F0086	0.7	100	8	10	$	20,000
F0088_F0087	0.7	200	8	10	$	30,000
F0089_F0088	0.7	400	8	10	$	70,000
G0012_G0013	2.1	400	15	18	$	100,000
G0013_G0014	2.1	400	15	18	$	100,000
G0014_G0015	2.1	400	15	18	$	80,000
G0015_G0016	2.1	200	15	18	$	60,000
G0016_G0017	2.1	300	15	18	$	70,000
G0017_G0010	2.1	400	15	18	$	90,000
				Total Cost:	**$**	**1,590,000**

Bridger Dr to North Rouse Ave

Conduit ID	Peak Flow (MGD)	Length (ft)	Existing Diameter (in)	New Diameter (in)	Estimated Cost
E0110_E0113	2.5	100	12	18	$ 20,000
E0113_E0114	2.6	300	12	18	$ 80,000
E0114_E0115	2.6	300	12	18	$ 70,000
E0115_F0109A	2.6	300	12	18	$ 80,000
E0116_E0110	2.5	300	12	15	$ 70,000
				Total Cost:	**$ 320,000**

Collection System Extensions

Aajker Creek Basin

Conduit ID	Peak Flow (MGD)	Length (ft)	Diameter (in)	Estimated Cost
CDT-125	3.4	2500	18	$ 610,000
P-5092	1	2200	10	$ 410,000
P-5093	1.7	2100	12	$ 430,000
P2-3120	2.8	6100	15	$ 1,340,000
P2-3121	3.8	5500	15	$ 1,210,000
P2-3122	4.3	5300	18	$ 1,270,000
P2-3125	4.5	7600	21	$ 1,900,000
P2-3126	6.7	2600	24	$ 730,000
P2-3127	0.2	7200	8	$ 1,300,000
P2-3128	1.6	1700	15	$ 370,000
P2-5125	0.4	1200	8	$ 210,000
P2-5126	0.7	2300	10	$ 440,000
			Total Cost:	**$ 10,220,000**

South University District

Conduit ID (Char)	Peak Flow (MGD)	Length (ft)	Diameter (in)	Estimated Cost
P-5114	0.5	900	15	$ 190,000
P-5174	0.4	1100	12	$ 220,000
P2-3157	0.7	2500	18	$ 600,000
P2-3245	0.7	500	15	$ 110,000
			Total Cost:	**$ 1,120,000**

Baxter Creek Basin

Conduit ID	Peak Flow (MGD)	Length (ft)	Diameter (in)	Estimated Cost
CDT-103	2.3	3100	15	$ 690,000
CDT-61	0.2	2100	10	$ 390,000
P-5110	1.5	800	15	$ 170,000
P2-3131	1.8	7500	12	$ 1,510,000
P2-3132	3.7	5300	18	$ 1,270,000
P2-3148	0	2200	8	$ 390,000
P2-3174	1.1	2900	12	$ 570,000
P2-3242	1.4	400	15	$ 100,000
P2-3244	1.5	2200	18	$ 540,000
P2-3251	1.3	1800	12	$ 360,000
P2-5094	0	3500	8	$ 630,000
P2-5095	1.3	3200	10	$ 600,000
P2-5110	0.8	3100	10	$ 580,000
P2-5111	0.4	2100	10	$ 400,000
			Total Cost:	**$ 8,200,000**

Cattail Creek Basin

Conduit ID	Peak Flow (MGD)	Length (ft)	Diameter (in)	Estimated Cost
CDT-89	0.4	4000	10	$ 770,000
CDT-91	1.4	2600	12	$ 530,000
CDT-93	1.4	5200	12	$ 1,040,000
CDT-95	0.3	2200	10	$ 420,000
CDT-97	2.5	3800	18	$ 920,000
P-5113	0.1	2100	8	$ 380,000
P-5185	0.6	3000	12	$ 610,000
P2-3165	2.5	2400	21	$ 610,000
P2-3166	2.2	5400	12	$ 1,080,000
P2-3167	2.1	5400	12	$ 1,080,000
P2-3170	0.4	4800	8	$ 870,000
P2-3171	0.7	5600	10	$ 1,070,000
P2-3172	1	4100	10	$ 780,000
P2-3176	0.2	4800	8	$ 870,000
P2-3247	0.1	3000	8	$ 540,000
P2-5096	0.3	2900	8	$ 510,000
P2-5097	0.8	2300	10	$ 430,000
P2-5127	0.1	3100	8	$ 560,000
			Total Cost:	**$ 13,070,000**

