CITY OF MISSOULA

2019 | WASTEWATER FACILITY PLAN

Approval
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By: Missoula City Council
Record #: RES 8507

Morrison Maierle
engineers • surveyors • planners • scientists

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THE GARDEN CITY
MISSOULA
HUB OF FIVE VALLEYS

WASTEWATER FACILITY PLAN

CHAPTER 1 - INTRODUCTION

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CHAPTER 1 INTRODUCTION

1.1. BACKGROUND

The City of Missoula has owned and operated the wastewater utility since 1962. Throughout its operation, the utility has been well managed and maintained and currently has one of the lowest sewer rates in the state.

The Wastewater\Compost Utility strategic goals are

- Fiscal Sustainability
- Harmonious Natural and Built Environments.

The Utility's strategies to meet these goals are to

- maintain and improve the level of service to citizens,
- work toward sustaining and diversifying fiscal resources, and
- to make sure that the natural and built environments continue to represent Missoula's values of clean water and clean air.

Guided by these strategic goals, the Wastewater/Compost Utility has been proactive in its approach to system improvements in order to stay abreast of a growing population and industrial/commercial development, anticipated regulatory changes, and demands posed by an aging system. Particular progress has been made to include sustainable projects and approaches that allow for beneficial reuse of waste products. Over the past 20 years, the City has completed a number of planning, design, and construction projects for the City's wastewater facilities. A summary of the major wastewater system projects undertaken by the City during this period is shown in the following figure.

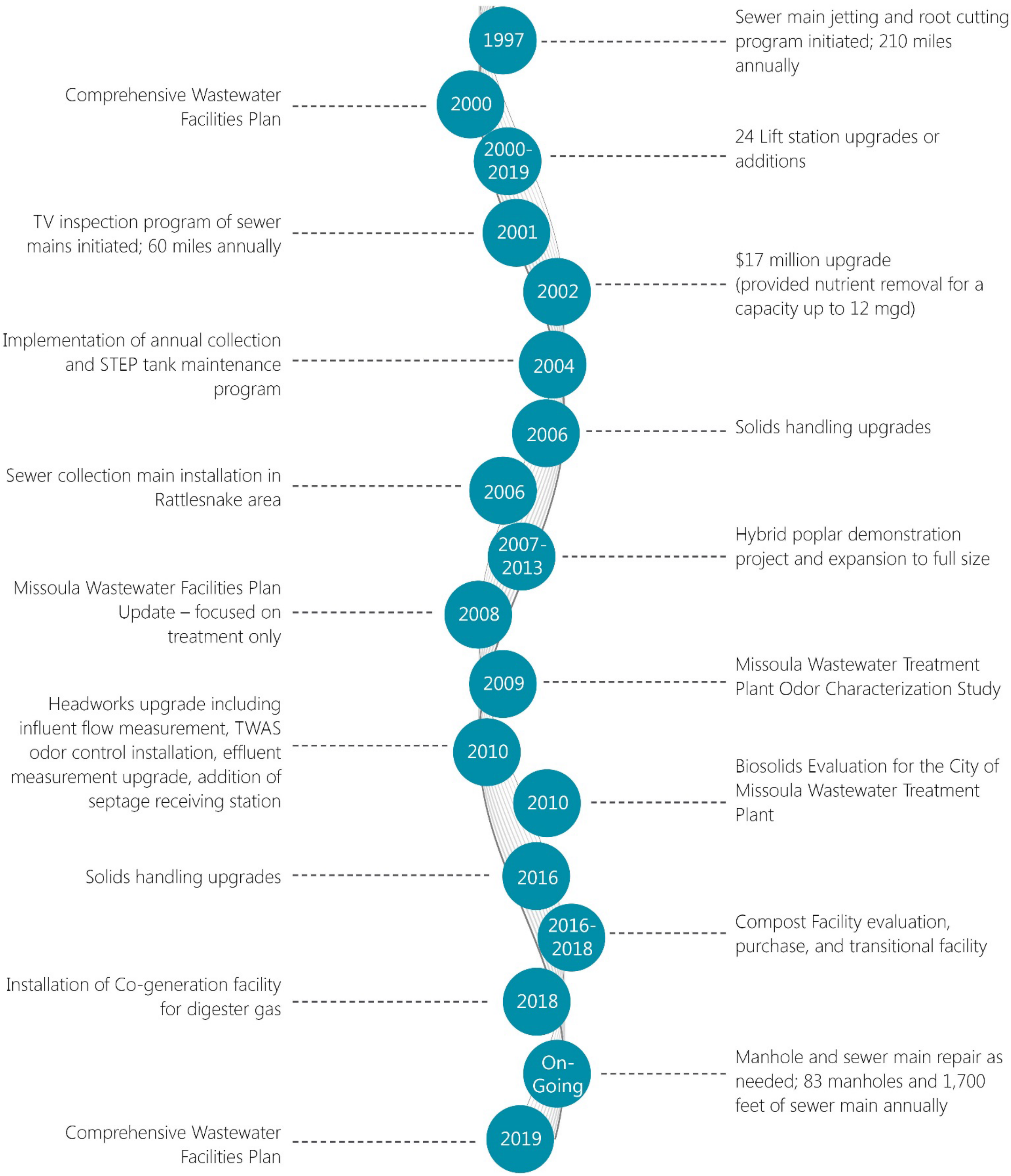

Figure 1-1: Summary of Major Wastewater System Planning, Design, and Construction Projects

1.2. REPORT OBJECTIVES

The purpose of the 2019 Wastewater Facility Plan is to describe the existing system, evaluate the capacity and condition of the system, identify system deficiencies, develop recommendations for improvements and provide effective tools for City staff to continue to evaluate system capacity and plan for future needs for treatment and collection system infrastructure.

Through meetings with City staff, the objectives of the facility plan were identified as follows:

- Develop planning values for future wastewater flows and loads as they pertain to the collection and treatment systems based on historical wastewater data and population growth projections provided by the City of Missoula
- Develop a collection system model to be used as a planning tool by the City to evaluate future collection system extensions
- Asses the current capacities of the collection system
- Assess the potential future capacities of the collection system within the 20-year planning period
- Assess the improvements required to convey potential future flows
- Identify opportunities to simplify or eliminate existing lift stations
- Identify and evaluate future WWTP effluent permit requirements and limitations with the existing facility
- Prepare a WWTP process model to reflect current performance and model future conditions
- Assess the existing and future WWTP hydraulic and treatment capacity
- Develop WWTP treatment and disposal alternatives to meet future wastewater flow, load, and permit requirements
- Provide an updated CIP for collection and treatment systems

The 2019 Wastewater Facility Plan comprises the evaluation of the existing and future system needs for the Wastewater Utility and preparation of an updated capital improvements plan (CIP). The Facility Plan focuses on the entire collection system and the wastewater treatment plant (WWTP) with incorporation by reference of the recently acquired composting facility. The collection system evaluation was performed using a hydraulic model developed to evaluate the current system capacity and simulate capacity needed to accommodate future growth within the service area.

The wastewater treatment plant evaluation includes condition and capacity evaluation of existing structures and equipment, preparation of a BioWin process model to determine treatment capacity under different effluent limit scenarios the City may experience over the planning horizon, and presentation of alternatives addressing identified deficiencies. Recommendations for future improvements were developed and prioritized to assist the City of Missoula in planning for identified upgrades of the WWTP and the collection system over a 20-year planning period from 2017 through 2037.

1.3. REPORT ORGANIZATION

This report is organized into chapters as follows:

- Executive Summary – Graphical format that can be used as a tool to inform City Council members and the public about the planning efforts
- Chapter 1: Introduction – Background, report objectives and organization of Plan
- Chapter 2: Basis of Planning – Service area definition, population, wastewater flows and loads for current and future conditions; regulatory framework and potential future permit limit changes
- Chapter 3: Collection System Data – Data sources, methods of data collection, and recommendation for ongoing database management
- Chapter 4: Existing Collection System Description – Inventory of existing system and summary of known deficiencies
- Chapter 5: Existing and Near-Term Collection System Analysis – Model assumptions and results for the existing collection system and near-term conditions
- Chapter 6: Future Collection System Analysis – Model assumptions and results for 2037 future collection system
- Chapter 7: Existing Treatment Plant Description and Capacity Assessment – Description, performance, condition and capacity assessment of existing equipment, structures, and processes
- Chapter 8: Future Treatment Plant Analysis and Alternatives – Alternative development and analysis addressing deficiencies identified in Chapter 7 and potential lower effluent limits addressed in Chapter 3
- Chapter 9: Capital Improvements Plan

Supporting information is provided in appendices following each chapter as appropriate.

WASTEWATER FACILITY PLAN

CHAPTER 2 - BASIS OF PLANNING

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CHAPTER 2 BASIS OF PLANNING

2.1. INTRODUCTION

This chapter presents population data as well as wastewater flow and load information that was used throughout this study for current and future capacity analyses of the wastewater collection and treatment systems. In addition, this chapter describes the current regulatory requirements for the wastewater treatment plant (WWTP) and explores potential future MPDES permitting changes. The following are described in this chapter:

- Study Area
- Wastewater Planning Area
- Planning Periods
- Historical and Projected Wastewater Service Populations
- Wastewater Flows and Loads
- Regulatory Requirements and Potential Future Regulatory Changes

The wastewater collection system and treatment plant are complex systems and determination of their current condition and capacity required analysis of an extensive set of reliable data. In addition, planning data for the city and the wastewater service area was required to develop wastewater flow and load projections for the future. This data was made available by the City of Missoula and included Geographical Information System (GIS) data for current and future service areas and collection system attributes, tabulated wastewater flow and load data, lift station and wastewater treatment plant (WWTP) operational data, and other information needed to fully characterize the wastewater systems. General data sources are described further in applicable sections in this chapter and information regarding data acquisition for the collection system is provided in Chapter 3.

2.2. STUDY AREA AND PLANNING PERIODS

The study areas, including the 2017 Active Wastewater Account Area and the 2037 Wastewater Planning Area are shown in Figure 2-1. The figure also shows the existing Missoula city limits, although water and wastewater services are not restricted to areas within city limits. Planning areas and conditions for three planning periods were determined and used for the collection system and WWTP analyses as follows:

- Existing Conditions (2017): The 2017 Active Wastewater Account Area was determined through wastewater billing account information and comprises the total area connected to and served by the City of Missoula in 2017. This area is smaller than the Wastewater Service Area adopted by City of Missoula Resolution No. 8348 in April 2019. The 2017 account area and associated wastewater flows and loads were used as the basis for evaluation of existing conditions.

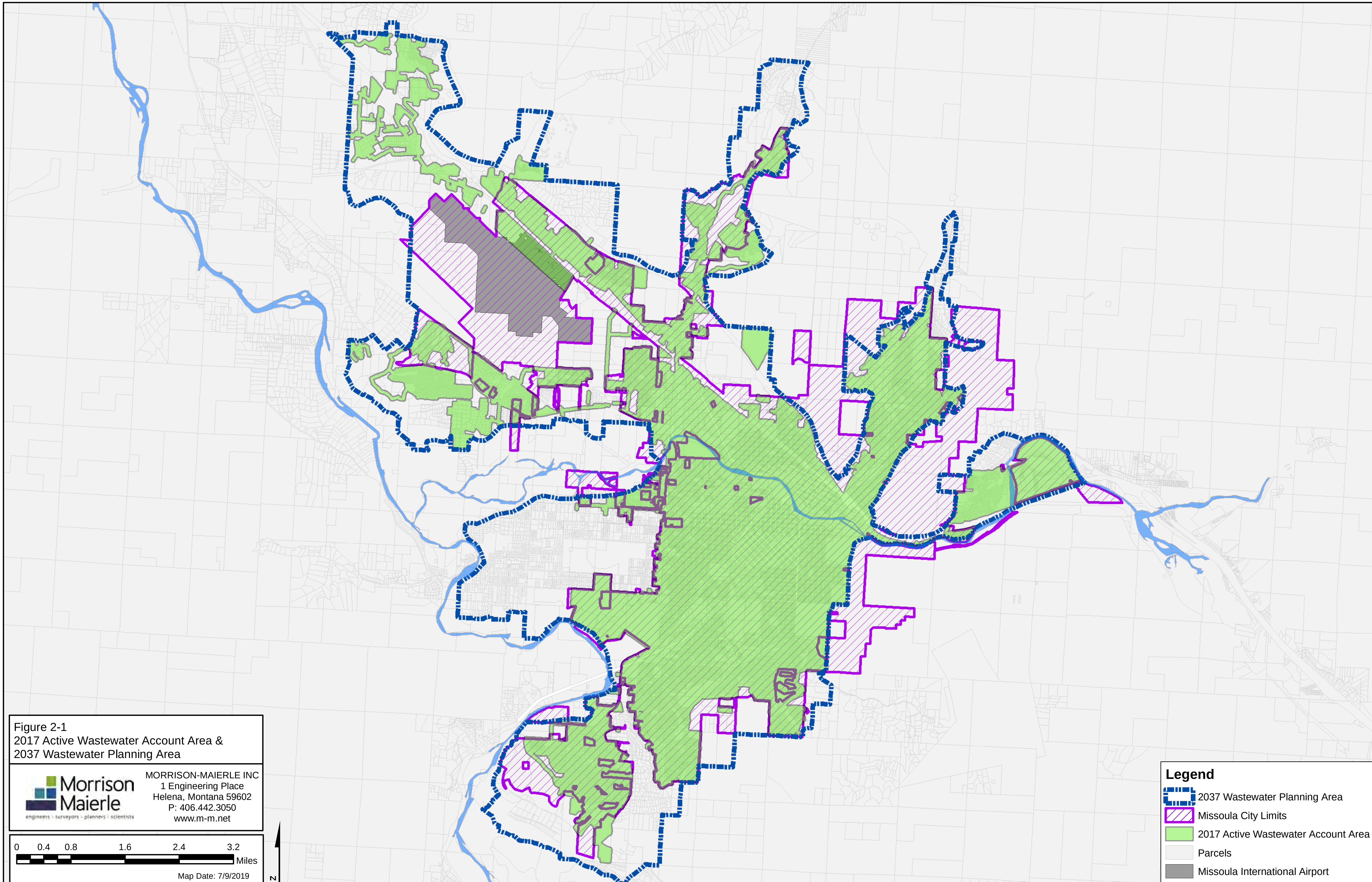

Figure 2-1
2017 Active Wastewater Account Area &
2037 Wastewater Planning Area

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MORRISON-MAIERLE INC
1 Engineering Place
Helena, Montana 59602
P: 406.442.3050
www.m-m.net

0 0.4 0.8 1.6 2.4 3.2 Miles

Map Date: 7/9/2019

N

Legend

2037 Wastewater Planning Area
Missoula City Limits
2017 Active Wastewater Account Area
Parcels
Missoula International Airport

- Near-Term Conditions: A near-term scenario developed for collection system modeling only includes the 2017 Active Wastewater Account Area area plus six developments that are currently in the planning stages and their estimated wastewater flows and loads. These developments are Riverfront Triangle, ROAM Student Living, Millsite, Mercantile/Residence Inn, Linda Vista Estates, and Hillview Way.
- Future Conditions (2037): The extent of the 2037 Planning Area is similar to the wastewater service area adopted by City of Missoula Resolution No. 8348 and was developed in collaboration with City of Missoula staff. City-provided GIS data for this area was used in conjunction with transportation planning information and wastewater billing account information to fully develop the characteristics of the 2037 Planning Area.

2.3. POPULATION ESTIMATES

2.3.1. Existing Population

In March of 2017, the City of Missoula completed a Long Range Transportation Plan (LRTP) (City of Missoula, 2017). Traffic analysis zones (TAZs) identical to the 2000 census block geography were used as the basis of planning for the LRTP. The population data was updated by the City to 2015 values for the LRTP and forecasted to 2045 in five-year increments. In 2017, the City used this data to develop population and land use data for 2017 and 2037 for the water and wastewater facility planning efforts.

Using the TAZ data provided by the City in March 2018, the total 2017 population within the study area planning boundary was calculated to be 87,279. This population includes both the TAZ residential population and an added group quarters residential population (hospitals, nursing homes, etc.) of 3,311, which is assumed to remain fixed throughout the planning period. The TAZ shapefiles provided population projections for the future while the group quarters population was assumed to be constant throughout the planning period. This method of population projection was retained for the wastewater facility plan analysis. The majority of the total residential planning boundary population of 87,279 currently receives wastewater services from the City but a small percentage does not. Cross-referencing billing account data with TAZ data yielded a total residential population receiving wastewater service of 68,015, including group quarters which represents approximately 78 percent of the total study area population.

Residential versus Non-Residential Population

For planning and collection system modeling it is useful to distinguish between residential and non-residential flows. The wastewater service population of 68,015 described above represents the residential population of Missoula receiving wastewater services. Non-residential flows were developed based on TAZ populations for retail, service, basic, education, healthcare, and leisure employment. These populations represent 2017 Active Wastewater Account Area residents and non-residents (transient population) who work within the account area. There is an overlap between residential and non-residential population among those city residents who are also employed in the city; however, separate per capita flow rates for residential and employment flows account for the overlap and ensure that flow is not double counted as further explained below.

2.3.2. Projected 2037 Population

Residential

The planning horizon for the Wastewater Facility Plan is 20 years, from 2017 to the year 2037. In discussions with the City, it was determined that the entire 2037 TAZ population occurring within the study area boundary would be utilized for the wastewater analyses and planning efforts. This conservatively assumes that in 2037 the entire residential population will receive sewer service, while in 2017 only approximately 78% of the total residential population was connected to City sewer.

The TAZ information contains residential population projections from 2017 to 2037 as prepared by the City of Missoula. To determine the future population, the study area boundary is cross-referenced with the respective TAZ areas, yielding a projected wastewater service population of 115,616 including group quarters. This represents an annual growth rate of 1.7 percent of the residential population with sewer service in Missoula from 2017 to 2037.

It was noted that approximately 904 of 2,299 TAZ areas within the study area boundary were projected by the City to decrease in population by year 2037 due to a predicted trend of decreasing population per household into year 2037. This total population decrease in these TAZ areas amounts to 2,068.

Non-Residential

TAZ data projections for employment population were not available for the planning period from 2017 to 2037. Therefore, the non-residential population was assumed to experience growth at the same rate as the residential population and was calculated based on the same growth projection of 1.7 percent used for the residential population.

2.3.3. Wastewater Study Area Population Summary

Figure 2-2 summarizes the population numbers that were used for this Facility Plan.

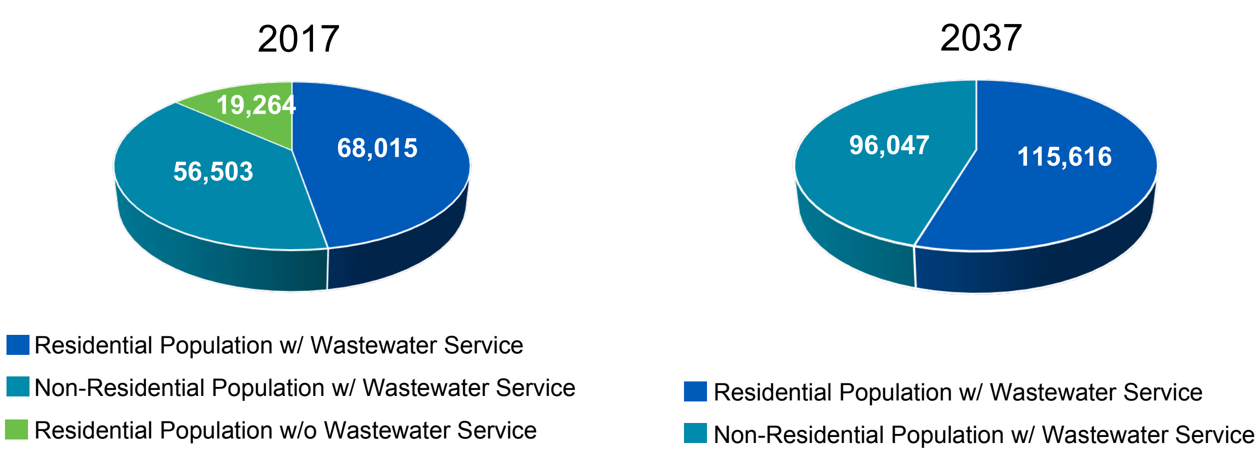

Figure 2-2: Wastewater Study Area Population

For the near-term scenario used for collection system modeling only, the residential and non-residential population of the six developments currently in the planning stages was added to the 2017 population to approximate collection system flows within and downstream of these developments. Populations and resultant flows are presented in Chapter 4.

2.4. WASTEWATER FLOWS AND ORGANIC LOADS

2.4.1. Existing Flows

Existing flows were evaluated based on daily influent flow meter data collected from the WWTP for the period of record (POR) for January 2013 through November 2017. An influent flow meter was installed with the headworks project in 2012 and therefore influent flow data is not available prior to 2013. Figure 2-3 shows the flow data from the influent flow meter for 2013-2017. As shown in the Figure, 2013, 2015, and 2016 were very similar while 2014 and 2017 have higher peak flows in late spring.

In 2018 the collection system and WWTP saw extreme flows caused by flood-related infiltration. This data was not included in the analysis as it was considered an outlier. Once flooding receded, the City did due diligence to ensure that the flows were not caused by damaged collection system piping or other repairable causes.

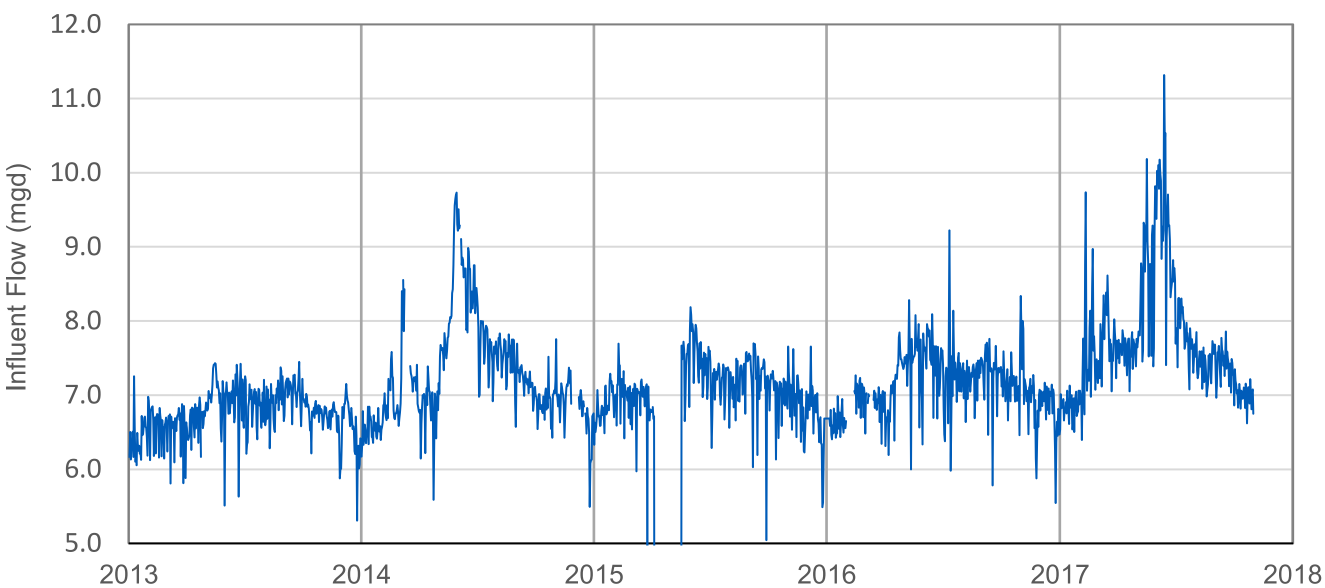

Figure 2-3: Missoula WWTP Flow Data for 2013-2017

Figure 2-4 presents the average annual, minimum and maximum flows for 2013-2017.

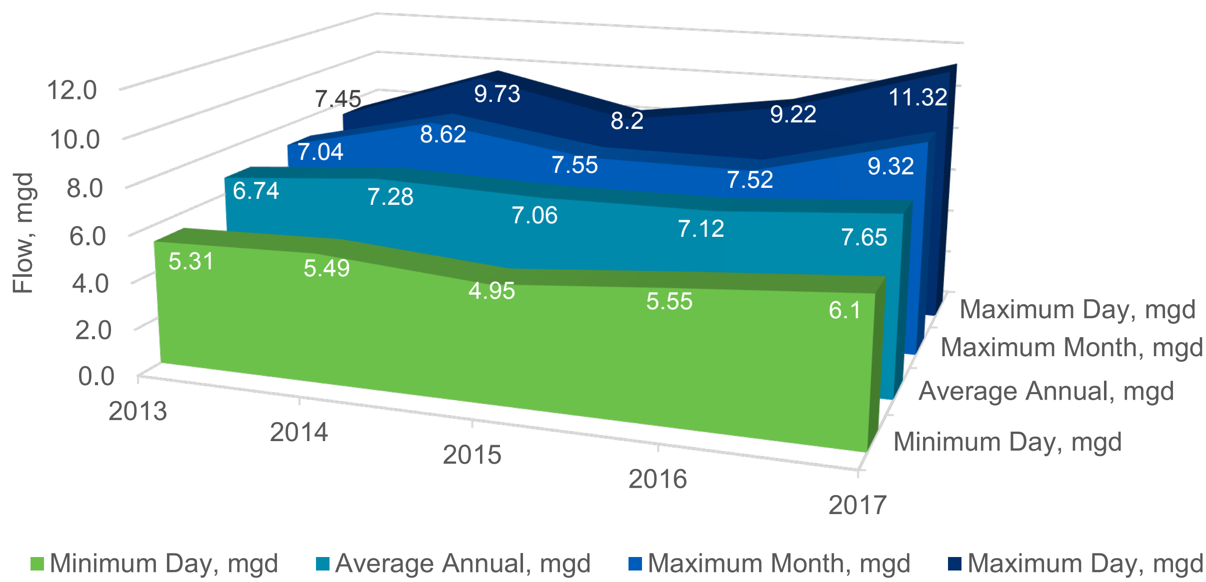

Figure 2-4: Missoula WWTP Flow Data

Peaking factors relative to average annual flow were computed across the range of minimum and maximum values for each year as shown in Table 2-1. The planning value for average annual flow was based on an average of data from the three most recent years, 2015, 2016, and 2017. This captures one wet and two average or dry years for population numbers similar to current population. Peaking factors for the minimum and maximum days and maximum month are based on the ratio of the absolute minimum and maximum day flows and maximum month flow in 2015-17 to the 3-year average annual flow. This resulted in planning values for maximum month and day equal to the maximum flows measured in 2017. These were the highest measured in the period of record and represent a “wet” or high infiltration year and therefore include infiltration as further discussed below.

Table 2-1: Missoula WWTP Historical and Planning Flow Factors and Flows

	2013	2014	2015	2016	2017	Planning Factor	Planning Flow
Minimum Day	0.79	0.75	0.70	0.78	0.80	0.68	4.95 mgd
Average Annual	1.0	1.0	1.0	1.0	1.0	1.0	7.27 mgd
Maximum Month	1.05	1.18	1.07	1.06	1.22	1.28	9.32 mgd
Maximum Day	1.11	1.34	1.16	1.30	1.48	1.56	11.32 mgd

Diurnal Flow Pattern

The diurnal flow curve shown in Figure 2-5 was developed using hourly WWTP influent flow data for January 14-20, 2015, January 13-19, 2016, and January 11-17, 2017 provided by the City of Missoula. The highest peak was observed on weekends resulting in a peak diurnal factor of 1.47 times the annual average flow. Low flows are very similar for all weekdays with the lowest flow factor being 0.48 times the annual average flow. Overall, the diurnal pattern for Missoula is typical of municipal wastewater flows and reflects the residential, commercial, and industrial activity within the community throughout the day. Figure 2-5 shows diurnal curves for week days, weekend days and an average day. While differences between week days and weekend days are clear, the average curve will be used for modeling plant processes, as well as collection system modeling.

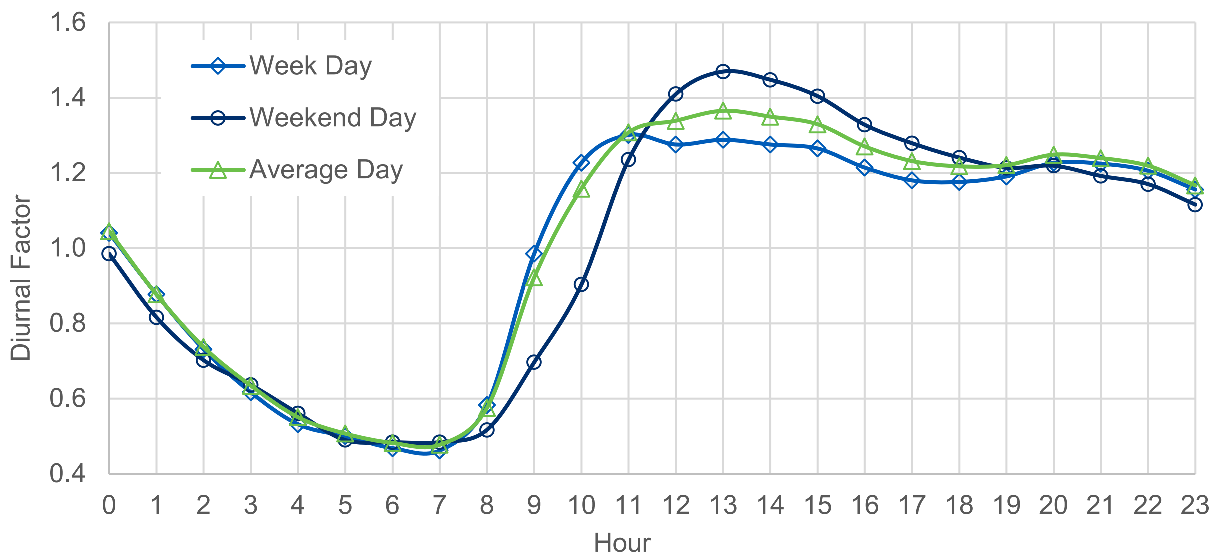

Figure 2-5: Diurnal Flow Pattern without the Influence of Seasonal Infiltration

The December/January period for all years with available data shows very similar flows regardless of high or low infiltration years. This is likely attributable to the absence of seasonal infiltration during these winter months. Data from a low-infiltration winter month was used to minimize the dampening effect of seasonal infiltration on the diurnal factors. While overall flow is higher when infiltration is high, the difference between the daily high and low flows caused by residential and non-residential water use patterns is less. However, as discussed further below, a baseline infiltration exists even during the winter months and is included in the diurnal factors. As this amount of infiltration is considered acceptable and always present, its influence on the diurnal factors was accepted.

Residential and Non-Residential Wastewater Flows

The sources of wastewater flow analyzed for this study include residential, non-residential, and inflow and infiltration (I&I). Residential and non-residential flows correspond to the population estimates presented in Section 3.4. Per capita flow rates for each population were developed based on metered water use for different account types. These separate residential and non-residential per capita flow rates were used for collection system modeling to approximate different flows for commercial and residential areas in Missoula. However, a conventional overall per capita flow rate was also calculated as it is used in projecting overall wastewater loads as further explained below. This overall per capita flow rate combines all domestic, commercial and industrial flows and assigns them to the residential population. The resulting per capita wastewater flow rates without I&I are shown in Table 2-2.

Table 2-2: Existing Residential and Non-Residential Average Wastewater Flows

	Population	Flow per Capita per day (gpcd)	Flow (mgd)
Residential	68,015	60	4.08
Non-Residential	56,503	27	1.53
Total Flow	--	--	5.61
Overall Per Capita	--	83	--
* per Capita or total flows presented do not include I&I flows			

Inflow and Infiltration

Inflow and Infiltration make up a significant portion of flow reaching the WWTP that is neither residential nor non-residential wastewater. Infiltration is defined as groundwater that enters the collection system through cracks and joints as a result of the groundwater table being above the collection piping. Inflow consists of water that enters the collection system directly from rainfall and/or improper connections (such as downspouts from roofs or sump pumps). In order to characterize the source(s) of I&I, several variables, including snow depth, rainfall, and groundwater table elevation were analyzed in relation to plant influent flows. Weather and flow data shows that a relationship exists between heavy snow years, high groundwater level, and extended high WWTP influent through late spring and early summer. A short-term correlation exists between heavy rain events and brief WWTP influent flow spikes. These correlations suggest that a large portion of I&I in the Missoula collection system consists of infiltration from groundwater and varies with groundwater depth. As weather affects groundwater depth, it will affect I&I into the wastewater collection system.

Two approaches were utilized to quantify infiltration within the collection system. The first approach uses plant influent data and compares high infiltration (wet) years to low infiltration (dry) years and high infiltration months to low infiltration months. The second approach uses calculated per capita flows as compared to plant influent flows.

Figure 2-6 helps illustrate the first approach. Average monthly flows are plotted for each of the five years of the period of record along with an overall average flow for all five years. Note that influent flows are lowest and do not vary much during December and January between high and low infiltration years. Average December flows vary from 6.6 to 6.8 mgd and average January flows vary from 6.4 to 6.8 mgd. The average of these low infiltration-month flows for all five years was used as a “dry-flow” baseline to determine infiltration during the late spring months. High infiltration months were selected based on plant flow data and groundwater elevation data as presented above, both of which show that May, June, and July are the months during which infiltration is peaking.

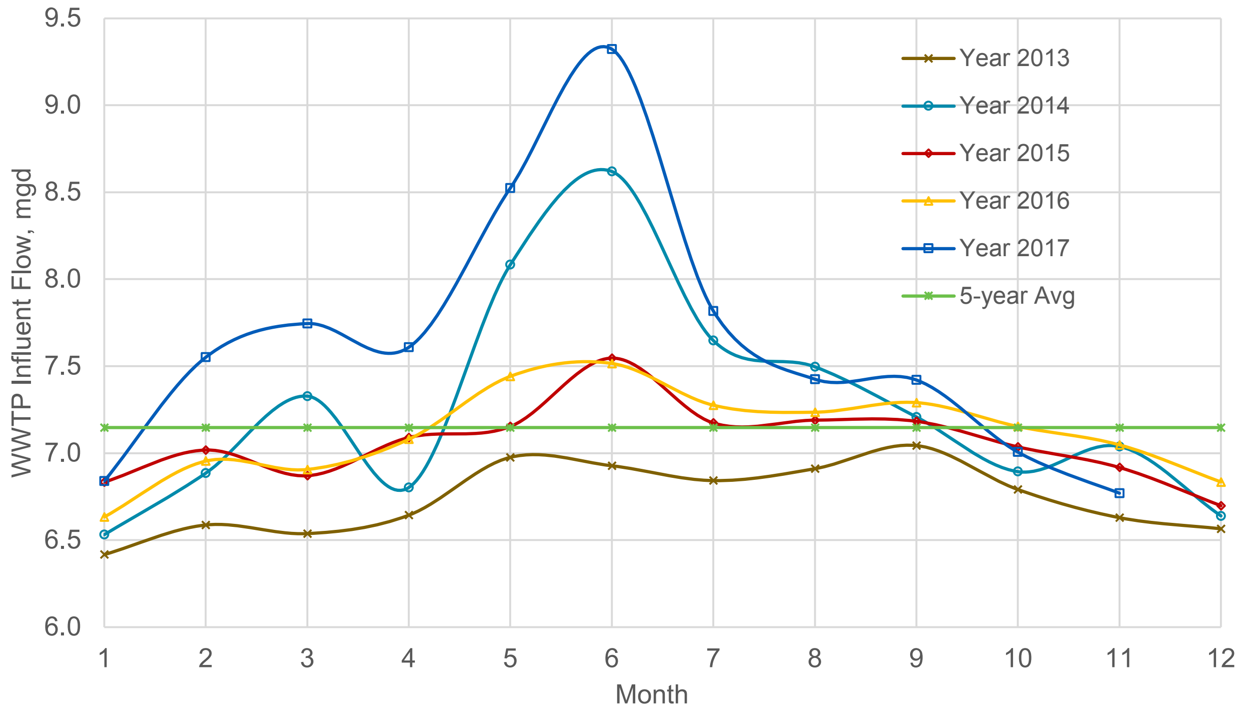

Figure 2-6: WWTP Monthly Influent Flow (2013-2018)

Data for various plant influent flow conditions were evaluated to quantify infiltration as shown in Table 2-3 below.

Table 2-3: Quantification of Infiltration Based on WWTP Influent Flow Data (2015-17)

	Flow	Estimated Infiltration
Avg. Dec. & Jan. (mgd, 2015-2017)	6.77	--
Overall Average (mgd, 2015-2017)	7.27	0.50
Avg. May, June, July (mgd, 2015-2017)	7.75	0.98
Max. Month (mgd, June 2017)	9.32	2.55
Max. Day (mgd, June 13, 2017)	11.32	4.55

The second approach uses per capita flow rates to determine I&I as the difference between the calculated wastewater flow and the measured plant influent. Basing non-infiltration flows on December and January plant influent data does not reveal if there is any infiltration occurring during these two months that should be taken into consideration. Low-lying collection system areas likely experience year-round infiltration. Comparing the calculated total non-infiltration flow of 5.61 mgd listed in Table 2-2 above to the average December/January flow of 6.77 mgd in Table 2-3 above, suggests that about 1.16 mgd of baseline infiltration is occurring year-round. This infiltration amount equates to about 17 gpd per capita. According to Montana Department of Environmental Quality (MDEQ) Circular DEQ-2 (Mt. Dept. of Env. Quality, 2016), a value of 100 gpcd is intended to include "normal infiltration." Adding the above calculated 17 gpcd to the overall per capita flow of 83 gpd results in 100 gpcd; therefore, this baseline infiltration is considered reasonable for a collection system of the size of Missoula's.

Comparing wet weather flows to dry weather flows also neglects increased residential/commercial flows that may be occurring at the same time. Table 2-4 summarizes infiltration quantities based on the per-capita flow analysis, which includes peaking residential and non-residential flows prior to estimating infiltration using the peaking factors presented in Table 2-1.

For collection system modeling purposes, it is useful to have an area-based infiltration quantification. An overall estimate of average area based infiltration, using the more conservative per-capita wastewater flow approach described above is included in Table 2-4. However, collection system modeling breaks down the system into smaller portions and due to the presence of pump stations, varying service types (residential, commercial, industrial), and the different characteristics and dynamics in small subsystems when compared to the system as a whole, localized flows were developed to more accurately represent the conditions in these smaller subsystems. More information is provided in Chapter 5.

Table 2-4: Quantification of Infiltration Based on Per Capita Flows

	Residential	Non-Residential	Infiltration	Total
2017 Average Flow (mgd)	4.08	1.53	1.66	7.27
2017 Maximum Month (mgd)	5.22	1.96	2.14	9.32
2017 Maximum Day (mgd)	6.36	2.39	2.57	11.32
2017 Active Wastewater Account Area (ac)	17,114			
Baseline Infiltration by Area[1] (gal/d-ac)	68			
Average Infiltration by Area[2] (gal/d-ac)	97			
Maximum Day Infiltration by Area[3] (gal/d-ac)	150			

1. Baseline infiltration is the infiltration experienced in December/January during otherwise dry months.
2. Average infiltration is infiltration experienced in an average month, which includes wet and dry months.
3. Maximum day infiltration is infiltration experienced during a maximum day flow event, typically in June.

2.4.2. Existing Organic Loads

Measured Influent Constituents

Existing organic loads to the WWTP were evaluated based on influent data collected at the WWTP and analyzed for carbonaceous biochemical oxygen demand ($cBOD_5$), total suspended solids (TSS), ammonia (NH_3), total Kjeldahl nitrogen (TKN), and total phosphorous (TP). TKN is often used in place of total nitrogen (TN) because the analysis is simpler. TKN plus nitrate/nitrite constitute TN. Raw wastewater typically contains only very small amounts of nitrate/nitrite and TKN is generally accepted as a valid method of quantifying influent nitrogen.

Influent loads were determined by multiplying the influent concentration by the influent flow for the day on which the influent concentration was recorded. Figure 2-7 shows $cBOD_5$, TSS, TKN, NH_3, and TP concentrations from 2013 to 2017. Individual data points represent individual sampling events, while the smoothed lines show running averages for each constituent. The running average period was chosen purely for illustrative purposes to show overall trends and varies by constituent.

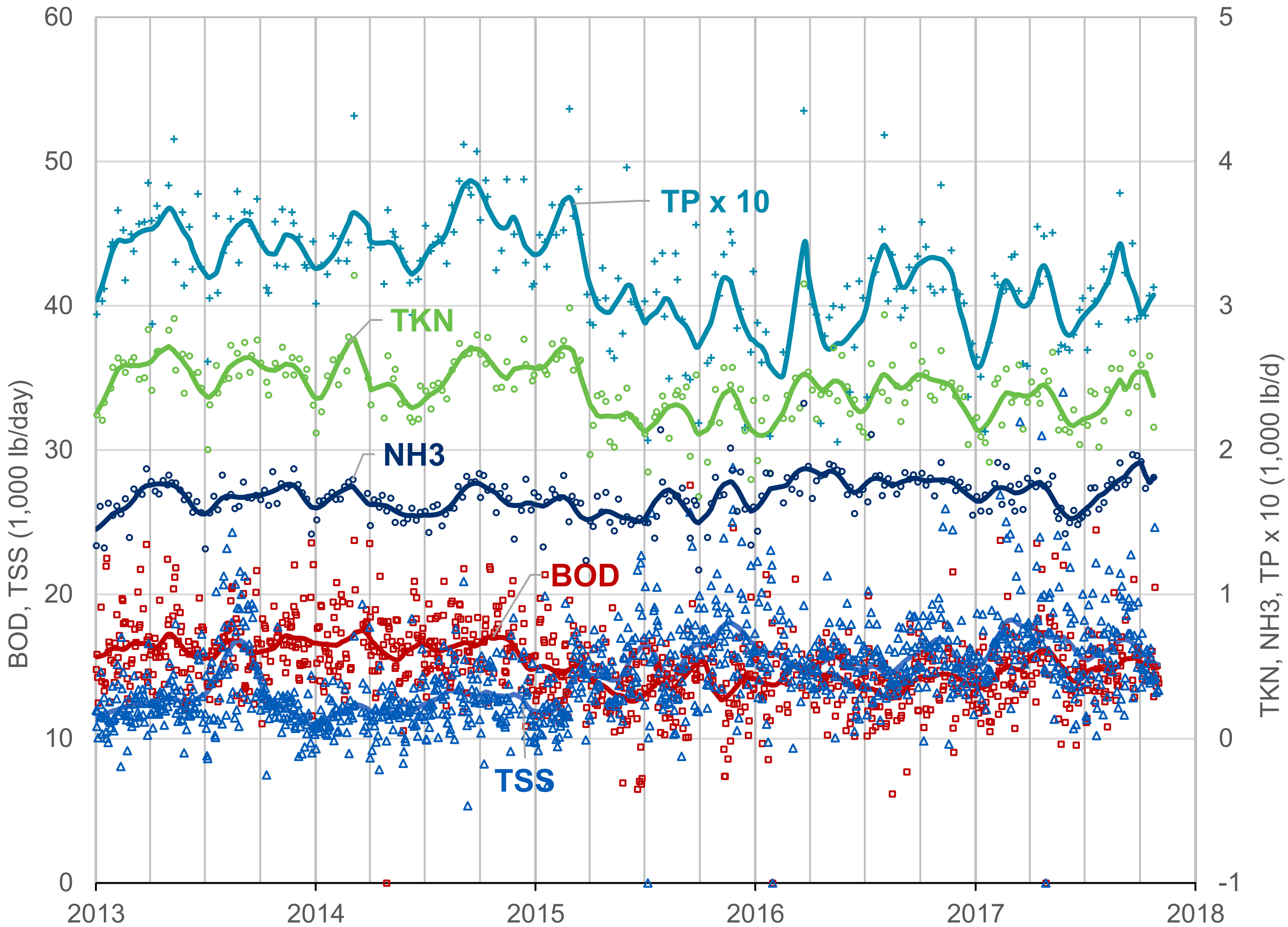

Figure 2-7: WWTP Influent Loading 2013-2017

The figure shows that with the exception of TSS, loads have been stable or have even decreased slightly over the past five years[1]. For planning purposes, a 3-year record was used to be consistent with the period used to analyze WWTP flow.

No local data exists to characterize residential versus non-residential wastewater loading. As loading is not needed for collection system modeling and treatment planning does not differentiate between different wastewater sources, per capita loading was calculated based on the total residential population of 68,015. Table 2-5 summarizes the total loadings, per capita loadings, and concentrations for a 3-year period of record for average, peak month, and peak day.

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[1] In March 2015, plant staff changed influent sample analysis from determining TKN to persulfate digestion (total nitrogen – TN), which appears to lead to somewhat lower concentrations and calculated loads; however, no obvious cause was identified for the apparent drop in TN concentrations after the change in analytical procedures. Additional investigations with plant staff determined that the change was made because the persulfate digestion is simpler, which generally translates to smaller laboratory error and more accurate and precise analysis results. The method is accepted by MDEQ and the U.S. Environmental Protection Agency (EPA).