Lower Bozeman Creek Basin

Conduit ID (Char)	Peak Flow (MGD)	Length (ft)	Diameter (in)	Estimated Cost
CDT-101	1	1900	12	$ 370,000
CDT-105	2.5	5000	21	$ 1,240,000
CDT-99	0.4	2300	8	$ 420,000
P2-3179	2.5	1500	15	$ 330,000
P2-3180	1.4	600	15	$ 130,000
P2-3181	1	2400	12	$ 490,000
P2-3183	1	6700	10	$ 1,270,000
P2-3184	0.3	5200	8	$ 940,000
P2-3185	0.2	3000	8	$ 540,000
P2-3186	0.3	4100	8	$ 730,000
P2-3187	0.9	3800	10	$ 720,000
P2-3188	0.4	3600	8	$ 660,000
P2-3191	2.5	5200	15	$ 1,140,000
P2-3192	2.5	2800	15	$ 620,000
P2-5098	0.1	3600	8	$ 650,000
P2-5099	0.4	2700	8	$ 480,000
			Total Cost:	**$ 10,730,000**

Upper Bozeman Creek Basin

Conduit ID (Char)	Peak Flow (MGD)	Length (ft)	Diameter (in)	Estimated Cost
CDT-79	2.7	300	18	$ 80,000
P2-3193	0.3	3600	8	$ 650,000
P2-3194	0.9	5300	10	$ 1,010,000
P2-3196	0.5	5700	8	$ 1,020,000
P2-3197	1.5	5300	10	$ 1,020,000
P2-3198	1.6	3900	10	$ 740,000
P2-3199	0.3	1900	8	$ 350,000
P2-3200	0.4	2000	8	$ 350,000
P2-3201	0	1200	10	$ 220,000
P2-3202	0.3	5300	8	$ 950,000
P2-3203	1.1	9100	10	$ 1,740,000
P2-3204	0.2	6400	8	$ 1,150,000
P2-3205	1	8600	10	$ 1,640,000
P2-3206	2.6	2000	15	$ 450,000
P2-3207	0.6	3100	8	$ 560,000
P2-3208	1.3	4600	12	$ 920,000
P2-3209	0.2	2800	8	$ 500,000
P2-3210	3.7	5500	15	$ 1,210,000
P2-3212	6.3	1500	21	$ 370,000
P2-3256	0.9	3500	10	$ 670,000
P2-3257	1	2300	10	$ 430,000
P2-3258	1.7	2600	15	$ 560,000
			Total Cost:	**$ 16,590,000**

Little Bridger Creek and Southern Gallatin River Basin

Conduit ID (Char)	Peak Flow (MGD)	Length (ft)	Diameter (in)	Estimated Cost
CDT-57	0.8	600	10	$ 120,000
CDT-59	2.7	3600	18	$ 870,000
CDT-81	10.1	3100	30	$ 930,000
P2-3213	9.3	3200	30	$ 970,000
P2-3216	10.2	3100	30	$ 940,000
P2-3250	9.9	2800	30	$ 830,000
			Total Cost:	**$ 4,660,000**

Big Gulch Basin

Conduit ID (Char)	Peak Flow (MGD)	Length (ft)	Diameter (in)	Estimated Cost
CDT-55	0.5	1500	10	$ 290,000
			Total Cost:	**$ 290,000**

Bridger Creek Basin

Conduit ID (Char)	Peak Flow (MGD)	Length (ft)	Diameter (in)	Estimated Cost
P-5090	1.8	1300	10	$ 240,000
P-5091	1.5	1800	10	$ 330,000
P2-3217	2.1	3200	12	$ 640,000
P2-3218	2.5	4400	12	$ 880,000
P2-3219	2.5	1300	15	$ 280,000
P2-3222	0.5	1600	8	$ 290,000
P2-3223	0.6	1900	8	$ 340,000
P2-3224	0.1	500	8	$ 90,000
			Total Cost:	**$ 3,090,000**