Table 2-5: Missoula WWTP Existing Influent Loading (3-year Period of Record)

	$cBOD_5$	TSS	NH_3	TKN	TP
Average Concentration (mg/l)	235	259	28	38	5.0
Average Load (lb/d)	14,184	15,657	1,699	2,325	306
Average Load per Capita (ppcd)	0.21	0.23	0.025	0.034	0.0045 [1]
DEQ-2 Guidelines (ppcd)	*0.20*	*0.22*	--	*0.033*	*0.009* [1]
Maximum Month (lb/d)	16,607	18,571	1,892	2,759	377
Maximum Month Peaking Factor	1.17	1.19	1.11	1.19	1.23
Literature Max. Month Peaking Factor [2]	*1.25*	*1.45*	*1.3*	*1.3*	*1.3*
Maximum Day (lb/d)	25,578	34,020	2,326	3,153	437
Maximum Day Peaking Factor	1.94	2.17	1.37	1.36	1.42
Literature Max. Day Peaking Factor	*2.4*	*2.7*	*1.5*	*2.05*	*1.65*

1. Phosphorus ban in Missoula reduces influent TP load.
2. Metcalf & Eddy, 4th ed., McGraw-Hill, New York, 2003, Figure 3-8.

The per capita loads are slightly higher than those recommended in Circular DEQ-2, 11.253, except for TP, which is lower due to the long-standing detergent phosphorous ban in the Missoula region. It should be noted that these per capita values include all infiltration mixed in with the wastewater as they are purely based on plant influent data. The calculated peaking factors vary widely – more widely than reported in the literature (Metcalf & Eddy, 2003). The literature values are significantly higher than the Missoula data. A balance must be struck between selecting peaking factors that are sufficiently conservative to include most possible scenarios, yet not overly conservative, leading to planning for more expensive upgrades than necessary. Therefore, peaking factors between the data-supported and literature values were chosen, with more weight given to the Missoula specific, data-based factors. The adjusted peaking factors are included in Table 2-6 below for projecting future wastewater flows.

2.4.3. Projected 2037 Flows and Organic Loads

2037 Flows

Future flows for 2037 are based on the population projections for the residential and non-residential populations presented in Figure 2-2. In addition, 1.66 mgd of infiltration was added to the projected average residential and non-residential flow, as described in Table 2-4 It was also assumed that the existing seasonal infiltration that occurs primarily in May, June, and July would essentially remain the same, although it may be reduced as the City implements I&I reducing measures. Infiltration and inflow is expected to be low in future service areas as newer piping and manholes are generally tighter and provide less opportunity for infiltration. Therefore, a fixed I&I flow was added to the projected residential and non-residential wastewater flow for 2037. Table 2-6 presents overall projected wastewater flows.

Table 2-6: Projected 2037 Wastewater Flows

	Peaking Factor	Residential	Non-Residential	Infiltration	Total
2037 Population	--	115,616	96,047	--	--
Per Capita (gpcd)	--	60	27	--	--
2037 Average Flow (mgd)	--	6.94	2.59	1.66	11.2
2037 Maximum Month Flow[1] (mgd)	1.28	8.88	3.32	2.14	14.3
2037 Maximum Day Flow[1] (mgd)	1.56	10.83	4.04	2.57	17.4
2037 Peak Hour Flow[2] (mgd)	2.14	14.85	5.54	6.21	26.6
Connected Service Area (ac)	2017 Active Account Area		Additional 2037 Planning Area		
	17,114		16,760		
Baseline Infiltration by Area (gal/d-ac)	68		68[3, 4]		
Average Infiltration by Area (gal/d-ac)	97		-		
Max Day Infiltration by Area (gal/d-ac)	150		-		

1. See peaking factors presented in Table 2-1.
2. Peaking factor calculated based on population per Circular DEQ-2.
3. A fixed 68 gal/d-ac planning value will be applied to the additional 2037 area for collection system modeling.
4. Accepting a fixed infiltration by area for the 2037 equal to the existing 2017 area assumes that newly installed collection system piping and manholes are relatively tighter than older ones and that the City has addressed some existing infiltration issues during the planning period.

2037 Organic Loads

Projected influent loading is based on the loading factors calculated for the 3-year average with slight adjustments as discussed above. The average influent concentration was calculated based on the projected flows and loads.

Table 2-7: 2037 Projected 2037 Loads for the Missoula WWTP

	$cBOD_5$	TSS	NH_3	TKN	TP
Population	115,616				
Average Load per Capita (ppcd)	0.21	0.23	0.025	0.034	0.0045
2037 Average Load (lb/d)	24,111	26,614	2,888	3,952	521
2037 Average Concentration (mg/l)	273	301	33	45	5.9
Maximum Month Peaking Factor	1.2	1.3	1.25	1.25	1.25
Maximum Month (lb/d)	28,933	34,598	3,610	4,940	651
Maximum Day Peaking Factor	2.0	2.3	1.4	1.6	1.5
Maximum Day (lb/d)	48,221	61,212	4,044	6,323	781

2.5. EFFLUENT LIMITATIONS AND REGULATORY REQUIREMENTS

This section summarizes the current effluent limitations and regulatory requirements for the Missoula WWTP and discusses potential future changes that may affect the City's approach to wastewater and biosolids treatment and disposal. In addition to the projected area growth, the Voluntary Nutrient Reduction Program (VNRP) requirements for the Clark Fork River, river sampling results, and MDEQ plans for regulating WWTP discharges were considered.

2.5.1. MPDES Permitting and Regulatory Requirements

Current MPDES Permit

The Missoula WWTP operates under the Montana Pollutant Discharge Elimination System (MPDES) permit no. MT0022594. This permit regulates effluent concentrations and loads for $cBOD_5$, TSS, *E. coli*, TN, TP, ammonia, oil & grease, copper, lead, and iron. Table 2-8 lists the current effluent limits as enforced by MDEQ. The 2015 permit and accompanying fact sheet are included in Appendix 2-1. In addition to the effluent limits, the following requirements are included in the permit:

- Effluent pH shall be between 6.0 and 9.0
- 85% removal for $cBOD_5$
- 85% removal for TSS
- No discharge of floating solids, no visible foam or visible oil sheen in the receiving stream

Details regarding various permitted parameters are discussed in more detail in the following sections.

Monitoring

Effluent sampling is required for all regulated wastewater constituents plus cyanide and whole effluent toxicity (WET). Influent sampling is required for $cBOD_5$ and TSS; however, WWTP personnel routinely sample for influent ammonia, nutrients, and metals as well. Upstream river sampling is required for cyanide only because insufficient data was available during the past permit renewal to determine reasonable potential to exceed instream standards. Plant staff also analyse upstream river samples for copper and hardness to aid in the determination of future copper limits.

Table 2-8: 2015 MPDES Permit Limits

Parameter	Units	Avg. Monthly Limit	Avg. Weekly Limit	Max. Daily Limit
Carbonaceous Biochemical Oxygen Demand ($cBOD_5$)	mg/L	19	30	--
	lb/d	1,874	2,999	--
Total Suspended Solids (TSS)	mg/L	23	34	--
	lb/d	2,249	3,374	--
E. coli [1, 2]	cfu/100 mL	126	252	--
E. coli [2, 3]	cfu/100 mL	630	1,260	--
Total Residual Chlorine [4]	mg/L	0.011	--	0.019
Total Nitrogen (TN) [5, 6]	lb/d	910	--	--
Total Phosphorous (TN) as P [6]	lb/d	101	--	--
Total Ammonia as N	mg/L	3.4	6.9	--
Oil & Grease	mg/L	--	--	10
Total Recoverable Copper [7]	µg/L	11	--	13.6
Total Recoverable Lead [7]	µg/L	2.6	--	2.9
Total Recoverable Iron [7]	µg/L	950	--	1,640

1. This limit applies during the period from April 1 through October 31, annually.
2. Report Geometric Mean if more than one sample is collected during reporting period.
3. This limit applies during the period from November 1 through March 31, annually.
4. As long as RAS chlorination equipment exists, this limit will stay in the permit.
5. Calculated as the sum of Total Kjeldahl Nitrogen (TKN) and nitrate/nitrite as N concentrations.
6. This limit applies during the period from June 1 through September 30, annually.
7. This limit is effective April 1, 2020.

Compliance

During the period of record for this Facility Plan (January 2013 through November 2017), the plant effluent violated MPDES limits on four occasions. Table 2-9 lists the violations. None of them were systemic and all were short-term. Ammonia had the longest out-of-compliance period of about one week. Violation letters were issued for these limit exceedances. Other violations exist for reporting irregularities but were not caused by treatment plant performance.

Table 2-9: MPDES Limit Violations during the POR

Parameter	Date	Limit Type	Limit	Effluent
Toxicity (WET)	Dec. 2013	Daily Maximum	Pass/Fail	Fail
pH, SU	Jan. 2015	Daily Minimum	6.0	5.98
E. coli, mpn/100 mL	Aug. 2015	Weekly Average	252	351
Ammonia, mg/L	Nov. 2016	Weekly Average	6.9	6.99
		Monthly Average	3.4	3.6

Special Conditions

The compliance section of the permit addresses sewage sludge, toxicity reduction evaluation, hybrid poplar land application, effluent dissolved oxygen (DO), and effluent metals limits compliance. The first two items are addressed with standard language about compliance with federal regulations for use and disposal of sewage sludge and the process following a failed WET test.

Land application at the hybrid poplar plantation is addressed in general terms as MDEQ does not actually permit this discharge as an outfall. Requirements include zero discharge from the land application site, complete treatment before land application, compliance of operation and maintenance with Circular DEQ-2 (Mt. Dept. of Env. Quality, 2016), and development, implementation, and annual updates to a best management practices plan.

Effluent dissolved oxygen was addressed by requiring the City to submit a dissolved oxygen study by May 30, 2016. The report was submitted to MDEQ in April 2016. It includes data and results from a Streeter-Phelps analysis performed for the summer months at design flow without land application for the most restrictive parameters for stream channel depth and velocity. Results show that a small DO sag occurs approximately 1.9 miles downstream of the outfall with a resultant DO concentration above the 1-day minimum standard but below the 7-day minimum standard. The report also notes that the Clark Fork River upstream of the outfall does not consistently meet the DO standard with 83% of available sample results falling below the 7-day minimum.

Compliance with final effluent limits for copper, lead, and iron is required by April 1, 2020. Until then, the City must submit annual reports of progress toward compliance with these limits. Reports were submitted as required for the past three years and are included in Appendix 2-2. No exceedances were measured for iron and lead; however copper exceeded limits in three of 19 samples in 2015 and three of 24 samples in 2016. No exceedances were measured in 2017 as shown in Figure 2-8.

The City implemented a number of checks and procedures to ensure the measured concentrations are correct and to allow for a complete picture of the nature of the metals in the influent and effluent. These procedures included comparative analysis in two laboratories, implementation of metals analysis in the WWTP lab, influent and effluent analysis for total and dissolved copper, and river analysis for hardness. The annual report for 2017 also states that the WWTP will work toward analyzing river samples for copper to expand the data base for instream copper concentrations. To date, no efforts at evaluating treatment processes with respect to metals removal has been made.

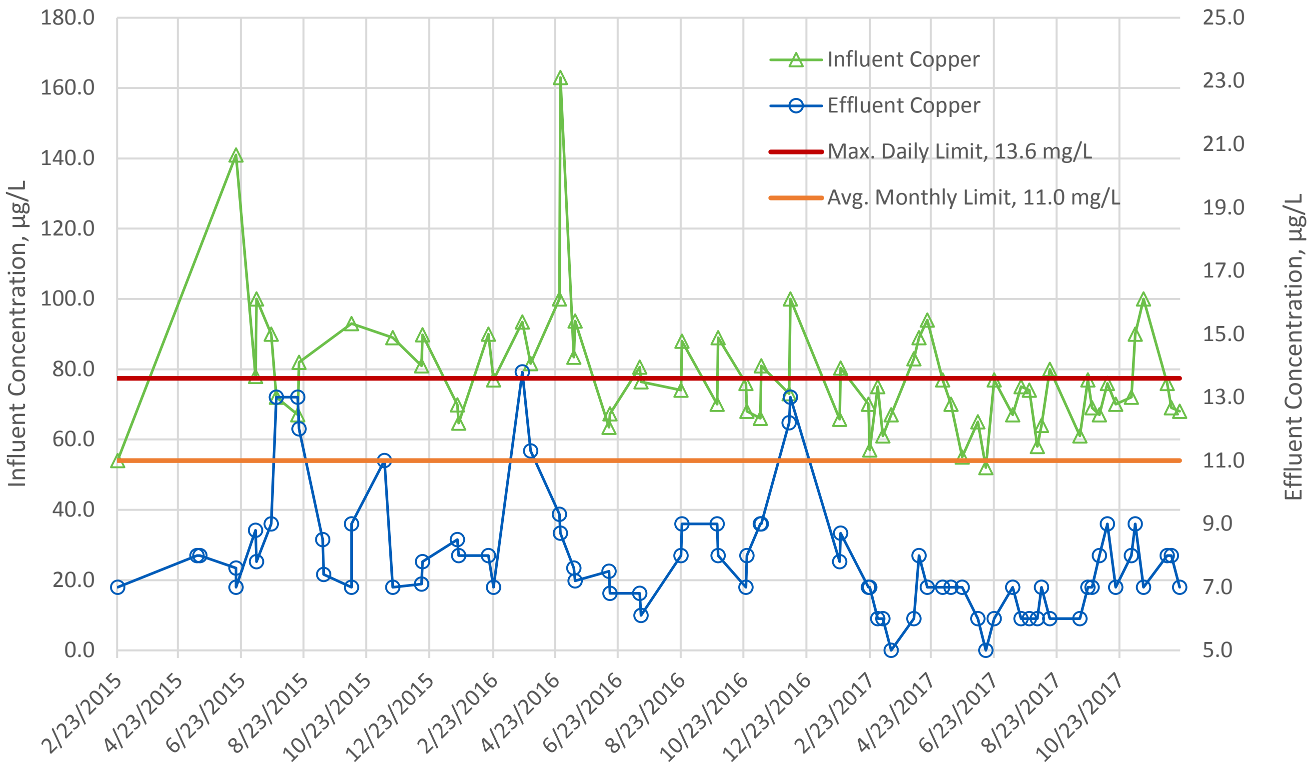

Figure 2-8: 2015-2017 Effluent Copper Concentrations and 2020 MPDES Permit Limits

2.5.2. MPDES Permitting History

cBOD$_5$, TSS, and pH

TSS, cBOD$_5$[2], and pH are technology-based effluent limits and generally follow National Secondary Treatment requirements for effluent concentrations and percent removal. The Administrative Rules of Montana (ARM) require that a mass-based limit is calculated, which is based on the design flow of the plant. This approach leads to an increase in the mass-based limit every time the design capacity of the plant is upgraded, as in 2004 when design capacity was increased from 8.99 mgd to 12.0 mgd. However, the ARM non-degradation rules do not allow an increase in the pollutant load beyond the amount authorized when the rule went into effect in the late 1990s. Therefore, the mass-based (load) limit for cBOD$_5$ and TSS remains the same after capacity upgrades. The concentration limit is back-calculated

[2] The Missoula WWTP monitors carbonaceous biochemical oxygen demand rather than total biochemical oxygen demand. Biochemical oxygen demand is a surrogate for the quantification of the presence and removal of organics, generally not assumed to include nitrogen in all its species. Biological nutrient removal (BNR) facilities have a healthy population of bacteria for nitrification that will add disproportionately to the overall oxygen demand when converting any remaining ammonia to nitrate/nitrite during laboratory analysis. This can skew the end result and show higher than actual BOD$_5$ concentration. Therefore, cBOD$_5$ is used because it measures only carbonaceous oxygen demand as exerted by those bacteria that consume carbonaceous (non-nitrogen) organic waste. The concentration limits for cBOD$_5$ are slightly lower than those for BOD$_5$ as no allowance for nitrogenous oxygen consumption is made. The Missoula MPDES permits have included cBOD$_5$ and associated limits at least since 1998.

using the set non-degradation mass limits and the current WWTP design flow to arrive at the concentrations included in the current permit. If plant design flows are expanded in the future, the concentration limits will be reduced further to maintain the same mass-based load to the environment.

E. coli and Oil & Grease

The limits for *E. coli* and oil and grease have not changed for the past couple of permit cycles and are not expected to change in the foreseeable future. Effluent limits apply at the end of pipe to protect recreational use of the river as well as aquatic life.

Total Residual Chlorine

Since 2004, the Missoula WWTP has not used chlorination for effluent disinfection. However, the facility maintains equipment for the chlorination of return activated sludge (RAS) in the event of process upsets. As long as the facility maintains any process chlorination equipment this limit will remain in the permit.

Metals and Cyanide

The Clark Fork River is impaired for metals, largely due to past mining activities upstream and remaining metal-laden sediments. While efforts continue to remove these contaminated sediments, the instream concentration for a number of metals, including copper, arsenic, lead, and iron are still very high. The Silver Bow Creek and Clark Fork River Metals Total Maximum Daily Loads (TMDLs) (Mt. Dept. of Env. Quality, 2014) were approved by the U.S. Environmental Protection Agency (EPA) in May 2014. The TMDL documents include wasteload allocations for the WWTP for copper, lead, and iron which translated into effluent limits included in the 2015 permit. In addition, the 2006 permit required bi-annual sampling for a long list of metals. The sampling results did not show reasonable potential for exceeding any other metals concentrations in the WWTP effluent; however, insufficient data was available for cyanide and additional monitoring is required to determine if a limit will be necessary. Further implications of the metals TMDLs are discussed below.

Ammonia

The 2006 permit introduced a monitoring requirement but did not apply a limit, recognizing that the BNR upgrade of the plant had only been in operation for about one year at that time. Data gathered during the permit cycle was used in the 2015 permit to introduce an ammonia limit shown above. This limit is in keeping with facilities across the state and takes into consideration stream temperature, pH, dilution ratio, and effluent characteristics. Calculation of the limits for ammonia and metals used the dilution ratios and mixing zone lengths were determined by a mixing zone study completed and submitted to MDEQ in 2008.

Nutrients – TN and TP

Since 1998, nutrient limits for the Missoula WWTP have been governed by the Voluntary Nutrient Reduction Program (VNRP) and the TMDLs as approved by EPA based on the work of the VNRP. The wasteload allocations (WLAs) for the Missoula WWTP based on this TMDL are 888.8 lb/d of total nitrogen and 88 lb/d of total phosphorous, which were incorporated for the first time into the 2006 permit. These

WLAs are more stringent than the mass-based load allocations used in permits prior to the 2006 permit. The Missoula WWTP was able to comply with the TMDL-based WLAs.

In addition, a nutrient trading program developed as part of the VNRP made it attractive for the Missoula WWTP to connect several neighborhoods with septic systems, as well as industrial dischargers, to their collection system and receive credit toward their assigned WLA for nutrients. This allowed MDEQ to increase the wasteload allocation for the WWTP in the 2015 permit renewal as listed in Table 2-8 above. Further background and detail about developments influencing nutrient effluent limits in Montana are discussed later in this chapter.

2.5.3. Voluntary Nutrient Reduction Program (VNRP)

In February 1994, a committee was formed to establish instream nutrient targets for the Clark Fork River and to develop a basin wide nutrient source reduction program to meet those targets. The committee included representatives from the cities of Butte, Deer Lodge and Missoula, Stone Container Corporation, the University of Montana, the Clark Fork-Pend Oreille Coalition, the Missoula City-County Health Department, and the Montana Department of Environmental Quality.

The VNRP called for site-specific measures to be taken by each of the key point source dischargers and significant reductions in key non-point sources to meet specific instream algal density and nutrient targets. The instream targets are 20 μg/L TP upstream of Missoula, 39 μg/L TP downstream of Missoula, and 300 μg/L TN throughout the River. These water quality standards were considered protective of the intended uses of the Clark Fork River in the vicinity of Missoula and it was hoped that achievement of these targets would reduce the algae growth in the river.

The VNRP was submitted to the EPA on September 21, 1998 requesting approval of the plan and calculation of phosphorous and nitrogen TMDLs for the Clark Fork River. In October 1998, the EPA approved the VNRP and developed TMDLs and WLAs based on the information provided in the report. The WLAs were then used by MDEQ during the 2006 MPDES permit renewal for the Missoula WWTP. The VNRP proposed nutrient water quality standards for the Clark Fork River upstream and downstream of the Reserve St. Bridge, suggested effluent limits for the Missoula WWTP, and EPA wasteload allocations are listed in Table 2-10 below.

Table 2-10: VNRP Proposed Nutrient Standards and Limits

VNRP Water Quality Standards	Clark Fork above Reserve St.	Clark Fork below Reserve St.
TN	20 µg/L	39 µg/L
TP	300 µg/L	300 µg/L
VNRP Effluent Limits[1]	**Concentration**	
TN	10.0 mg/L	
TP	1.0 mg/L	
Original EPA WLAs for the Missoula WWTP	**Mass**	
TN	891 lb/d (404 kg/d)	
TP	88 lb/d (40 kg/d)	
1. Limits were also proposed for BOD_5, TSS, and NH_3 but were not implemented as such.		

The VNRP acknowledged that a significant portion of nutrients that reach the Clark Fork are generated by non-point sources, including septic systems in portions of Missoula. It was recognized that a successful nutrient reduction strategy must eventually address these non-point sources as well. The City of Missoula, the City-county Health Department Board of Health, the Missoula county Commissioners, and MDEQ committed to developing strategies that would address non-point sources in addition to named point sources in Missoula. As a result, MDEQ in conjunction with the stakeholders developed a nutrient trading program, which is governed by Circular DEQ-13 (Mt. Dept. of Env. Quality, December 2012). The program allowed Missoula to receive credit for connecting septic system users and one industrial discharger to the treatment plant. This credit was introduced in the 2015 permit.

As part of the VNRP, the Clark Fork River was monitored to quantify the success of the nutrient removal strategies. Report updates were issued in 2002, 2005, and a final report in 2008. The 2008 report presents data that indicates that total phosphorous and total nitrogen levels had been reduced but algae growth was still stronger than desired. Not all of the VNRP goals for permitted dischargers had been met at that time, including completion of construction of the nutrient removal WWTP in Butte-Silver Bow and complete elimination of discharge during the summer by the Deer Lodge WWTP. However, Missoula had implemented or was in the process of implementing nutrient reduction measures originally suggested in the VNRP, which resulted in a significant reduction of the WWTP nutrient load discharged to the Clark Fork River. The VNRP itself is no longer active; however, the TMDLs developed under the program are still in force.

2.5.4. TMDLs for the Clark Fork River

General

The Silver Bow Creek and Clark Fork River Metals TMDLs report states that “a TMDL is a tool for implementing water quality standards and is based on the relationship between pollutant sources and water quality conditions. More specifically, a TMDL is a calculation of the maximum amount of a pollutant that a waterbody can receive from all sources and still meet water quality standards" (Mt. Dept. of Env. Quality, 2014).

TMDLs are developed for all pollutants included on the Montana 303(d) list, which lists impaired water bodies, the pollutants causing the impairment(s), and likely pollutant sources. This list is updated every two years. Data collected between publications may also be used during TMDL development ahead of its publication in the next 303(d) list to include the most comprehensive data set.

Once a TMDL is developed for a river segment, this total load is then divided into wasteload allocations for each contributor along this river segment. Contributors include non-point sources such as superfund sites, mining, agriculture, streambank erosion, etc., and point-sources, such as discharges from discrete points, such as pipes, ditches, wells, containers, or concentrated animal feeding operations. Many of the point sources are permitted under the MPDES program, which must include effluent limits for those pollutants with TMDLs. These limits are typically expressed as an effluent concentration but are calculated from the allowable load in pounds.

TMDLs are required to include a margin of safety to account for uncertainty or lack of data about the relationship between the pollutant loads and the quality of the receiving waterbody. For most TMDLs in Montana, the margin of safety is incorporated implicitly by using conservative assumptions and approach to developing final TMDLs.

Typically, a water body is listed as impaired for a pollutant if the concentration or load of this pollutant exceeds the assimilative capacity of the water body. Therefore, when MDEQ calculates a WLA for a permitted discharge, the concentration limit is set to ensure that the permitted point source does not contribute to the impairment. This limit is applied to "end-of-pipe" and no mixing zone is granted. However, as a pollutant in the water body shows improvement and assimilative capacity is created, it may be possible to increase WLAs and/or grant mixing zones at the end of which the limit concentration has to be achieved.

Nutrients

Statewide efforts around the development of nutrient TMDLs started with the VNRP and resulting TMDLs for Silver Bow Creek and the Clark Fork River to Missoula developed and approved by EPA in 1998. Since then, MDEQ has developed TMDLs for nutrients and other constituents for many surface waters in Montana based on the more recently established numeric nutrient standards further discussed below. As part of this effort, Silver Bow Creek and the Upper Clark Fork River above Deer Lodge were reassessed and new TMDLs developed and approved by EPA. Wasteload allocations calculated from the new TMDLs supersede and are generally lower than those in place previously under the VNRP. The Middle and Lower Clark Fork River are still subject to the TMDLs and associated WLAs approved in 1998. However, the TMDL workgroup at MDEQ has tentative plans to reevaluate the TMDLs on the Middle and Lower Clark Fork to be consistent in the methodology used to arrive at TMDLs and calculate WLAs (Yashan, 2018). While it is fairly certain that this reevaluation will occur eventually, a schedule for this effort has not yet been set. MDEQ's webpage for the TMDL program includes a list of priority areas for which TMDLs will be developed by 2022, which does not include the Clark Fork River. The list is updated as projects are completed and new ones are added. It is recommended that the City monitor this page over the next five to ten years for inclusion of the Middle and/or Lower Clark Fork River as well as staying in contact with MDEQ to have early warning of the potential for changing effluent nutrient limits. According to MDEQ (Yashan, 2018), it is possible that the development of the new TMDLs would result in significantly lower WLAs for the Missoula WWTP than established under the VNRP.

Metals

Metals TMDLs were published in 2014 for Silver Bow Creek and the Clark Fork River and a number of tributaries (Mt. Dept. of Env. Quality, 2014). The Clark Fork River was divided into seven segments, each with its own set of TMDLs for various metals. Not all segments are impaired for the same metals nor at the same level. Missoula is located on the river segment between the confluences of Rattlesnake Creek and Fish Creek. This segment is listed as impaired for copper, iron, and lead and TMDLs and WLAs for these three metals are presented in the 2014 Silver Bow Creek and Clark Fork River Metals TMDLs report (Mt. Dept. of Env. Quality, 2014).

Generally, wasteload allocations are mass-based quantities. However, metals are toxic on a concentration base; therefore, these wasteload allocations are developed as a concentration, generally following the water quality standards presented in MDEQ Circular DEQ-7 (Mt. Dept. of Env. Quality, May 2017). Mass is calculated using the plant discharge flow. The WWTP is considered in compliance with the WLAs as long as effluent concentrations are below the WLAs in the TMDL document and incorporated into the MPDES permit, even while total mass will vary with flow.

The copper WLA for the Missoula WWTP was set to the highest observed effluent concentration at the time of permit writing, while iron and lead WLAs were based on their respective Circular DEQ-7 (Mt. Dept. of Env. Quality, May 2017) water quality standards. The reasoning behind setting copper to the highest measured effluent concentration rather than the lower water quality standard was to provide an achievable limit in light of the overall very small contribution of the treatment plant effluent to copper loads in the river. In addition, MDEQ expects the Superfund cleanup on the main stem of the Clark Fork River to significantly reduce the overall metals loads to the river. Any upstream reduction in load would decrease the required downstream load removal to achieve water quality standards. If upstream removal exceeds downstream removal requirements, assimilative capacity would be created and downstream WLAs would not have to be as strict. However, improvements in overall metals loading in the river would not affect the water quality standards given in Circular DEQ-7 (Mt. Dept. of Env. Quality, May 2017). Table 2-11 summarizes the TMDLs, water quality standards, and wasteload allocations which apply to Missoula.

Table 2-11: Metals TMDLs, Water Quality Standards, and Wasteload Allocations for the Clark Fork River in Missoula and the Missoula WWTP

Pollutant	2014 Metals TMDL Report River Target Concentration at Missoula		2014 Metals TMDL Report Total River Metals Load	
	High Flow	Low Flow	High Flow	Low Flow
Copper	5.77 µg/L	7.1 µg/L	748 lb/d	220 lb/d
Iron	1,000 µg/L	1,000 µg/L	129,600 lb/d	30,915 lb/d
Lead	1.55 µg/L	2.12 µg/L	202 lb/d	66 lb/d
Water Quality Standards[1]	**Instream Concentration**			
Copper	8.2 µg/L			
Iron	1,000 µg/L			
Lead	2.6 µg/L			
WLAs for the Missoula WWTP[2]	**Permit Limit Concentration**		**Corresponding Load (@7.27 mgd)**	
Copper	11 µg/L		0.67 lb/d	
Iron	1,000 µg/L		61 lb/d	
Lead	2.6 µg/L		0.16 lb/d	

1. Per Circular DEQ-7 (Mt. Dept. of Env. Quality, May 2017) and the 2014 Silver Bow Creek and Clark Fork River Metals TMDL report (Mt. Dept. of Env. Quality, 2014).
2. Mass-based wasteload is calculated based on the given concentrations and will vary with effluent flow rate. Per the TMDL document, the WWTP is considered meeting its WLA as long as effluent concentrations are below those listed in this table.

2.5.5. Numeric Nutrient Standards

General

MDEQ developed numeric nutrient standards for most wadeable surface waters in Montana as presented in Circular DEQ-12A (Mt. Dept. of Env. Quality, July 2014). Surface waters in the entire state were evaluated by ecoregion as defined by the 2009 Draft 2 EPA Ecoregions of Montana map (U.S. Environmental Protection Agency, 2009). Among other parameters, streamflows, water quality characteristics, aquatic life and its responses to varying nutrient concentrations in the water were studied over several years to determine baselines for the development of instream nutrient criteria. The resultant numeric nutrient standards by ecoregion are applicable to all wadeable streams within each ecoregion. The Clark Fork River is considered a wadeable stream upstream of the confluence with the Bitterroot River, which includes the segment in Missoula, and is located in ecoregion 17, Middle Rockies. Ecoregion 17 numeric nutrient standards are 30 µg/L for total phosphorous and 300 µg/L for total nitrogen. These standards are very similar to those developed under the VNRP.

Where surface waters are listed on the 303(d) list as impaired for nutrients, MDEQ develops TMDLs and corresponding WLAs based on the nutrient standards developed for the ecoregion of the surface water in question. The Clark Fork River is an exception because it was the only stream with existing nutrient TMDLs when the numeric nutrient standards were introduced. Up until now, development of TMDLs for other regions and surface waters that did not have established TMDLs was more pressing. However, MDEQ will eventually return its attention to the Clark Fork River and reassess the nutrient TMDLs for the Middle and Lower Clark Fork River. Resulting permit limits may be unattainably low, similar to those for most of the permitted facilities in the state.

Nutrient Standards Variance

The State recognized that meeting the extremely low effluent limits required to meet instream standards would be either technologically impossible or economically prohibitive for point source dischargers and developed the variance process. Originally rolled out in 2014 and updated in 2017/18 to be consistent with requirements of a 2015 EPA rule regarding variances, the Nutrient Standards General Variance are presented in Circular DEQ-12B (Mt. Dept. of Env. Quality, May 2018). The General Variance allows for a time period of about 17 years for mechanical plants and up to ten years for lagoons (specific to each facility) to work toward meeting the numeric nutrient standards. If a General Variance is granted, the end-of-pipe effluent limits of 6.0 mg/L and 0.3 mg/L for TN and TP, respectively currently apply to facilities with flows equal to or greater than 1.0 mgd. However, if a plant performed better than that prior to July 1, 2017, the Variance limits will be based on actual plant performance. It is important to understand that these concentration-based limits are converted to loads when incorporated into individual MPDES permits. The loads are calculated based on the design average annual flow of the plant. Effluent reuse is currently not included in the determination, but it is possible that MDEQ will change its approach in the future. The Variance program is currently subject to a tri-annual review and it is possible that the General Variance effluent limits will be lowered as a result of these reviews.

Current Situation for Missoula

The revised General Nutrient Standards Variance was approved by EPA in October 2017 (U.S. Environmental Protection Agency, October 2017). Approval required that MDEQ submit an economic analysis demonstrating that implementation of treatment technologies capable of reaching the numeric nutrient standards would impose widespread economic hardship among permitted communities. The treatment method used for this overall economic analysis was reverse osmosis (RO). As stated by MDEQ and cited by the EPA, RO has consistently been viewed "as the best available technology to get as close to the base numeric nutrient standards as possible, in the absence of dilution" (U.S. Environmental Protection Agency, 2017). This overall analysis did not take into consideration individual differences between facilities as discussed further below but applied RO as the treatment method of choice to all of them. "Widespread economic hardship" was defined as a household wastewater utility bill exceeding 2 percent of the median household income (MHI) in a community. The analysis included current wastewater costs (rates) so that for towns that currently have high wastewater rates due to recent upgrades or small populations the cost for addition of RO treatment resulted in a higher percentage of MHI than cities with currently low wastewater rates. The economic analysis for Missoula concluded that it would only take 1.15 percent of the Missoula MHI to add RO treatment facilities capable of meeting the numeric nutrient standards.

MDEQ submitted the Economic Analysis of Meeting Base Numeric Nutrient Standards to EPA in April 2017 (Mt. Dept. of Env. Quality, April 2017). This analysis included a list of communities likely to require a variance (Mt. Dept. of Env. Quality, 2018). The Missoula WWTP was not included on this list because the facility is currently meeting its existing effluent limits for nutrients, and therefore does not have to implement additional treatment to achieve compliance. However, the EPA approval document (U.S. Environmental Protection Agency, October 2017) of the General Nutrient Standards Variance acknowledges that MDEQ will make the final determination regarding eligibility for a Variance and is not strictly bound to this list. Strategies for dealing with lower numeric nutrient standards for Missoula are discussed below for future permit limits. Table 2-12 lists the development of Missoula nutrient limits in numerical form up to the limits included in the 2015 MPDES permit. It also lists current plant performance and current General Nutrient Standards Variance limits for comparison.

Table 2-12: History of Nutrient Limit Development for the Missoula WWTP

1998	VNRP: Water Quality Standards	Clark Fork above Reserve St.	Clark Fork below Reserve St.
	TN	0.020 mg/L	0.039 mg/L
	TP	0.300 mg/L	0.300 mg/L
	VNRP: Effluent Limits	Concentration	
	TN	10.0 mg/L	
	TP	1.0 mg/L	
	VNRP: EPA WLAs for the Missoula WWTP	Mass	
	TN	891 lb/d (404 kg/d)	
	TP	88 lb/d (40 kg/d)	
2015	Current MPDES Loads incl. credits for trading	Mass	
	TN	910 lb/d	
	TP	101 lb/d	
2017	Current Performance	Average Month Mass (Concentration)	Maximum Month Mass (Concentration)
	TN	590 lb/d (9.9 mg/L)	803 lb/d (13.43 mg/L)
	TP	39 lb/d (0.65 mg/L)	68 lb/d (1.14 mg/L)
2014	DEQ-12A: Water Quality Standards	Concentration	
	TN	0.300 mg/L	
	TP	0.030 mg/L	
2017	DEQ-12B[1]: Current Variance Limits	Concentration	Resultant Load @ 7.27 mgd
	TN	6.0 mg/L	364 lb/d
	TP	0.3 mg/L	18 lb/d
After ~ 2027	Future TMDLs and WLAs	Significantly Lower than Current MPDES Load Limits	

1. Variance limits and requirements are reevaluated every 3 years with the next reevaluation due in 2020.

2.5.6. Future MPDES Permit Limit Considerations

Permit writers depend on a combination of rules passed by EPA and development of state requirements such as TMDLs, numeric nutrient standards, or changes in approaches to the determination of limits. As all of these factors are potentially in flux at any given time, it is impossible to speculate on effluent limits for more than about two permit cycles (10 years) into the future. The following discussion considers current EPA directives, TMDL schedules, numeric nutrient criteria, and multiple conversations with MDEQ staff in developing a recommended approach in planning for future permit limits which would trigger necessary improvements at the WWTP. The Missoula MPDES permit has an expiration date of April 2020. If permit renewal occurs immediately, the two following permits will be in force until 2030. Therefore, any speculation about future permit changes only include a planning period of thirteen years, to 2030.

cBOD$_5$, TSS, and pH

No fundamental permitting changes are expected for cBOD$_5$, TSS, and, pH. The mass-based load limits for TSS and cBOD$_5$ will continue to be used to calculate the effluent concentration limits. As discussed above, if the plant design flow is increased in the future, the resulting concentration limits will be lower than the current limits.

E. coli and Oil & Grease

No permitting changes are expected for *E. coli* and oil & grease in the foreseeable future.

Metals and Cyanide

Copper, Iron, and Lead. Looking ahead to the next one or two permit cycles, effluent limits for these metals are not likely to change. As discussed above, the limits for iron and lead are based on instream water quality standards and no changes to these standards are anticipated in the foreseeable future. The effluent limit for copper is more lenient than the water quality standard and was developed in anticipation of upstream improvements to metals loading as a result of Superfund site cleanup. This cleanup effort is expected to take between 10 and 15 years and its effect on river water quality will not be fully apparent until then. On this basis, no change in the copper limit is expected in the next two permit cycles.

Plant staff have been monitoring upstream background concentrations and hardness and have found that the measured river hardness is higher than the hardness used by MDEQ in the calculation of the water quality standards. The difference in hardness based on 2017 data would only increase the calculated standard for copper by less than 1 mg/L. Unless, additional sampling documents a long term higher hardness, the permit limit is not expected to be revised based on new hardness data.

Cyanide. Plant staff are currently required to sample for cyanide in the plant effluent and Clark Fork River upstream of the outfall to allow MDEQ to determine reasonable potential for exceeding water quality standards in the next permit renewal. Sampling results to date show non-detects with a reporting limit of 5 µg/L for the majority of the samples. Circular DEQ-7 includes a required reporting value (RRV) of 0.3 µg/L for cyanide because some of the applicable standards are less than 5 µg/L. The human health standard for surface water, for example, is 4 µg/L. With non-detect results at a reporting level of 5 µg/L, MDEQ would use the reporting level to calculate reasonable potential to exceed the water quality standards. This would mean that the effluent exceeds standards and would likely result in a permit limit for cyanide in the next permit renewal. Plant staff was informed of the Circular DEQ-7 requirements and instructed the analytical laboratory to use a lower RRV for future samples.

Ammonia

No fundamental changes to ammonia limits are expected. If plant or river data warrant minor adjustments due to changes in recorded effluent concentrations or changes in river pH or temperature, the limit could fluctuate. There are no indications from MDEQ that the approach to data use, statistics, and calculation of ammonia limits will be changing in the foreseeable future.

Dissolved Oxygen

In the past, MPDES permits often stated that facilities that remove BOD from the wastewater to reasonable effluent concentrations are not considered to be exerting an oxygen demand on the receiving stream and therefore did not receive effluent DO limits. In recent years, MDEQ has focused more on effluent dissolved oxygen and compliance with instream water quality standards as evidenced by the special condition of a DO study in Missoula's 2015 MPDES permit. It is assumed that MDEQ will continue to be vigilant about effluent DO concentrations and consideration to stream DO requirements was given during the alternative analysis.

Circular DEQ-7 (Mt. Dept. of Env. Quality, May 2017) provides instream DO standards applicable to the Clark Fork River at Missoula as follows:

- 30-Day Mean: 6.5 mg/L
- 7-Day Mean: 9.5 mg/L
- 7-Day Mean Minimum: 5.0 mg/L
- 1-Day Minimum: 8.0 mg/L

Stream temperature data show that summer time temperatures in the river easily reach 17°C and above, which results in saturated DO concentrations of about 8.5 mg/L or less, which do not achieve the required 7-day mean of 9.5 mg/L. Stream DO data bears this out, showing average summertime DO concentrations of about 8.2 – 8.9 mg/L as reported by the 2015 permit Fact Sheet.

The average effluent DO concentration from 2013 through November 2017 was about 5.2 mg/L. This concentration is relatively high because the last treatment zone in the bioreactors is aerated. For treatment systems that finish with an anoxic zone and do not include re-aeration, effluent DO may be as low as 0.5 mg/L. If the 2037 maximum month flow of 14.3 mgd were mixed with 100 percent of the 7Q10 flow (574 cfs) of the Clark Fork River, the resulting DO concentration at the point of mixing would be about 0.41 mg/L below a saturated DO concentration or 8.49 mg/L.

The 2008 Mixing Zone Study determined that the plant's effluent plume does not mix with the full 7Q10 flow within the straight river segment near the WWTP outfall. The study instead presents the available flow for instantaneous mixing (two river widths downstream of the outfall) at 7Q10 flow condition, which is 86.1 cfs. If this flow is used in the DO calculations, the resulting DO concentration at the point of mixing would be 1.67 mg/L below the saturated DO concentration or 6.8 mg/L. As no additional significant oxygen demand exists, there would be no DO sag and DO concentrations would be allowed to increase slowly as the effluent plume mixes with additional ambient water. Simple DO calculations, such as the Streeter-Phelps equation and mixing zone models such as Cormix can approximate the DO in the straight river segment near the outfall. However, sophisticated modeling may be needed to predict DO concentrations and mixing behavior downstream of the straight river segment as the river bed divides into multiple meandering channels. Mixing and re-aeration through this meandering river section is assumed to be good due to shallower water depths and increased twists and turns, both of which favor surface aeration. Necessity for such a study should be discussed with MDEQ when effluent DO concentrations become an issue.

Nutrients – TN and TP

Based on the information presented on numeric nutrient limits and the TMDL process for nutrients in the Clark Fork River, nutrient effluent limits are not expected to change within the next two to three permit cycles. By then, at least two additional tri-annual reviews of the Variance program will have occurred and the program may have been revised again. It will be important to regularly check potential revisions to the Nutrient Standards Variance program as well as scheduling and progress of development of a new TMDL on the Middle and Lower Clark Fork River to have early warning of lower nutrient limits and their implications. The trigger for future nutrient treatment planning would be listing of a Clark Fork River nutrient TMDL on MDEQ's schedule.

An additional twist to the story is the current lawsuit of Upper Missouri Waterkeeper vs EPA, DEQ, and others, in which the US District Court of Montana recently decided in favor of Waterkeeper on one issue: the District Court found the General Nutrient Standards Variance to not be in compliance with the intent of the Clean Water Act in that it does not currently provide a pathway toward meeting the stringent instream standards. Appeals to this decision are likely to happen and speculation on the impact of this lawsuit on Missoula is premature at this time. Since the Missoula limits are not currently relying on a Variance, there will be no immediate effect on the Missoula nutrient limits.

It is difficult to predict the numeric value of future lower nutrient limits. Assuming that a new Clark Fork River TMDL is modeled after other existing nutrient TMDLs in the state, resulting instream nutrient standards would be unachievable with conventional treatment. Therefore, application for a Nutrient Standards Variance would become necessary. As Missoula is not included as approved for a General Variance, the application would need to be for an Individual Variance. Individual Variances may be granted due to economic impacts, limits of technology, or both. An earlier version of Circular DEQ-12B (Mt. Dept. of Env. Quality, May 2018) also included site-specific water quality modeling as a route towards obtaining an individual variance. This option was removed from the May 2018 edition of the Circular because it is already covered under ARM 17.30.660 (4) and offers a third avenue for requesting an individual variance. When the time comes that Missoula may need to apply for a variance, the outcome of the Waterkeeper lawsuit may affect the process and available options. The following discussion assumes that the Variance process will be in place as currently administered.

In order to make a case for economic hardship, the City would need to demonstrate that implementation of treatment capable of meeting the new numeric nutrient limits would create economic hardship, i.e., cost more than 2 percent of MHI. In conversations with Mike Suplee, Water Quality Planning, MDEQ, he stated that in the case of individual variances, the treatment method used to show economic impact should be the most applicable for the particular facility, rather than the blanket use of RO. Potential treatment strategies and associated cost estimates are discussed in Chapter 8. If economic hardship can be shown, MDEQ would submit this documentation demonstrating economic hardship to EPA and EPA would review this information as part of its approval process.

Given the current MDEQ economic analysis result, it may not be possible to show economic hardship as a result of sweeping treatment upgrades; however, the technology to actually meet required standards may still not exist. As mentioned above, RO may not be able to meet the stringent Circular DEQ-12A (Mt. Dept. of Env. Quality, July 2014) nitrogen limits currently in place for most of Montana, even though it is considered the best available technology. This means, that any other technology, short of stopping discharge to the river altogether, would not be able to meet those stringent limits. Therefore, a variance based on showing that available technology and disposal methods are not able to achieve the required standards may be a better approach. This approach would require a new analysis of available treatment technologies specific to Missoula at the time of the analysis to support this argument.