Spring Hill Lift Station

Conduit ID (Char)	Peak Flow (MGD)	Length (ft)	Diameter (in)	Estimated Cost
CDT-121	3.4	1500	18	$ 370,000
CDT-63	0.6	4100	18	$ 980,000
P2-3225	0.9	3100	10	$ 600,000
P2-3226	0.1	3000	8	$ 540,000
P2-3227	0.2	1100	10	$ 210,000
P2-3228	1.4	1000	15	$ 220,000
P2-3229	1.7	700	15	$ 160,000
P2-3231	0	1900	8	$ 350,000
P2-3232	3.5	1900	18	$ 470,000
P2-3233	0.1	1900	8	$ 350,000
P2-3235	0	2400	8	$ 430,000
P2-3236	3.6	500	21	$ 130,000
P2-3237	3.7	2200	21	$ 540,000
P2-3261	0.8	4600	8	$ 820,000
P2-5100	2.5	2000	15	$ 440,000
P2-5101	2.9	2000	18	$ 490,000
P2-5102	0.3	2500	8	$ 450,000
P2-5105	0.5	2400	10	$ 450,000
			Total Cost:	**$ 8,000,000**

Hidden Valley Lift Station

Conduit ID (Char)	Peak Flow (MGD)	Length (ft)	Diameter (in)	Estimated Cost
P2-3140	0.6	4900	10	$ 920,000
P2-3142	0.2	1000	10	$ 180,000
P2-3143	1.3	2900	12	$ 590,000
P2-3144	1.5	2600	12	$ 530,000
			Total Cost:	**$ 8,000,000**

Aajker Creek Diversion

Conduit ID (Char)	Peak Flow (MGD)	Length (ft)	Diameter (in)	Estimated Cost
P-5140	1.7	2100	15	$ 470,000
P-5175	1.6	3400	15	$ 740,000
P2-3137	1.6	1900	15	$ 420,000
			Total Cost:	**$ 1,630,000**

Norton East Ranch Diversion

Conduit ID (Char)	Peak Flow (MGD)	Length (ft)	Diameter (in)	Estimated Cost
CDT-115	6.2	5700	27	$ 1,660,000
P2-3139	6.2	5700	27	$ 1,660,000
			Total Cost:	**$ 3,320,000**

Davis Lane Lift Station

Conduit ID (Char)	Peak Flow (MGD)	Length (ft)	Diameter (in)	Estimated Cost
CDT-83	6.4	1600	27	$ 450,000
P2-3146	0.2	2700	8	$ 480,000
			Total Cost:	**$ 930,000**

Force Main Extensions

Conduit ID	Description	Peak Flow (MGD)	Length (ft)	Diameter (in)	Estimated Cost
CDT-117	Hidden Valley Lift Station to Gooch Hill Connection	1.6	2,214	12	$ 440,000
CDT-69	Gooch Hill Lift Station to Hidden Valley Connection	6.7	4,024	18	$ 970,000
CDT-71	Gooch Hill Connection to Davis Lane Connection	8.3	7,786	21	$1,950,000
CDT-73	Spring Hill Lift Station to WTP	4.2	6,596	15	$1,450,000
CDT-75	Davis Lane Connection to WRF	14.8	4,211	27	$1,220,000
CDT-87	Davis Lane Lift Station to Gooch Hill Connection	6.9	285	21	$ 70,000
				Total Cost:	**$6,100,000**

Bozeman Wastewater Collection Facilities Plan Update

Chapter 6

Summary Recommendations and Capital Improvement Summary

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6.1 INTRODUCTION

The wastewater collection system was evaluated to identify deficiencies in capacity. The system was evaluated under the following flow scenarios to prioritize improvements:

- Existing Flows
- Existing and Build-out within City Limits
- Existing and Build-out within the Community Plan Boundary

This chapter summarizes and prioritizes the recommended improvements based on the analysis of the different flow scenarios.

6.2 COLLECTION SYSTEM IMPROVEMENTS

Base on the three scenarios evaluated improvements to the existing collection system were grouped into the following two categories to address the collection system needs in the future:

- Category 1: Required to serve growth within the existing City Limits
- Category 2: Required to serve growth outside the existing City Limits

The recommended improvements do not include any ongoing maintenance within the existing system. The City has been investing in repairing and replacing older and deteriorating sewer lines and manholes. This can have an impact on the amount of infiltration and wet weather inflow entering the collection system. It is recommended that the City continues to invest in repairing and replacing damaged sewer lines and continue to take steps to reduce infiltration into the system.

6.2.1 Category 1 Improvements

The projects summarized in Table 6-1 are recommended to serve future growth within the existing city limits. This includes providing new sewer to areas that currently aren't developed and only includes pipes sized 12-inches and greater. The analysis performed to identify the projects and more details on the projects is provided in Chapter 5 of this report.

Table 6-1 – Category 1 Improvements

Project Name	Improvements Description	Probable Cost
Front Street Interceptor	Replace or parallel 8,500 feet of sewer along Front St and Haggerty Ln from E Tamarack St to Ellis St	$ 2,180,000
North Frontage Road Interceptor	Replace or parallel 11,500 feet of the North Frontage Rd Interceptor between Springhill Rd and Bridger Dr	$ 5,290,000
South University District	New 5,000 feet of sewer to divert South University District development flows to the Davis-Fowler Interceptor	$ 1,120,000
Norton Ranch Lift Station	Increase capacity at the Norton Ranch Lift Station to support further development of the Norton Ranch Subdivision	$ 500,000
Davis Lane Lift Station	Construct small initial Davis Lane Lift Station to serve area north of the Cattail Lake Lift Station	$ 500,000
Bridger Drive Extension	Install a new 2.5 mgd capacity sewer along Bridger Drive from Birdie Drive to Story Mill Rd	$ 300,000

Additionally, 29,800 feet of sewer extensions were identified to serve currently undeveloped areas within the city limits that were less than 12-inches in diameter. The total probable cost of these extensions was estimated at $6,680,000.

6.2.2 Category 2 Improvements

The projects summarized in Table 6-1 are recommended to serve future growth outside the existing city limits. The analysis performed to identify the projects and more details on the projects is provided in Chapter 5 of this report. Many of these projects will be required outside of the City's normal 20 year planning period and should be re-evaluated as the land use and growth patterns are updated. It is important to note that if development is concentrated in a basin contributing to the project, the project may be required sooner to serve new development.

Table 6-2 – Category 2 Improvements

Project Name	Improvements Description	Probable Cost
Davis-Fowler Interceptor	Replace or parallel 2700 feet of the Davis-Fowler Interceptor between Durston Rd and W Oak St	$ 760,000
WWTP Interceptor	Replace or parallel 1200 feet of sewer from I-90 to the WRF	$ 420,000
27th Avenue/Cattail Creek	Replace or parallel 3300 feet of the WWTP Interceptor Sewer	$ 950,000
Bridger Creek Golf Course to North Frontage Rd Sewer	Replace or parallel 7700 feet of sewer around the Bridger Creek Golf Course and from the Course to North Frontage Rd	$ 1,590,000
Bridger Dr to North Rouse Ave	Replace or parallel 1300 ft of sewer along Bridger Drive between Birdie Drive and North Rouse Avenue	$ 320,000
Norton East Ranch Diversion	Divert flow from the Norton East Ranch Sewer to the Davis Lane Lift Station	$ 3,320,000
Davis Lane Lift Station Expansion	Expand the Davis Lane Lift Station for the Norton East Ranch Diversion flows	$ 5,300,000
Gooch Hill Lift Station and Forcemain	Construct a 6.2 mgd lift station and forcemain	$ 7,820,000
Hidden Valley Lift Station and Forcemain	Construct a 1.5 mgd lift station and forcemain	$ 5,190,000
Spring Hill Lift Station and Forcemain	Construct a 4.2 mgd lift station and forcemain	$ 4,650,000

In addition to the improvements described above, an additional 380,000 feet of trunk lines at an estimated cost of $78,000,000 will be required to serve areas outside of the current city limits and within the Community Plan Boundary. All of the expansions to the City is heavily dependent on where development occurs and service is required.

6.3 CONCLUSIONS

The City's existing collection system has enough capacity to serve the City's existing population without any major improvements. With the City expected to grow in population, it will be important for the City to stay ahead of the growth and provide capacity for the growth. This document has shown that the City has been providing adequate capacity and has room for growth in many areas of the City. Five projects were identified to provide additional capacity in portions of the system that are expected to reach capacity as the City grows.

It is important that the City continue to monitor the impacts that expansion of the service area has on the existing collection system and ensure that needed capacity improvements are realized with the expansion of the service area. This plan provides the framework to allow for the service area to be expanded, however scheduling of the improvements is dependent on the need to provide sewer service to specific areas. Sixteen specific projects have been identified to provide additional capacity within the existing collection system area. The timing of such projects are dependent on the location of future development projects and their impact on the collection system capacity.

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