In addition, Circular DEQ-12B (Mt. Dept. of Env. Quality, May 2018) still leaves the door open for MDEQ to use compliance schedules, effluent reuse, and nutrient trading as regulated by Circular DEQ-13 (Mt. Dept. of Env. Quality, December 2012) to work with communities toward meeting strict nutrient limits. Nutrient trading could be attractive when investment in the reduction of non-point source nutrient discharges to the Clark Fork River would yield larger reductions in nutrient loads than if the same amount of money were used to upgrade plant processes. As part of the strategy to respond to lower nutrient limits, searches for additional nutrient trading opportunities may yield relatively lower cost options for reducing nutrient loads to the river. Trading may either augment treatment changes at the WWTP or even replace them, depending on the calculated nutrient offset.

Site-specific water quality modeling may be pursued if existing river data suggests that “greater emphasis on the reduction of one nutrient may achieve similar water quality and biological improvements as would the equal reduction of both nitrogen and phosphorous.” (ARM 17.30.660, 2014) River sampling and modeling may be used to determine the impact of the WWTP discharge on river health. A river study would target separate analysis of the influence of phosphorous and nitrogen on river health to determine if one or the other nutrient has more impact or is limiting, therefore directing treatment and removal strategies, as well as permitting. The Clark Fork River has been sampled and studied for two decades and comprehensive data for both nutrients is available for the river, including sampling stations upstream and downstream of Missoula. Additional sampling may be advisable to create a database specifically for evaluation of the WWTP impact immediately downstream of the outfall. Coordination with MDEQ, the Clark Fork Coalition, the Missoula Valley Water Quality District, and others is encouraged to leverage all available data and minimize the need for additional sampling.

THE GARDEN CITY
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WASTEWATER FACILITY PLAN

CHAPTER 3 - COLLECTION SYSTEM DATA

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CHAPTER 3 COLLECTION SYSTEM DATA

3.1. EXISTING COLLECTION SYSTEM DATA

As described in Chapter 2, the City of Missoula collection system currently serves an area of 17,114-acres with a population of 68,015 including group quarters. The collection system is comprised of manholes, gravity mains, lift stations, force mains, STEP tanks, and community STEP tanks.

The City has not had the benefit of a hydraulic model of the collection system for use in evaluating capacity of the system and expansion of services. A hydraulic model was developed as part of this facility planning effort using the City's existing GIS data as a starting point and collecting additional data to complete the model. Chapters 5 and 6 present modeling information for existing and future conditions.

3.1.1. Existing GIS Data

The City of Missoula currently uses a Geographic Information System (GIS) to store mapping and attribute information for the wastewater collection system. The GIS is built within the Environmental Systems Research Institute (ESRI) software platform and uses ArcGIS as the interface. The following available collection system GIS data sets (accessed September 2017) were used in the analysis for this Facility Plan:

- Manholes, Lift Stations, STEP Tanks, and Community STEP Tanks:
  - Total number
  - Status (connected, abandoned, other)
  - Facility Identification Number
  - Referenced construction project number
  - Year constructed
- Gravity Mains, Force Mains (polyline shapefile):
  - Total number
  - Status (connected, abandoned, other)
  - Facility Identification Number
  - Referenced construction project number
  - Year constructed
  - Gravity main type
  - Slope
  - Calculated Length
  - Diameter
  - Material
  - From and To Manhole

3.1.2. Default Values

Critical attributes required for hydraulic model development were determined using industry standards. The Manning's roughness coefficients used for the various gravity pipe materials, and Hazen-Williams friction coefficients for the pressurized force main materials are available from numerous published sources. The hydraulic modeling software used for this analysis provides Manning's "n" and Hazen-Williams "C" values for numerous pipe materials and was utilized as the primary reference.

Manhole diameters were collected as part of the field survey and were utilized in the hydraulic model. The manhole diameters associated with the 12-inch diameter and larger gravity mains were either 4-feet or 6-feet, with the 6-feet diameter manholes typically connected to gravity mains of 18-inch or larger. The default values for any additional manholes included in the hydraulic model for extensions or future analyses utilize the same manhole diameter assignment based upon gravity main size.

3.1.3. Filling in Data Gaps

Several data gaps existed in the City GIS data sets, including facility ID, manhole attributes, gravity and force main pipe attributes, and "To Manhole" and "From Manhole" information. In most cases, partial information was available but was not complete.

Data gaps in the available shapefiles for manhole attributes of rim elevation, depth, and invert elevations were filled by conducting a field survey in the winter of 2017-2018 as further detailed below. Missing diameter and materials information for one third to one half of the gravity sewer and force main pipes was obtained from review of record drawings and field observations or deduced from construction year and properties of adjoining pipe sections.. The missing gravity main information for "To Manhole" and "From Manhole" attributes were assigned manually during model development with input from field survey information, City staff, and deductions from information available for adjoining components.

3.1.4. Additional Data Used for Flow and Loading Development

In addition to the GIS data, an active wastewater account spreadsheet was provided for users on the wastewater system. These were geocoded in order to compare water usage data (provided in GIS form) to wastewater data to develop loads in the current system. Water meter data was provided and the months of November and December were used to develop non-irrigation water usage to correlate to wastewater loadings. Finally, transportation analysis zone (TAZ) data was provided as described in Chapter 2. The TAZ data for residential and non-residential population for 2017 and 2037 was used to develop existing and future wastewater loading.

3.1.5. GIS Recommendations

The following GIS data set recommendations are provided for consideration for future maintenance of the collection system GIS data. The primary goal of the recommendations is to increase reliability and completeness of collection system GIS data for future facility planning efforts or updates. Implementation of the recommendations would also improve the usefulness of the GIS data for routine maintenance, troubleshooting, and for future use in extended collection system modeling.

- Assign a unique Facility ID to all existing GIS cataloged collection system components including manholes, gravity mains, lift stations, force mains, STEP tanks, and community STEP tanks. This is needed in order for use of the GIS data in the collection system model.
- Survey and map all manholes that connect gravity mains less than 12-inch diameter. The survey should capture at a minimum the x and y coordinates of the manhole in addition to rim and invert elevations. Invert elevations can be calculated from measured depths of invert from rim. Secondary data acquired during manhole survey should include manhole diameter and material of construction.
- Complete the GIS shapefile attributes associated with gravity mains and force mains. The critical fields for assessment, planning, and modeling purposes include the main type, year built, material, diameter, slope, and upstream and downstream manhole IDs.
- Develop a polylines shapefile for wastewater service lines to individual parcels. Develop a polygons or points shapefile for wastewater service accounts.
- Include records of the inspections, maintenance, repairs, surcharges, etc. to associated collection system components. Records could include date of activity, tasks undertaken, and hyperlinks to observation notes or other relevant files.

3.2. FIELD SURVEY

Given the lack of invert elevation attributes included in the existing manhole and gravity main shapefiles, a field survey of manholes was performed as part of this project. The field survey was designed to obtain elevation information critical to accurate hydraulic model development.

A total of 1,253 of the approximately 7,550 existing manholes (17 percent) were included in the skeletonized collection system of gravity mains with predominantly 12-inch and larger diameters that were identified for hydraulic model development. Fifty-one manholes were inaccessible, verified abandoned, non-existent, duplicated, or owned by the University of Montana at the time of survey and were not surveyed. A total of 1,202 manholes were surveyed by Morrison-Maierle surveyors.

The data collection and quality assurance process employed yielded repeatable observations within estimated positional accuracies of 0.10-feet in horizontal and 0.15-feet in vertical reported values at 95 percent confidence.

Additional manhole information was also obtained to be used as supporting information for hydraulic model development or the condition assessment. The additional data included observations of the inlet and outlet pipe diameters, pipe material, manhole diameter, drop manhole measurements, flow direction, unique observation notes, and a photograph of each manhole. A comprehensive spreadsheet with all observed and recorded information was submitted to the City previously.

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WASTEWATER FACILITY PLAN

CHAPTER 4 - EXISTING COLLECTION SYSTEM DESCRIPTION

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CHAPTER 4 EXISTING COLLECTION SYSTEM DESCRIPTION

4.1. INTRODUCTION

The City of Missoula wastewater collection system serves an area of approximately 17,600 acres with a population of 68,000. The collection system is comprised of approximately 7,550 manholes, 327 miles of gravity sewer mains, 46 lift stations, and 25 miles of lift station force mains. The collection system also includes approximately 1,400 septic tank effluent pump (STEP) systems, 150 dry laid STEP systems, and 13 community STEP systems with an additional seven miles of pressurized STEP mains. All wastewater is conveyed to the City of Missoula Wastewater Treatment Plant (WWTP) located near Mullan Road and Reserve Street. This chapter provides an overview of the existing collection system, describes the hydraulic model developed for the City, and provides an evaluation of the existing collection system. Figure 4-1 provides an overview of the existing City of Missoula wastewater collection system and shows all major system components.

4.2. GRAVITY SYSTEM

4.2.1. Age and Material

As mentioned in Chapter 3, the GIS data sets have incomplete records of critical attributes such as gravity main and force main diameter, material, and year constructed. Therefore, statistical analysis alone would not accurately or fully describe the condition of the entire existing collection system. Therefore, a truncated statistical analysis was performed in conjunction with input from City staff to evaluate the existing collection system condition. Based on the analysis of GIS dataset attributes presented in Chapter 3, Table 4-1 categorizes gravity main segments (excluding STEP tank effluents) and their respective materials and age.

Table 4-1: Existing Gravity Main Material and Age

Year Built	Pipe Age (years)	Total Length (miles)	Primary Material[1]	Percent of Total Length[2]
Before 1950	> 67	42.0	VCP	12.5
1950 - 1959	67 - 57	10.1	VCP, ACP	3.0
1960 - 1969	57 - 47	43.5	VCP, ACP	12.9
1970 - 1979	47 - 37	48.2	VCP, ACP, PVC	14.3
1980 - 1989	37 - 27	26.6	PVC	7.9
1990 - 1999	27 - 17	56.2	PVC	16.6
2000 - 2009	17 - 7	87.1	PVC	25.8
After 2010	< 7	23.5	PVC	7.0

[1] Pipe Material abbreviations: VCP = vitrified clay pipe; ACP=asbestos cement pipe; PVC=polyvinyl chloride pipe.
[2] Total length is the calculated total length of active gravity mains in the collection system with material reported in the gravity main shapefile, 327 miles.

Observations and conclusions about the gravity mains include the following:

- Between 1970 and 1979 a shift in the primary materials used in gravity main construction from VCP or concrete derivatives to PVC. This coincides with industry introduction of PVC for gravity sewer main applications.

- Of the active gravity mains with material attributes reported, nearly 43 percent of the collection system was installed prior to 1980 and consists primarily of VCP and ACP. Vitrified clay pipe and ACP are rigid pipes that are susceptible to leakage due to joint separation and cracking due to ground movement such as settling. These pipe materials are assumed to be a significant source of infiltration in the collection system due to the age, antiquated technology of manufacture and jointing, and prevalence in the low-lying areas adjacent to the Clark Fork River.

- The remaining 57 percent of the collection system was installed after 1980 and is comprised primarily of PVC, which is a flexible pipe with more technologically advanced manufacturing and jointing methods.

4.2.2. Characterization

To provide a comprehensive method of collection main modeling, evaluation, and reporting, the existing gravity sewer system is differentiated by lateral and trunk mains. Lateral mains, sometimes referred to as collectors, make up the smallest diameter mains in the system. These mains are designed to collect wastewater from individual residences, commercial and industrial properties. For the purposes of this report, lateral mains are classified as pipes with diameters 10-inches and smaller. The City of Missoula gravity collection system is primarily comprised of 8-inch diameter lateral mains with approximately 216 miles of the total 328 miles. However, the gravity collection system also contains 4-inch and 6-inch diameter pipes which were installed prior to the establishment of Montana Department of Environmental Quality (MDEQ) Circular DEQ-2 (Mt. Dept. of Env. Quality, 2016) design standards. Overall, lateral mains account for approximately 82 percent of the total collection system gravity main lineal footage.

For the purposes of this report, trunk mains are classified as pipes with diameters 12-inches and larger serving major drainage basins. The primary purpose of trunk mains is to intercept wastewater flows from laterals, but can also collect additional individual service flows when collectors and laterals are not within close proximity to users. Trunk mains account for approximately 18 percent of the total collection system footage.

The largest diameter mains are often classified as interceptor mains. These mains intercept and accumulate wastewater flows from multiple drainage basins. They are typically fed by trunk mains or other interceptors. The Missoula collection system has two interceptor mains that collect flow from the entire collection system:

- The first interceptor is a combination 30-inch and 42-inch main located in and adjacent to Clark Fork Road (northwest access to WWTP) which collects all wastewater flows originating on the north side of the Clark Fork River and conveys flow to the WWTP. The interceptor is approximately

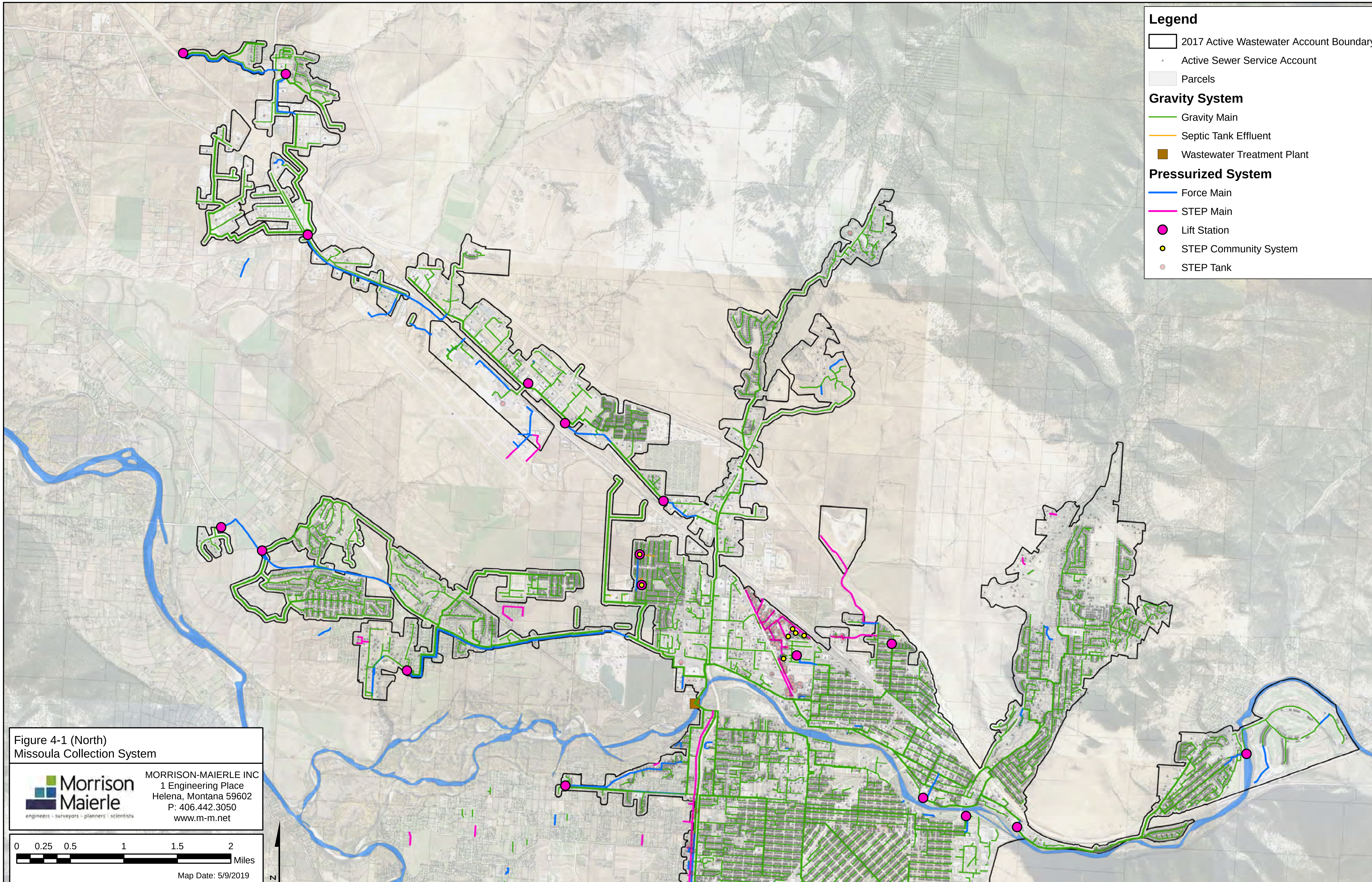

Figure 4-1 (North)
Missoula Collection System

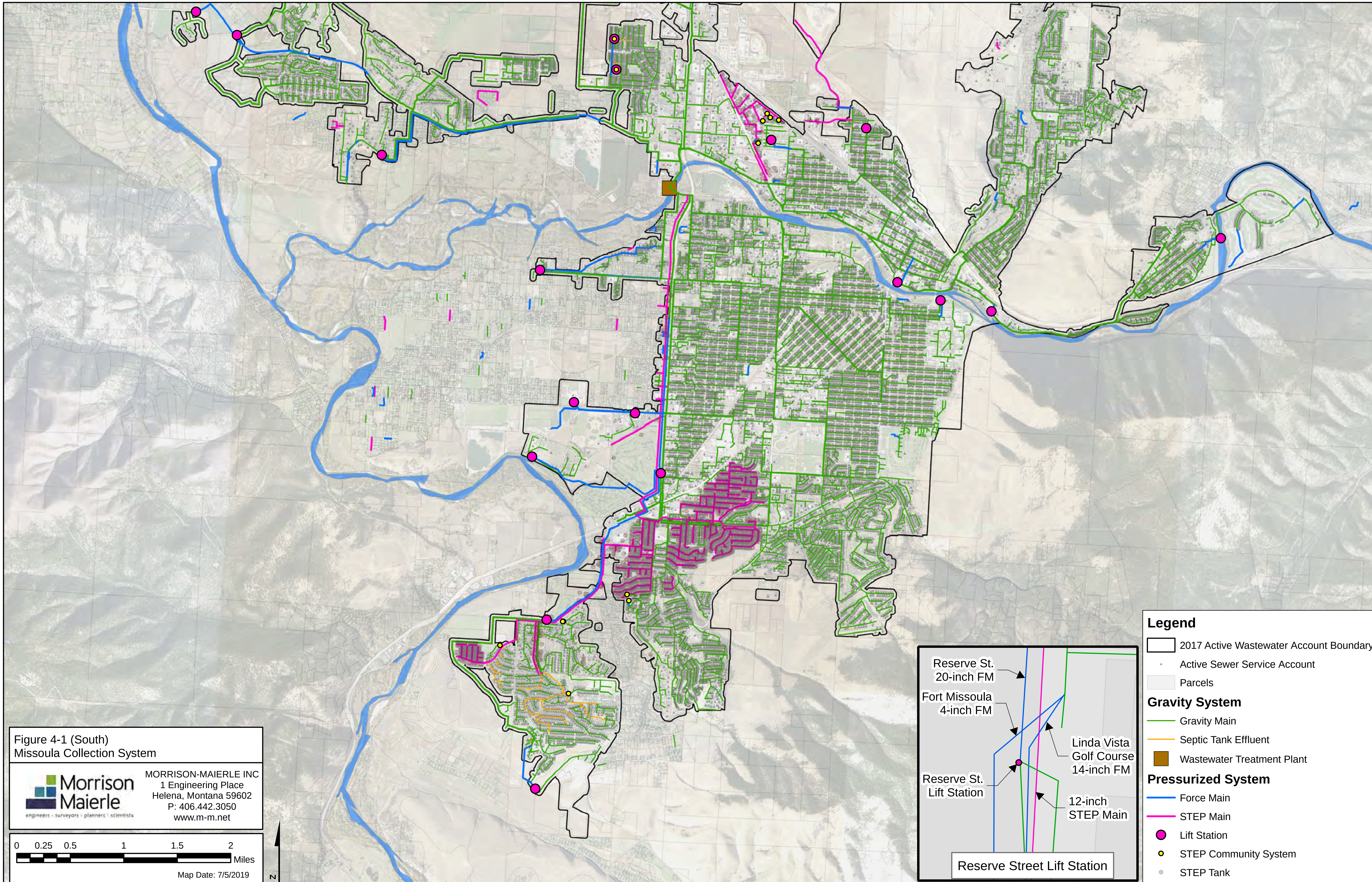

Figure 4-1 (South)
Missoula Collection System

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2,150 feet long and collects flow from four major drainage basins. Approximately 650 feet of the 30-inch interceptor built in 1977 is in parallel with a newer 42-inch main constructed in 2011. When the newer 42-inch interceptor was constructed, manhole P01-6-12 was installed with a flow split that conveys wastewater through both the 42-inch and 30-inch mains. Manhole P01-6-12 is shown on Figure 4-2.

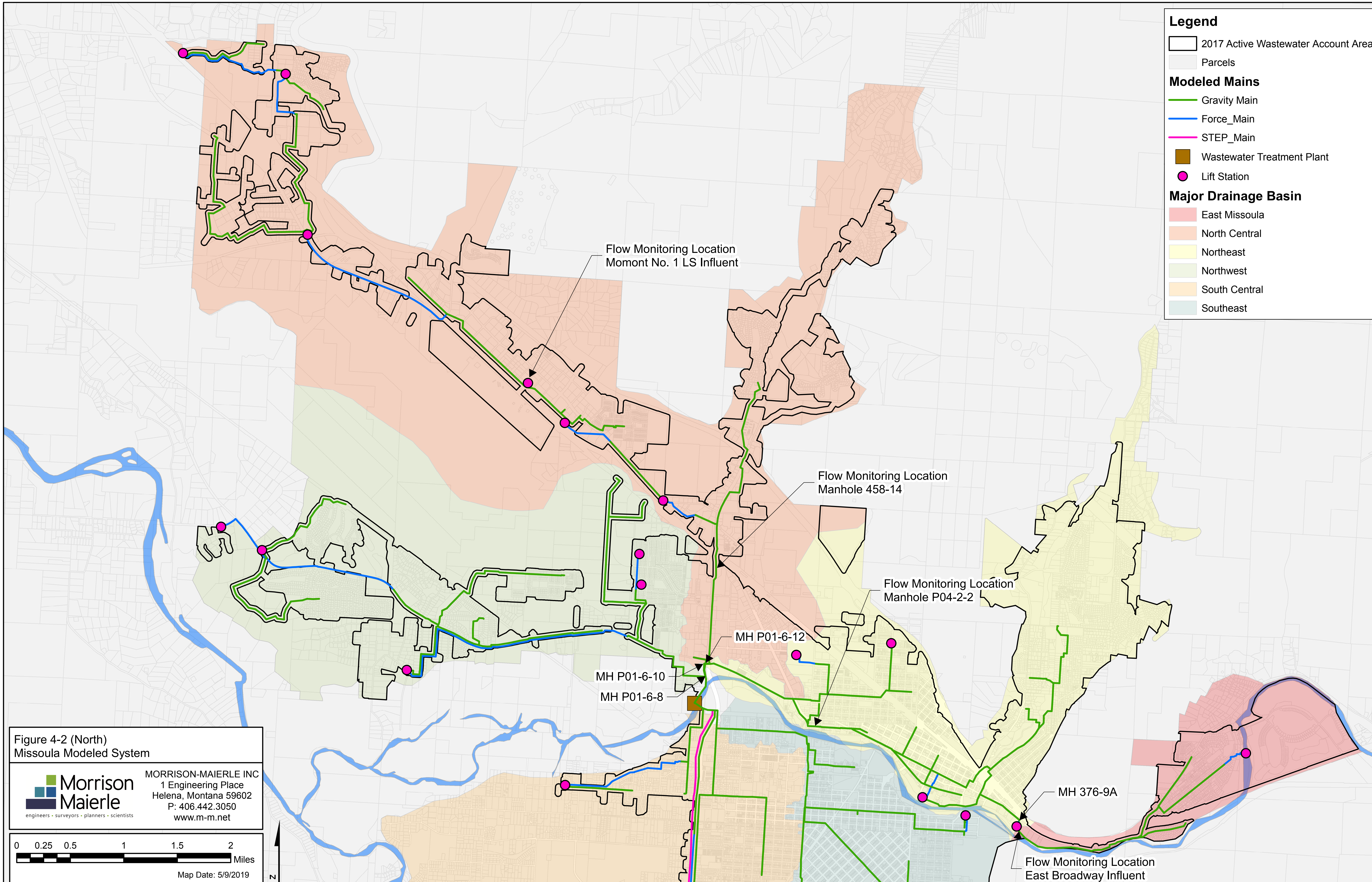
- The second interceptor main is a 36-inch main that crosses the Clark Fork River and Reserve Street immediately east of the WWTP, ending at the intersection of Davis Street and River Road. This interceptor collects all wastewater flows originating on the south side of the Clark Fork River and conveys flow to the WWTP. The interceptor is approximately 2,078 feet long and collects flow from three major drainage basins.

Table 4-2: Summary of Trunk Mains

Trunk Main / Major Drainage Basin	Discharge	Size (in)	Length (ft)	Avg. Slope (ft/ft)	Year Installed	Avg. Manning
North Central	North Interceptor MH P01-6-12	30	290	0.0076	1971-2011	0.010
		24	1,340	0.0024		
		21	270	0.0045		
		18	19,970	0.0139		
		15	15,290	0.0099		
		12	22,680	0.0101		
		10	1,670	0.0023		
		8	1,430	0.0022		
Northwest	North Interceptor MH P01-6-8	36	4,510	0.0006	1977-2016	0.010
		30	8,340	0.0004		
		24	6,090	0.0013		
		18	14,810	0.0051		
		15	3,990	0.0032		
		12	19,730	0.0064		
		10	150	0.0044		
		8	1,090	0.0051		
Northeast	North Interceptor MH P01-6-10	36	6,060	0.0026	1910-2005	0.011
		30	4,640	0.0053		
		27	900	0.0059		
		24	20	0.0052		
		21	4,930	0.0098		
		20	1,850	0.0035		
		18	3,530	0.0090		
		15	20,660	0.0082		
		14	1,530	0.0036		
		12	10,440	0.0103		
		10	1,980	0.0030		
		8	1,030	0.0040		

Trunk Main / Major Drainage Basin	Discharge	Size (in)	Length (ft)	Avg. Slope (ft/ft)	Year Installed	Avg. Manning
South Central	South Interceptor [1] 27-inch @ MH C61-6 [2] 30-inch @ MH P09-48-1 [3] 36-inch @ MH P98-38-1	36 30 27 24 21 20 18 15 12 10 8	4,800 4,550 5,020 2,760 11,300 4,990 3,800 5,050 11,110 1,220 2,490	0.0028 0.0015 0.0020 0.0016 0.0014 0.0058 0.0019 0.0024 0.0047 0.0026 0.0039	1957-2008	0.012
Southeast	South Interceptor MH C61-6	36 30 27 24 21 20 18 15 12 10 8	4,320 3,340 170 6,270 6,140 2,640 6,610 9,260 4,730 360 380	0.0016 0.0071 0.0075 0.0004 0.0039 0.0009 0.0030 0.0031 0.0031 0.0015 0.0087	1927-2001	0.013
Southwest	South Central Trunk / Drainage Basin [4] MH C65-T33C	15 12	6,680 9,490	0.0113 0.0355	2009-2016	0.010
East Missoula	Northeast Trunk / Drainage Basin [5] MH 376-9A	15 12 10	11,510 2,800 3,810	0.0103 0.0026 0.0163	1973-2002	0.010

[1] 27-inch trunk main conveys S. Reserve St. gravity main collection and Linda Vista Golf Course lift station flows.
[2] 30-inch short length trunk main primarily conveying S. Reserve St. lift station flows.
[3] 36-inch short length trunk main primarily conveying flows immediately south and east of WWTP.
[4] MH C65-T33C receives flow from Linda Vista Golf Course Lift Station.
[5] MH 376-9A receives flow from East Broadway Lift Station.



Legend
2017 Active Wastewater Account Area
Parcels
Modeled Mains
Gravity Main
Force_Main
STEP_Main
Wastewater Treatment Plant
Lift Station
Major Drainage Basin
East Missoula
North Central
Northeast
Northwest
South Central
Southeast
Flow Monitoring Location
Momont No. 1 LS Influent
Flow Monitoring Location
Manhole 458-14
Flow Monitoring Location
Manhole P04-2-2
MH P01-6-12
MH P01-6-10
MH P01-6-8
MH 376-9A
Flow Monitoring Location
East Broadway Influent
Figure 4-2 (North)
Missoula Modeled System
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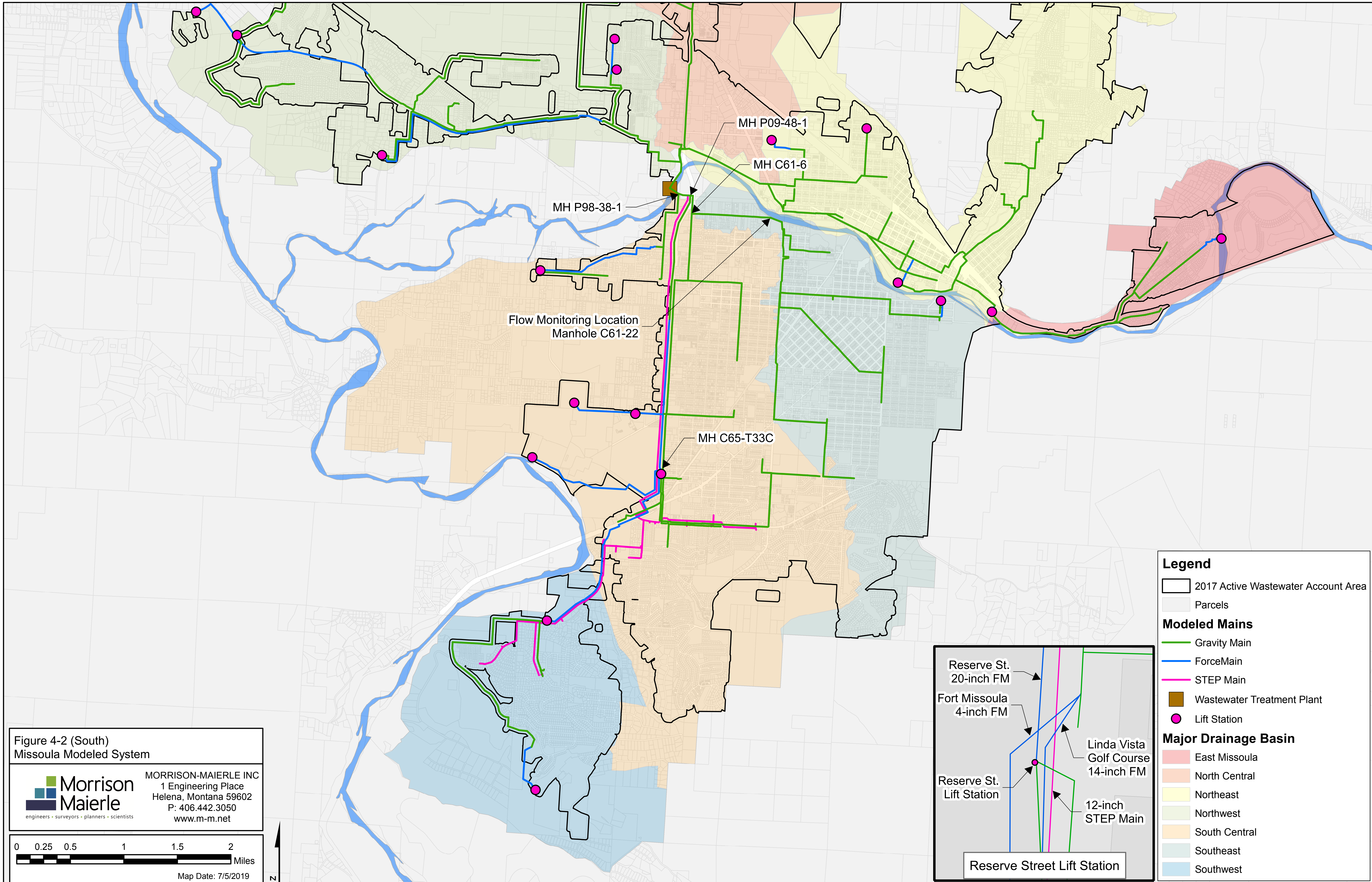

Figure 4-2 (South)
Missoula Modeled System

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4.2.3. Known Gravity Collection System Issues

Lincoln Hills Area

Missoula collection system maintenance personnel are aware of concrete pipe deterioration in the Lincoln Hills area. The Lincoln Hills portion of the collection system serves an area of approximately 200 acres, and is generally bound by Lincoln Hills Drive and Mountain View Drive to include the area east of Rattlesnake Drive. GIS data confirms that the collection system materials are predominantly ACP or other concrete products. Collection system maintenance personnel suspect the deterioration is associated with poor manufactured quality of the concrete mains. Hydrogen sulfide may also be a contributing factor to the concrete deterioration despite the absence of STEP systems in the area. Maintenance to manage root intrusion is performed annually.

Reserve Street and Brooks Street Intersection

An area of known hydrogen sulfide deterioration of existing concrete gravity mains occurs in the vicinity of the intersection of Reserve Street and Brooks Street due to conveyance of STEP systems south of Brooks Street. Concrete gravity mains in this area range from 8-inch to 21-inch diameter, and ultimately discharge to lift station P91-1-LS. These concrete gravity mains convey wastewater originating from the majority of STEP tanks within the collection system, which are known causal points of hydrogen sulfide formation.

3rd Street Corridor West of Reserve Street

An area of known infiltration and inflow is the 3rd Street corridor west of US Highway 93 (Reserve Street). The 3rd Street gravity main serves an area of approximately 500 acres generally bound by the Clark Fork River to the north and S. 7th Street W. to the south. The inflow problem is known by collection system staff to be residents dewatering basements by pumping into the sanitary sewer system. Gravity main diameters in the area range from 8-inch to 36-inch. The predominant material of the gravity mains is PVC, with some segments of concrete pipe associated with diameters 21-inch and 30-inch.

Aging Vitrified Clay and Asbestos Cement Pipe

Over 30 percent of the collection system is older than 40 years and consists of vitrified clay pipe (VCP) and asbestos cement pipe (ACP). While asbestos cement pipe generally displays better longevity than vitrified clay pipe, both require close monitoring due to their age. Vitrified clay pipe is prone to breaks caused by root intrusion and seismic events due to the brittle nature of its material. Especially in low-lying areas, this may lead to excessive infiltration of groundwater. Asbestos cement pipe is susceptible to corrosion caused by hydrogen sulfide gas as experienced throughout and downstream of the southern STEP systems. Asbestos cement pipe also presents the greatest hazard during emergency repairs, because it cannot be cut without releasing friable asbestos. Staying ahead of deteriorating VCP and ACP will mitigate groundwater infiltration, reduce the need for emergency repairs, and allow for planning of ACP pipe rehabilitation in a safe manner that protects workers and the environment. Therefore, prioritization of replacement of VCP and ACP in the City's annual sewer rehabilitation program is recommended, especially in areas of high groundwater and downstream of STEP systems.

4.2.4. Comparison to Circular DEQ-2 Standards

The Circular DEQ-2 provides guidance and design requirements for gravity sewer mains, lift stations, and force mains and was used to assess the existing collection system condition. However, more than 50 percent of the collection system was installed before 1990 and prior to the adoption of Circular DEQ-2 (1995) and deviations from the now-current standards are acceptable for these older portions of the collection system with appropriate justification.

Recommended Minimum Slopes

The following table presents the minimum slopes that must be provided per Circular DEQ-2 compared to existing slopes calculated from the City collection system GIS data for gravity mains, dated September 2017. As reported in Chapter 3, the gravity main slope attributes were not available for all pipe segments. Therefore, gravity mains with no slope attribute were omitted from the statistics presented below. Also, the main segments with slope attributes were primarily obtained from design or record drawings during development of the gravity mains shapefile, with accuracy assumed to be commensurate with the quality of design and construction practices.

Table 4-3: Existing Gravity Main Minimum Slopes

Gravity Main Diameter (in)	Circular DEQ-2 Min. Slope (%)	Length Meeting Min. Slope (ft)	Length Less Than Min. Slope (ft)	Percent of Total Length Less Than Min. Slope (%)
6	0.60	38,894	3,204	7.6
8	0.40	915,809	156,537	14.6
10	0.28	155,919	14,764	8.7
12	0.22	71,642	7,566	9.6
14	0.17	1,521	0	0.0
15	0.15	67,769	5,837	7.9
16	0.14	0	0	0.0
18	0.12	48,505	702	1.4
21	0.10	19,179	3,573	15.7
24	0.08	13,193	2,308	14.9
27	0.067	4,677	0	0.0
30	0.058	19,953	0	0.0
33	0.052	0	0	0.0
36	0.046	20,448	0	0.0
39	0.041	0	0	0.0
42	0.037	614	0	0.0
Total		**1,378,123**	**194,491**	**12.4**

Of the active gravity mains reported with slope attributes, 12.4 percent by length have slopes that do not meet the minimum requirements set forth in Circular DEQ-2 including segments of mains constructed prior to 1995. Shallow slopes throughout the collection system are a known issue to collection system maintenance personnel. Some gravity mains were known or postulated to have had flush tank facilities in the past. Flush tank facilities were typically connected to the domestic water supply to provide a source of cleaning water to periodically flush gravity mains, thereby allowing main segments to be installed with shallower than normal slopes. Such physical cross connections between sanitary sewer and domestic water are no longer permitted by Circular DEQ-2, and the City of Missoula has removed known cross connections including flushing facilities.

Manhole Location and Spacing

Circular DEQ-2 requires manholes to be installed at the end of each sewer line; at all changes in grade, size, or alignment; at all intersections; and at distances not greater than 400 feet for sewers 15-inches or less in diameter, and 500 feet for sewers 18 to 30-inches. Distances up to 600 feet may be approved where cleaning is provided. Statistics obtained from the gravity main shapefiles regarding manhole spacing are provided below.

- 494 of 15-inch and smaller gravity main segments in excess of 400 feet (40 of which are in excess of 500-ft).
- 5 of 18-inch to 30-inch gravity main segments in excess of 500 feet.
- 9 of 36-inch and larger gravity main segments in excess of 500 feet (all near outfall to WWTP).

The manhole spacing analysis shows that approximately 6.4 percent of manholes in the existing collection system do not meet the published spacing requirements of Circular DEQ-2 including many older sections of mains installed prior to 1995. The majority of 18-inch and larger main segments exceeding manhole spacing requirements occur in the vicinity of the wastewater treatment plant where mains cross the Clark Fork River or adjacent riparian areas.

4.3. LIFT STATIONS

As of 2017, there were 46 active lift stations located within the City of Missoula with two additional lift stations in construction. Of the 46 active lift station, 39 are owned and operated by the City. Table 4-4 summarizes pertinent data about the 39 lift stations owned and operated by the City and one non-City owned lift station that serves a relatively large area, Travois Village. The remaining eight lift stations not listed are owned and operated by entities other than the City of Missoula and serve single buildings or small developments. Five of the 39 city-owned lift stations discharge to the STEP collection system in the southern portion of the City. Chapter 5 presents more information on those lift stations included in the modeling effort.

Figure 4-3 shows the locations and modeled single pump capacities of lift stations discharging to the gravity main collection system.

Table 4-4: Summary of City Owned Lift Stations

Lift Station Name	Year Lift Station Installed	Wet Well Dia. (ft)	No. of Pumps	Year Force Main Installed	Force Main Dia. (in)	Force Main Material (in)	Force Main Length (ft)	Discharge Trunk / Drainage
Grant Creek	1980	8	2	1980	10	DIP	1,910	North Central
Traynor Dr.	2010	4	2	2010	4	PVC	1,000	Northeast
Caras Park	2016	10	2	2016	14	PVC	1,580	Northeast
Fort Missoula	NA	6	2	1997	4	VCP	8,250	South Central
Dickens St.	1973	5	2	1973	4	DIP	30	Northeast
D.J. Ct.	2000	Septic Tank	2	2000	2	PVC	20	South Central [1]
Birdie Ct.	2000	Septic Tank	2	2000	2	PVC	20	South Central [1]
Pleasant View 1	2003	Septic Tank	2	2003	4	PVC	1,880	Northwest
East Missoula	2002	8	2	2002	6	PVC	1,420	East Missoula
Third St.	2008	8	2	2008	4 & 6	PVC	6,490	South Central
Otis St.	2002	6	2	2002	4	PVC	840	Northeast
Mullan Rd.	2004	8	2	2004	10	PVC	1,730	Northwest
Kelly Island	2004	8	2	2004	12	PVC	13,650	Northwest
Council Way	2004	6	2	2004	4	PVC	2,100	Northwest
Pleasant View 2	2003	Septic Tank	2	2003	4	PVC	260	Northwest
Kona Ranch	2005	6	2	2004	4	PVC	3,310	Northwest
Futurity	2008	8	2	2007	12	PVC	8,480	North Central
Mastad	2008	8	2	2008	8	PVC	3,210	North Central
Waldo	2011	8	2	2011	8	PVC	5,370	North Central
Canyon River	2008	6	2	2008	4	PVC	680	East Missoula
Railroad	2009	Septic Tank	2	2009	2	HDPE	130	North Central
University	2007	7	2	NA	6	DIP	790	Southeast
Linda Vista Golf Course [2]	2011	8	4	2011	12 & 14	PVC	6,810	South Central
East Broadway	2014	8	2	1973	10	DIP	340	Northeast
Lower Miller Cr. [3]	2015	8	2	2015	10	PVC	2,640	Southwest
Travois Village [4]	1971	6	2	1971	6	ACP	1,240	Northeast
Big Sky	1978	6	2	1978	4	PVC	3,400	South Central
Reserve St.	1991	10	3	1991	20	PVC	7,680	South Central
Linda Vista [5]	1988	Septic Tank	2	1992	4	PVC	2,730	South Central [1]
Lamoreaux	1993	Septic Tank	2	1993	2	PVC	280	South Central [1]
Highwood	1996	Septic Tank	2	1996	2	PVC	40	South Central [1]
Dorothy Ct.	1996	Septic Tank	2	1996	2	PVC	310	Couth Central
Kennedy	1997	Septic Tank	2	1992	2	PVC	20	North Central
Leo	1997	Septic Tank	2	1997	2	PVC	60	North Central
Industry St.	1997	Septic Tank	2	1998	2	PVC	30	North Central
Maloney Ranch	2003	Septic Tank	2	1997	3	PVC	150	South Central [1]
Community Hospital	1978	8	2	1978	8	PVC	1,530	South Central
Momont No. 1	2018	6	2	1980	12	PVC	230	North Central
Momont No. 2	1980	6	2	1980	10	DIP	2,600	North Central
VFW Trailer Ct.	1995	unknown	2	1995	4	PVC	110	North Central

[1] Lift station discharges to STEP pressurized collection system that discharges to south interceptor through the South Central drainage basin.
[2] aka Lower Miller Creek No. 2 lift station
[3] aka Linda Vista 14 lift station
[4] Travois Village lift station is a private facility that discharges to the City collection system, and was included in the model analyses due to the relative size and area served.
[5] aka Jack Drive lift station

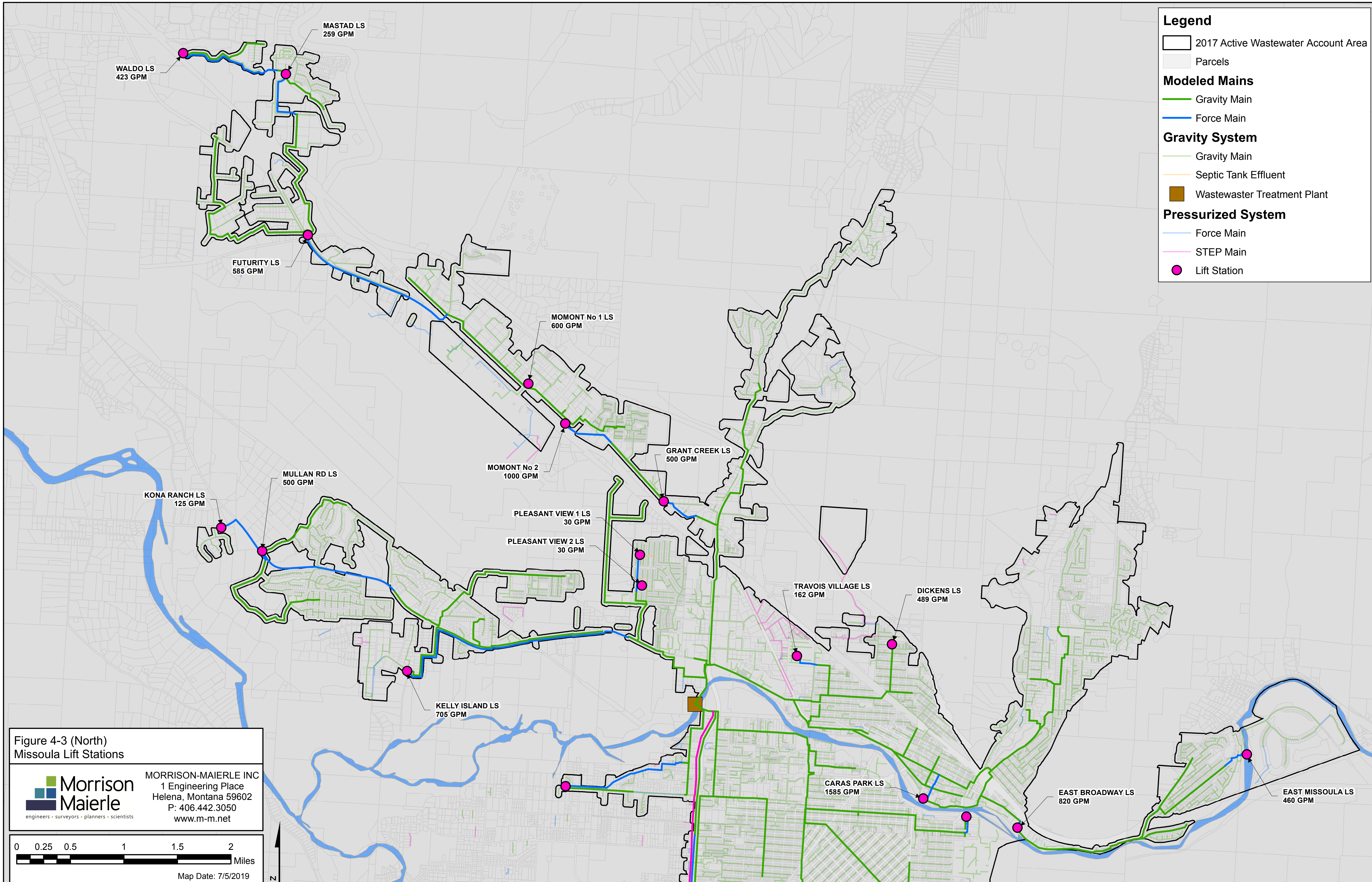
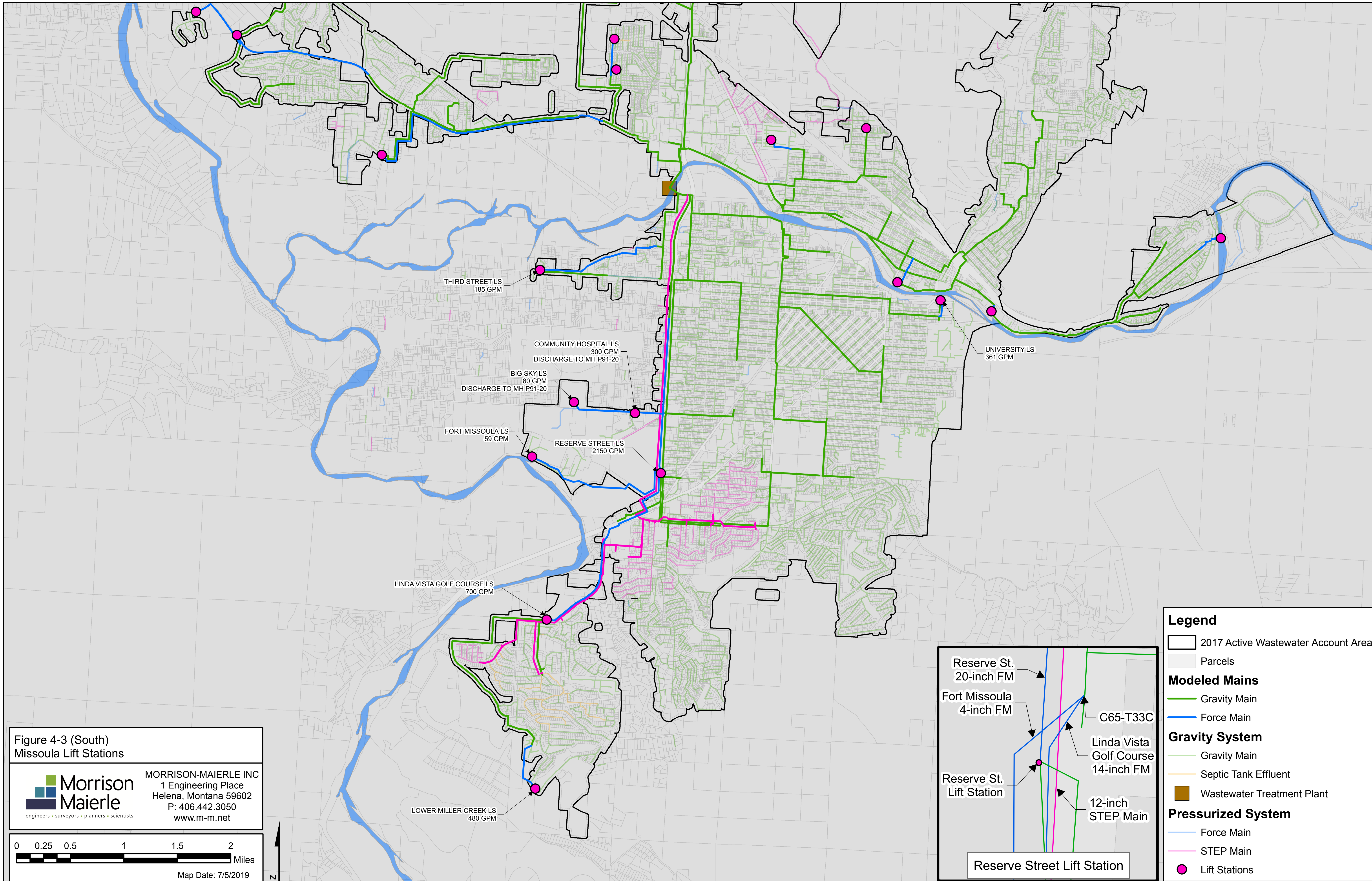


Figure 4-3 (North)
Missoula Lift Stations



THIRD STREET LS
185 GPM
COMMUNITY HOSPITAL LS
300 GPM
DISCHARGE TO MH P91-20
BIG SKY LS
80 GPM
DISCHARGE TO MH P91-20
FORT MISSOULA LS
59 GPM
RESERVE STREET LS
2150 GPM
UNIVERSITY LS
361 GPM
LINDA VISTA GOLF COURSE LS
700 GPM
LOWER MILLER CREEK LS
480 GPM
Reserve St.
20-inch FM
Fort Missoula
4-inch FM
C65-T33C
Linda Vista
Golf Course
14-inch FM
Reserve St.
Lift Station
12-inch
STEP Main
Reserve Street Lift Station
Legend
2017 Active Wastewater Account Area
Parcels
Modeled Mains
Gravity Main
Force Main
Gravity System
Gravity Main
Septic Tank Effluent
Wastewater Treatment Plant
Pressurized System
Force Main
STEP Main
Lift Stations
Figure 4-3 (South)
Missoula Lift Stations
Morrison
Maierle
engineers • surveyors • planners • scientists
MORRISON-MAIERLE INC
1 Engineering Place
Helena, Montana 59602
P: 406.442.3050
www.m-m.net
0 0.25 0.5 1 1.5 2
Miles
Map Date: 7/5/2019
N

4.3.1. Known Lift Station Issues

Hospital Lift Station No. 1

Blockages from rags and other fibrous refuse in lift station P98-17-LS, “Hospital Lift Station No. 1” and the connected gravity mains have been reported on a regular basis. This lift station serves an area of approximately 50 acres generally bound by South Avenue W. and Fort Missoula Road. The rags and other fibrous material are suspected to originate at the adjacent Community Medical Center. Collection system maintenance personnel have responded by replacing the pumps with chopper pumps to better manage the fibrous materials.

4.4. SEPTIC TANK EFFLUENT PUMP (STEP) SYSTEMS

Currently, the City owns and maintains approximately 1,442 active septic tank effluent pump (STEP) systems, with approximately 150 dry laid STEP systems owned and maintained by private entities as shown on Figure 4-1. The systems were first constructed in 1991 to 1993 to serve the Wapikiya, Bellevue, and Cold Springs areas, which had relatively flat topography coupled with extensive existing utilities and roadway infrastructure. Implementation of the STEP systems involved installation of an effluent pump and discharge pipe in each individual septic tank. The effluent pump typically installed by the City of Missoula collection system staff is a 0.25-Hp pump with maximum flow rating of 10-gpm. Individual STEP systems discharge to a pressurized main that is often networked to collect numerous STEP systems. The pressurized effluent piping accommodated the topographical and existing utility challenges, since the mains did not have to be laid at controlled grades for gravity conveyance, and the diameters could be smaller than gravity mains.

STEP systems typically began as existing septic tanks associated with parcels or residences that treat and discharge the wastewater on premise. These parcels or residences were not receiving wastewater service from the City. The treatment provided by septic tanks is limited to solids settling and anaerobic processes. By installation of the STEP, the wastewater generated by the associated parcels or residences still undergoes solids settling and anaerobic processes in the septic tank before it is conveyed to the City collection system and wastewater treatment plant.

The solids settling of the STEP system is beneficial to the overall collection system and wastewater treatment plant. However, periodic septic tank maintenance to remove accumulated solids is required. The anaerobic process in the STEP systems also continues, providing a benefit to the wastewater treatment plant. However, a by-product of the anaerobic process is formation of hydrogen sulfide gas which is detrimental to cementitious materials in the collection system such as manholes and concrete gravity mains.

4.4.1. Community Tank Systems

Community tank systems were typically installed to treat wastewater from multiple residences within a subdivision. These systems originally treated and discharged wastewater on-site as currently regulated by Circular DEQ-4 (Montana Dept. of Env. Quality, 2013). Typical installations in the City of Missoula included a conveyance network of gravity mains and manholes from the residences to the community treatment system.

Connection of the on-site community treatment systems to the City of Missoula collection system followed a similar process as individual STEP. Currently, the City manages 13 active community tank systems. Community tank sizes typically range from 3,000 to 5000 gallons. Some of community tank systems discharge to the southern STEP collection system without the use of effluent pumps. The upper reaches of the southern STEP collection system employ a gravity main network of predominantly 8-inch mains that allow the tanks to overflow into the collection system. Community tank systems with effluent pumps that discharge to the southern STEP collection system are included in Table 4-4 with capacities ranging from 10 to 75 gpm.

4.4.2. Southern STEP Collection System

The southern STEP collection system is a predominantly pressurized collection system in the southern portion of the City. This southern STEP collection system is shown in Figure 4-1 (South) and collects all STEP systems south of Brooks Street with isolated systems collected immediately adjacent to Reserve Street. It collects flows from individual STEP systems and community tank systems both pumped and by gravity flow, in addition to gravity main flows from the upper reaches of the collection system. Gravity mains that convey septic tank effluent are classified as septic tank effluent gravity (STEG) mains and only exist in the upper most reaches of the collection system where elevations of the STEG mains are higher than the surcharge level in the otherwise pressurized STEP collection mains. STEG mains generally discharge to lift stations and, in one instance, connect directly to the pressurized system. The demarcation between pressurized STEP mains and gravity flow STEG mains approximates the maximum surcharge elevation.

Figure 4-1 depicts the active southern STEP collection system components including individual STEP systems, pumped community tanks also classified as lift stations, and STEG gravity mains. The surrounding gravity collection system and associated lift stations and force mains that are not hydraulically connected to the southern STEP collection system are also shown. At manhole P91-1-I, the southern STEP collection system discharges to the 36-inch interceptor main that crosses the Clark Fork River immediately east of the WWTP. The diameter of the primary STEP collection system main is 16-inch at the interceptor discharge location. Additional isolated STEP discharges exist to the gravity mains in 39th Street and 24th Avenue.

The southern STEP collection system was modeled as a pressurized collection network. The model includes the primary STEP system discharge to the 36-inch interceptor as well as the isolated discharges to other gravity mains to simulate current system functionality.

4.5. COLLECTION SYSTEM MAINTENANCE

The City of Missoula conducts a comprehensive collection system maintenance and repair program. Collection system staff conduct annual maintenance tasks including root cutting, jetting, closed circuit TV surveying, and STEP system maintenance such as system pumping for removal of settled solids and repairs. Table 4-5 presents the maintenance statistics that have been compiled and recorded by collection system staff.

Table 4-5: Summary of City Collection System Maintenance Activities

City Maintenance Activity	Date Initiated	Annual Average Length 2012 - 2016	Percent of Total System
Sewer Main Cleaning[1]	1997	210 mi	64
Root Cutting	2006	113 mi	34
Jetting	1997	98 mi	30
Television Inspection of Sewer Mains	2001	57 mi	18
STEP System Pumping[2]	2004	104	7
Manhole Repair[3]	ongoing	83	1
Sewer Main Repair[4]	ongoing	1,700 ft	--

[1] Sewer Main Cleaning includes root cutting and jetting.
[2] The total number of STEP systems pumped between 2004 and 2016 was 1,524, which represents 107% of the currently active STEP systems in the collection system. This indicates that the existing STEP systems require pumping maintenance on a 12-year cycle.
[3] Repairs include complete replacement or component replacement with new risers, grade rings, lids, etc.
[4] Sewer main repair varies from patching to replacing pipe.

THE GARDEN CITY
MISSOULA
HUB OF FIVE VALLEYS

WASTEWATER FACILITY PLAN

CHAPTER 5 - EXISTING AND NEAR-TERM COLLECTION SYSTEM ANALYSIS

Morrison Maierle
engineers • surveyors • planners • scientists

CHAPTER 5 EXISTING AND NEAR-TERM COLLECTION SYSTEM ANALYSIS

5.1. EXISTING COLLECTION SYSTEM MODEL DEVELOPMENT

5.1.1. Introduction

The City did not have a hydraulic model of the existing collection system for use in analyzing the impacts of potential new developments and future growth on system capacity and identifying improvements needed to accommodate this growth. As part of this project, a steady state hydraulic model was developed. The software used for the model was InfoSewer by Innovyze, which runs within the ESRI ArcGIS environment and allows use of the City's GIS data.

This chapter describes the development of the steady state model including calibration and assumptions and presents the evaluation of the existing and near-term conditions. The existing population planning values associated with residential and nonresidential population and growth presented in Chapter 2 were used as the basis of current population distribution throughout the study area. The City's GIS data relevant to the collection system was refined and applied as presented in Chapter 3 to generate the resultant flow loading of the hydraulic model for existing and near-term conditions.

5.1.2. System Components Included in Model

The collection system model is comprised of a skeletonized system composed of the primarily 12-inch or larger gravity mains and associated manholes summarized in Table 4-2. The model also includes 24 lift stations and associated force mains listed in Table 5-1, the southern STEP system, and other periphery STEP systems of appreciable size. The lift stations were selected based on capacity, size of area served, and location relative to trunk mains.

5.1.3. Existing Collection System Model

Existing Conditions Average Day Flow Allocation / Loading

Model loading for the average day existing system analysis allocated a total of 7.27 mgd of combined wastewater and infiltration to the skeletonized collection system network. Allocation of the residential and non-residential loads across the 2017 Active Wastewater Account Area was accomplished by applying a fixed wastewater ratio of winter water meter usage to individual parcels in the account area that totaled 5.61 mgd. Winter water meter data was utilized to exclude water usage associated with irrigation.

Loading of individual manholes comprising the skeletonized system utilized Thiessen polygon methodology. The existing water meters accompanied by sewer accounts that occur within individual Thiessen polygons had a fixed wastewater ratio applied. Existing water meters that are not accompanied by sewer accounts were omitted. Existing sewer accounts not accompanied by water meters applied a fixed wastewater ratio of estimated non-metered winter water usage. The existing condition average day infiltration value of 1.66 mgd was distributed uniformly to the active wastewater account area within each Theissen polygon based on the calculated average infiltration by area of 97 gal/day-acre.

Existing Conditions Maximum Day with Peak Diurnal Factor Flow Allocation / Loading

Model loading for the maximum day existing system analysis uses the maximum day flow established in Chapter 2 and incorporates a diurnal peak factor. In addition, the model assumes that all lift stations and STEP systems are running simultaneously. These assumptions result in a total modeled flow that is higher than the maximum day flow established in Chapter 2. This approach ensures that all sub-basins in the system are modeled at peak conditions. The peak diurnal factor was not applied to infiltration loads which were distributed as described for the average day loading. The total maximum day infiltration hydraulic loads were distributed uniformly to the active wastewater account area based upon the calculated maximum day infiltration of 150 gal/day/acre.

Southern STEP Collection System Flow Allocation / Loading

The average day and maximum day loading methodology described above was superseded in Theissen polygons defined for STEP system communities. Loading of the STEP collection system replaced residential, non-residential, and infiltration loads based on individual STEP pump operational conditions that would occur after a City-wide power outage. During a power outage, individual STEP systems and community tanks with pumped discharges/lift stations retain flow. When electrical service is restored, all pumped systems will activate to draw down the respective tank levels. This results in the plausible maximum flow condition within the southern STEP collection system.

Based on previous studies of the southern STEP collection system, individual STEP systems were assigned a discharge of 2.1 gpm assuming all pumps discharging to the system concurrently. Community tanks with pumped discharges classified as lift stations were assigned the discharge capacity values listed in Table 5-1. While the frequency of pump starts may vary, the pump flow rates are fixed by constant pump capacities and are not peaked. By contrast, the loading of areas served by STEG mains, which collect flow via gravity without pumped interruption, can vary due to peaks. Therefore, loading of areas served by the STEG mains was identical to the loading distribution methodology utilized for all other areas of the collection system. Figure 5-1 depicts the active southern STEP collection system components and allocated loads for individual STEP systems and pumped community tanks.

5.1.4. Model Validation

Validation is intended to check the accuracy of representation to the actual collection system. It is a substantiation that the model provides a satisfactory range of accuracy consistent with the intended application as a master planning tool. To accomplish this process, recent collection system flow monitoring reports completed for the City were referenced and compared with the resulting model flows with wastewater loads allocated. The collection system flow monitoring results were compared to the modeled existing conditions flows, and are summarized in Table 5-2. Flow monitoring locations are also shown on Figure 4-2.

Table 5-1: Summary of Lift Stations Included in the Model

Lift Station Name	Ownership	Year Lift Station Installed	Wet Well Dia. (ft)	No. of Pumps	Year Force Main Installed	Force Main Dia. (in)	Force Main Material (in)	Force Main Length (ft)	Discharge Trunk / Drainage	Modeled Capacity (gpm)	Capacity Reference
Grant Creek	City	1980	8	2	1980	10	DIP	1,910	North Central	500	Maintenance records pump design point
Caras Park	City	2016	10	2	2016	14	PVC	1,580	Northeast	1,585	Measured SCADA 2018
Fort Missoula	City	NA	6	2	1997	4	VCP	8,250	South Central	59	Maintenance records pump design point
Dickens St.	City	1973	5	2	1973	4	DIP	30	Northeast	489	Measured SCADA 2018
Pleasant View 1	City	2003	Septic Tank	2	2003	4	PVC	1,880	Northwest	30	Record drawing pump model
East Missoula	City	2002	8	2	2002	6	PVC	1,420	East Missoula	460	Measured SCADA 2018
Third St.	City	2008	8	2	2008	4 & 6	PVC	6,490	South Central	185	Measured SCADA 2018
Mullan Rd.	City	2004	8	2	2004	10	PVC	1,730	Northwest	500	Record drawing pump design point
Kelly Island	City	2004	8	2	2004	12	PVC	13,650	Northwest	705	Record drawing pump design point
Pleasant View 2	City	2003	Septic Tank	2	2003	4	PVC	260	Northwest	30	Record drawing pump model
Kona Ranch	City	2005	6	2	2004	4	PVC	3,310	Northwest	125	Maintenance records pump design point
Futurity	City	2008	8	2	2007	12	PVC	8,480	North Central	585	Measured pump performance 2019
Mastad	City	2008	8	2	2008	8	PVC	3,210	North Central	259	Measured pump performance 2019
Waldo	City	2011	8	2	2011	8	PVC	5,370	North Central	423	Measured pump performance 2019
University	City	2007	7	2	NA	6	DIP	790	Southeast	361	Measured SCADA 2018, 452-gpm @ 70-ft TDH recommended
Linda Vista Golf Course [1]	City	2011	8	4	2011	12 & 14	PVC	6,810	South Central	700	Measured SCADA 2018
East Broadway	City	2014	8	2	1973	10	DIP	340	Northeast	820	Measured SCADA 2018
Lower Miller Creek [2]	City	2015	8	2	2015	10	PVC	2,640	Southwest	480	Measured SCADA 2018
Travois Village [3]	Private	1971	6	2	1971	6	ACP	1,240	Northeast	162	Record drawing pump Hp and TDH
Big Sky	City	1978	6	2	1978	4	PVC	3,400	South Central	80	Maintenance records pump design point
Reserve St.	City	1991	10	3	1991	20	PVC	7,680	South Central	2,150	Measured pump performance 2018
Community Hospital	City	1978	8	2	1978	8	PVC	1,530	South Central	300	Measured SCADA 2018
Momont No. 1	City	2018	6	2	1980	12	PVC	230	North Central	600	Construction commissioning flow test, VFD capable of 1200 gpm
Momont No. 2	City	1980	6	2	1980	10	DIP	2,600	North Central	1000	Maintenance records pump design point

[1] aka Lower Miller Creek No. 2 lift station

[2] aka Linda Vista 14 lift station

[3] Travois Village lift station is a private facility that discharges to the City collection system, and was included in the model analyses due to the relative size and area served.

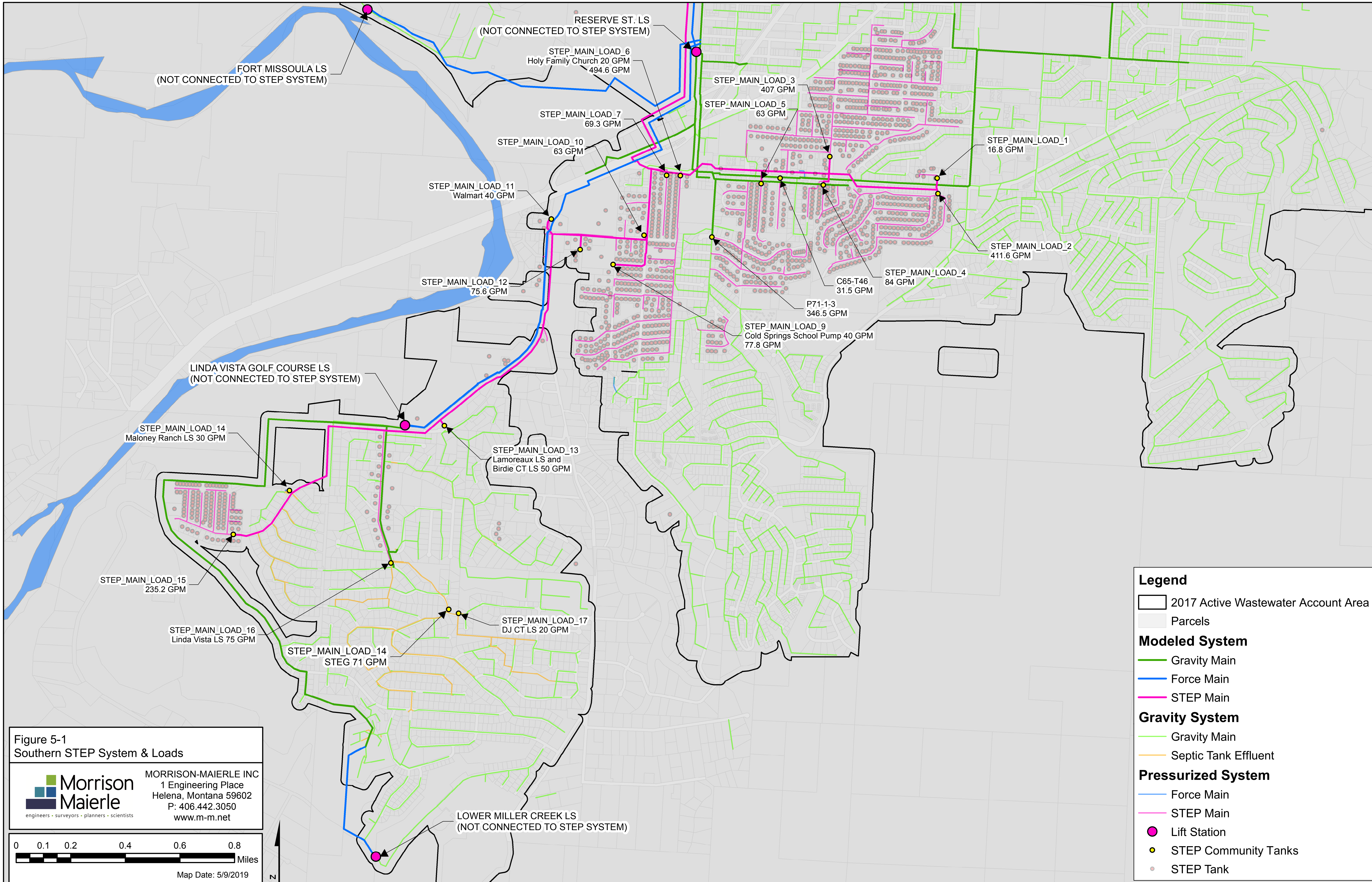


Figure 5-1
Southern STEP System & Loads

Table 5-2: Summary of Collection System Flow Monitoring Locations

Monitoring Location	Monitoring Date	Monitored Maximum Flow (gpm)	Modeled Maximum Flow (gpm)	Summary
Momont No. 1 lift station influent	June 2017	615	815	Difference associated with gravity main attenuation not captured by the model and operational status of Futurity lift station discharge
Existing 36-inch main at manhole C61-22 in River Rd.	June 2012	1,434	2,414	Difference associated with growth over 5 years since flow monitoring, monitored June flows in Southeast basin are lower with University of Montana summer population, and flow monitoring conducted for a single day. Attenuation of University lift station discharge is not captured by the model.
Manhole P04-2-2 Near North Russell St. and Cedar St.	June 2017	2,479	2,430	Difference associated with gravity main attenuation not captured by the model and operational status of Caras Park lift station discharge
East Broadway lift station influent (flow study)	Dec. 2012-Jan. 2013	188	705	Difference associated with growth over 4 years since flow monitoring completed, and monitoring occurred during typical low flow periods of December and January.
East Broadway lift station influent (SCADA data)	July-Aug 2018	236	705	Maximum day flows correlate within 5% without upstream East Missoula lift station operating at 460 gpm. Attenuation in 2.4 miles of gravity main is not captured by the model.
Manhole 458-14 in Reserve St.	June 2016	796	887	Difference associated with gravity main attenuation not captured by the model and operational status of Grant Creek lift station discharge.

The comparison of loading methodology used in the collection system model to the recent flow monitoring demonstrated a practical and balanced distribution of model loads in areas conveying flows collected predominantly by gravity without interruption by lift stations. The higher percent differences in monitored vs. modeled flows occur in areas conveying flows with upstream lift stations, in addition to the summary comments provided above. In these areas, the percent differences are primarily due to the assumption that all lift stations and STEP systems are running, and flow attenuation of the pumped flows is not accounted for in the model results. Flow attenuation in a modeled sewer collection system is a process that reduces the peak flow rate by redistributing the pumped volume over a period of time accounting for internal storage and diffusion in the gravity mains. Calculation of flow attenuation is not possible with a steady-state model. A dynamic model would account for flow attenuation but would require input of additional critical wet well operational variables to accompany extended period simulations.

Further refinement of the collection system model loads is recommended with more comprehensive flow monitoring data. Longer term flow monitoring over a period of months provides data more likely to capture maximum flows and establish reliable average flows. Flow monitoring of multiple primary basin trunk mains concurrently provides data that can be utilized for loading calibration refinements specific to individual basins. Along with more robust flow monitoring data, critical lift station wet well operational data would need to be available to perform extended period simulations that account for flow attenuation.

5.1.5. Summary of Model Flows

A summary of the major drainage basin flows and southern STEP system flows is provided in Table 5-3. The flows listed in the table are the calculated results of the load distribution assuming all lift stations and STEP pumps are operating as may be experienced for a short time after a city-wide power outage. Alternative scenarios with only the largest lift stations discharging to particular locations were run with very similar results. Therefore, the more conservative approach of having all lift stations operating was retained for all scenarios.

Table 5-3 demonstrates a key point regarding flow distribution, conveyance, and operations within the collection system. All of the major drainage basin flows have a calculated average day to maximum day difference that is less than the combined peaking factors due to the interruption of gravity flows by the numerous lift stations throughout the collection system. The portion of the existing active wastewater account service area that drains to the WWTP exclusively by gravity flow is approximately 45 percent. The remaining 55 percent of the existing service area is intercepted by lift stations with flows discharged downstream at a fixed rate, with variable frequency of pumps on or off.

The major drainage basin average day to maximum day flow difference indicates the level of influence that lift stations have on individual basins. For example, the Southeast Basin conveys flow primarily by gravity in a relatively large basin, with only the University lift station within the basin. Therefore, the average day to maximum day flow difference is higher than other basins.

In contrast, the southern STEP system has a relatively small average day to maximum day flow change. This is due to the majority of the southern STEP system flows being pumped, with a relatively small portion of gravity flows from STEG areas that also drain to the southern STEP system. The pumped flows are not influenced by average to maximum day peaking factors, while only the relatively small STEG areas draining to the southern STEP main are influenced by the average to maximum day peaking factors.

Table 5-3: Summary of Model Loading Flows for Current Conditions

Trunk Main / Major Drainage Basin	Discharge	Average Day Flow [1] (gpm)	Maximum Day Flow [1] (gpm)	Notes
North Central	North Interceptor MH P01-6-12	903	1,209	
Northwest	North Interceptor MH P01-6-8	872	992	
Northeast	North Interceptor MH P01-6-10	3,008	3,661	
South Central	South Interceptor [2] 27-inch @ MH C61-6 [3] 30-inch @ MH P09-48-1 [4] 36-inch @ MH P98-38-1	1,719 2,154 192	2,409 2,156 198	
Southeast	South Interceptor MH C61-6	1,324	2,453	
Southern STEP System 16-inch Main Discharge	South Interceptor MH P09-48-2	2,255	2,301	
Southwest	South Central Trunk / Drainage Basin [5] MH C65-T33C	700	700	Flow is conveyed through South Central Major Drainage Basin
East Missoula	Northeast Trunk / Drainage Basin [6] MH 376-9A	820	820	Flow is conveyed through Northeast Major Drainage Basin

[1] Note that a diurnal flow factor was applied to these flows and all lift stations and STEP systems are assumed running simultaneously, resulting in a total greater than maximum day flows shown in Chapter 2.
[2] 27-inch trunk main conveys S. Reserve St. gravity main collection and Linda Vista Golf Course lift station flows.
[3] 30-inch short length trunk main primarily conveying S. Reserve St. lift station flows.
[4] 36-inch short length trunk main primarily conveying flows immediately south and east of WWTP.
[5] MH C65-T33C within South Central drainage basin receives flow from Linda Vista Golf Course Lift Station.
[6] MH 376-9A within Northeast drainage basin receives flow from East Broadway Lift Station.

5.2. EXISTING COLLECTION SYSTEM ANALYSIS

The existing collection system model analysis showed that some capacity deficiencies currently exist and are due to various issues such as undersized mains or a bottleneck in diameter, inadequate or adverse sloped main segments, and lift station pump capacities versus the receiving gravity main sizes. This section details the model-predicted deficiencies within the existing collection system.

5.2.1. Evaluation Criteria

Evaluation of the existing collection system was performed to quantify the maximum potential flows within the system and identify capacity deficiencies. To accomplish this evaluation, model analysis assumed that each lift station within the system was operating, resulting in the maximum potential downstream flows. The model output was evaluated based on the following parameters:

- q/Q - ratio of modeled flow within a gravity main segment to the theoretical open channel flow rate. The q/Q parameter indicates the capacity utilized in a gravity main segment. q/Q ratios were categorized by thresholds of 0.50 indicating 50 percent full, and 0.75 indicating 75 percent full. 0.75 q/Q is a typical maximum capacity for design purposes to ensure that the gravity main segments are adequately vented to ensure open channel flow conditions. Exceeding the 0.75 q/Q capacity threshold indicates gravity mains within the system that could require revision or replacement due to inadequate slope or diameter to convey predicted flows.

- Lift Station In/Out - ratio of modeled lift station influent flow to the single pump operating discharge capacity. This parameter is used to quantify the overall capacity of a lift station regardless of the wet well diameter and potential storage volume. A threshold ratio of 0.80 indicates that the lift station can adequately convey the influent flow with an additional 20 percent reserve capacity. Lift stations that exceed the 0.80 ratio may operate by utilizing reserve capacity and are either at risk or not adequate to convey the influent flows.

5.2.2. Evaluation Results – Gravity Mains

The following sections summarize the model results for the existing average day loading condition and maximum day both simulated with all modeled lift stations and STEP system pumps operating.

Table 5-4 summarizes the gravity mains which have existing flows that exceed 75 percent of total capacity (q/Q > 0.75) for each flow scenario. All capacity issues that occur during average day conditions also occur during maximum day conditions. Therefore, the gravity main segments that are predicted to have capacity deficiencies during average day conditions, also have maximum day with peak diurnal factor capacity deficiencies quantified. Main segments that only experience capacity deficiencies associated with the maximum day flow conditions do not have average day flows and capacities tabulated.

Table 5-4: Summary of Modeled System Capacity Deficiencies in Gravity Mains under Existing Conditions

Location	Gravity Main Segment Facility ID	Average Day Flow (gpm)	Average Day Capacity (q/Q)	Average Day Surcharge Depth (ft)	Maximum Day Flow (gpm)	Maximum Day Capacity (q/Q)	Maximum Day Surcharge Depth (ft)	Full Flow (gpm)	Slope (ft/ft)	Diameter (in)	Length (ft)	Cause Summary
Upstream of Caras Park LS	C60-6C84-2-B1	862	0.751		1,154	1.006	0.012	1,147	0.00156	15	439	
	C60-8C60-7	852	9.267	0.095	1,132	12.311	0.274	92	0.00000	15	110	
	C60-7C60-6	853	0.881		1,134	1.171	0.106	968	0.00111	15	253	
	C60-11C60-10	851	0.861		1,130	1.144	0.322	988	0.00115	15	353	
	C60-12C60-11	847	0.886		1,120	1.172	0.408	956	0.00108	15	451	Inadequate diameter
	C60-13C60-12	846	0.766		1,120	1.014	0.408	1,105	0.00145	15	235	
	C60-10C60-9	851	0.899		1,130	1.194	0.293	946	0.00106	15	244	
	C60-9C60-8	851	0.758		1,130	1.006	0.281	1,124	0.00149	15	246	
	376-1286-2				1,097	0.900		1,220	0.00176	15	297	
Reserve St. between Brooks St. and River Rd.	C65-T28AC65-T28	778	0.864		798	0.886		900	0.00016	21	170	Inadequate slope
	C65-T27C65-T26A	779	0.866		801	0.890		900	0.00016	21	157	Inadequate slope
	C65-T23C65-T22A	1,310.00	2.137	0.054	1,513	2.468	0.077	613	0.00007	21	203	Inadequate slope
	C65-T21C65-T20	1,333.00	1.679	0.070	1,564	1.970	0.114	794	0.00012	21	307	Inadequate slope
	C65-T21AC65-T20	1,333.00	0.934		1,564	1.096	0.003	1,427	0.00040	21	40	Inadequate slope
	C65-T19AC65-T18	1,370.00	0.800		1,646	0.961		1,712	0.00058	21	191	Inadequate slope
	C65-T15C65-T14	1,420.00	1.500	0.077	1,755	1.854	0.150	947	0.00018	21	346	Inadequate slope
	C65-T13C65-T12	1,425.00	0.763		1,766	0.946		1,866	0.00069	21	267	Inadequate slope
	C65-T12C65-T11A				1,766	0.817		2,162	0.00092	21	233	Inadequate slope
	C65-T11C65-T10				1,826	0.873		2,093	0.00086	21	180	Inadequate slope
	C65-T10C65-T9				1,826	0.764		2,391	0.00113	21	288	Inadequate slope
	C65-T9C65-T8A	1,453.00	0.901		1,827	1.134	0.027	1,612	0.00051	21	180	Inadequate slope
	C65-T5C65-T4				1,855	0.949		1,955	0.00075	21	375	Inadequate slope
	C65-T4C65-T3				1,860	0.821		2,266	0.00101	21	315	Inadequate slope
	C65-T3C65-T2	1,470.00	0.754		1,864	0.956		1,951	0.00075	21	175	Inadequate slope
Downstream of Dickens St. lift station	371-A334-EX	503	0.815		520	0.842		617	0.00148	12	403	
	371-3A371-2A	489	0.938		489	0.938		522	0.00280	10	284	Inadequate diameter
	371-2A371-1A	489	0.937		489	0.938		522	0.00280	10	282	
	371-1A371-A	489	0.938		489	0.937		522	0.00280	10	280	
Between Momont No. 1 LS & No. 2 lift stations	R369-24R369-23	600	0.757		600	0.757		793	0.00145	12	91	
	R369-15R369-14	630	0.833		661	0.874		757	0.00132	12	220	
	R369-16R369-15	630	0.834		661	0.875		755	0.00131	12	251	Inadequate diameter
	R369-17R369-16	621	0.761		644	0.789		816	0.00153	12	400	

Location	Gravity Main Segment Facility ID	Average Day Flow (gpm)	Average Day Capacity (q/Q)	Average Day Surcharge Depth (ft)	Maximum Day Flow (gpm)	Maximum Day Capacity (q/Q)	Maximum Day Surcharge Depth (ft)	Full Flow (gpm)	Slope (ft/ft)	Diameter (in)	Length (ft)	Cause Summary
Upstream of Momont No. 1 lift station	R369-25R369-LS1	696	2.146	0.145	815	2.512	0.213	324	0.00018	10	37	Inverted diameter transitions with Inadequate slope and diameter
	P68-1-12R369-25	685	2.118	0.210	792	2.452	0.316	323	0.00107	10	56	
	P68-1-10P68-1-12	684	1.040	0.209	792	1.203	0.739	658	0.00446	10	259	
	P68-1-9P68-1-10	684	1.270	0.887	792	1.407	2.106	539	0.00299	10	420	
	P68-1-8P68-1-9	684	1.737	2.141	791	2.010	4.045	394	0.00159	10	418	
	P68-1-5P68-1-8	681	1.201	2.744	787	1.386	5.404	568	0.00331	10	474	
	P68-1-3P68-1-5	681	2.267	2.744	786	2.615	5.404	300	0.00305	8	432	
	P97-4-2P97-4-1	680	2.614	2.912	784	3.014	5.613	260	0.00135	8	276	
42-in. interceptor	P01-6-10P01-6-9	3,274	1.109	0.002	4,037	1.368	0.007	2,952	0.00003	42	318	Inadequate slope
30-in. interceptor	P76-8-1C74-1-4	4,782	1.736	0.035	5,863	2.129	0.061	2,754	0.00013	30	129	Receive flows from 30, 36, and 42-inch mains, inadequate slope and diameter
	C74-1-4C74-1-3				5,863	0.880		6,661	0.00077	30	350	
36-in. interceptor	P09-48-2C61-3	7,453	0.987		9,322	1.235	0.081	7,550	0.00037	36	401	Receive 16-inch STEP main discharge and shallow slopes
	C61-2C61-1				9,521	0.886		10,741	0.00128	36	486	
Lower Miller Cr. Rd. & Linda Vista Blvd	P08-28-A21P08-28-A22	574	0.845		674	0.992		679	0.00032	15	316	Inadequate slope and diameter
Briggs St. & 24th Ave.	P71-1-3P71-1-2A	550	0.754		782	1.072	0.047	729	0.00207	12	145	Receive STEP main discharge, inadequate slope
Monroe St. flow split	R426-21R426-20	259	0.885		293	1.000	1.932	293	0.00000	21	72	Flat slope
1,900-ft upstream of E. Broadway LS	376-16P95-22-A1	549	0.754		651	0.895		727	0.00037	15	89	Inadequate slopes
	P95-22-A1376-15				682	0.775		880	0.00092	15	143	
Pattee Creek Dr. & Bancroft St.	AC60-37AC60-36				212	1.278	0.022	166	0.00011	12	317	Inadequate slope
Russell St. between Milwaukee Way & 6th St. S.W.	AC60-4AC60-3				1,024	0.978		1,048	0.00011	24	359	Inadequate slope
	N354-CN354-B				2,314	0.915		2,530	0.00019	30	112	
River Rd. between Hendricksen Dr. & Missys Way	526-OO1C61-17				2,432	0.951		2,558	0.00006	36	226	Inadequate slope
	P00-16-AP86-3-1				2,421	0.947		2,557	0.00007	36	110	
Liberty Ln. and Russell St.	P97-1-9P97-1-8				2,536	0.801		3,164	0.00007	36	213	Inadequate slope
Alley North of Philips St. between Byron St. & Burns St.	89-10889-109				973	0.779		1,249	0.00185	15	379	Inadequate slope and diameter
						Average Day Flow Conditions Total Deficient Length					10,717	
						Maximum Day Flow Conditions Total Deficient Length					14,803	
						Total Skeletonized Model Gravity Main Length					327,500	

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In addition to the main segments that exceed 75% of total capacity, the model also reported 14 gravity main segments with adverse slopes. Adverse sloped segments have an inlet invert elevation that is lower than the outlet invert elevations. The model software assumes that adverse slope mains are pressurized with a water depth to main diameter ratio (d/D) equal to 1.0, and does not calculate q/Q values. Therefore, adverse slope mains are reported separately in Table 5-5 and are identified on the average day and maximum day capacity figures.

Table 5-5: Adverse Gravity Main Segments

Location	Gravity Main Segment Facility ID	(d/D)	Slope (ft/ft)	Diameter (in)	Length (ft)
W. Broadway and Owen St.	C80-1B3-1-26	1.000	-0.02345	10	6
Between Mount Ave. & 14th St.	AC60-13281-201	1.000	-0.00443	24	27
W. Broadway and McCormick St.	R426-AP01-30-26	1.000	-0.00441	30	42
Davis St. and Wyoming St.	526-KK1C61-9	1.000	-0.00223	27	34
Reserve St. downstream of Linda Vista Golf Course Force Main	C65-T33CC65-T33	1.000	-0.00175	21	54
36" interceptor, immediately east of Clark Fork River crossing	P98-38-1C61-2	1.000	-0.00172	36	12
Russel St. between 2nd St. S.W. and 3rd St. S.W.	217-13E3B1-1-19	1.000	-0.00142	30	95
Reserve St. and South Ave. W.	C65-T25328-0	1.000	-0.00119	21	165
River Rd. between Hunton Ln. and Bondurant Ct.	P91-28-1C61-20	1.000	-0.00092	36	73
End of Hanks Dr.	456-2P78-9-10	1.000	-0.00041	15	117
Between Mount Ave. & 14th St.	281-200AC60-13	1.000	-0.00028	24	102
Reserve St. and 7th Ave. S.W.	C65-T7C65-T6	1.000	-0.00014	21	254
Reserve St. and Old Highway 93	C65-T37C65-T36	1.000	-0.00008	18	193
Reserve St. and Mary Ave.	C65-T30C65-T29	1.000	-0.00005	21	330
				Total Length	**1,504**

The gravity main segments reported with adverse slopes were calculated from the field survey of manholes. Most adverse mains have relatively short lengths between manholes with diameters of 21-inches or larger, which are typically installed with shallower slopes than are smaller diameter mains. In some instances, newer manholes were inserted into existing mains to accomplish a lateral connection. Such lateral connections can create adverse slopes particularly when new manholes with sloped channels are inserted within existing mains laid at relatively shallow slopes. Adverse slopes can also be a product of the precision inherent with the field survey and physical measurement of inverts, which is exacerbated by relatively short main lengths and large diameters laid at shallow slopes.

In most cases, a relatively short length of adverse sloped gravity main results in a decrease in pipe capacity and may require development of a hydraulic head or surcharge upstream to provide required capacity. While not a desirable condition, isolated instances are not uncommon in collection systems and may be acceptable if surcharge depths are minimal. Eliminating adverse slopes typically requires detailed engineering study to devise a replacement plan that can provide the required slope correction, and may involve lengthy and expensive main segment replacements. Flow monitoring and more detailed survey of identified adverse slopes is recommended to verify severity and impact on capacities.

5.2.3. Collection System Capacities for Average Day Flow Conditions

Table 5-4 shows that model analysis of the existing collection system during the average day loading condition shows relatively few capacity deficiencies. Figure 5-2 graphically demonstrates the average day gravity main capacities with break points at 50 percent and 75 percent. Overall, the existing system is adequately sized to convey the average day flow conditions with the exception of 41 gravity main segments with q/Q values that exceed 0.75. Most of these main segments are grouped together and are conveying flows without adequate diameter or slope. These groups are located within the 15-inch mains immediately upstream of the Caras Park lift station, within Reserve Street between Brooks Street and River Road, the 12-inch mains between Momont No. 1 and Momont No. 2 lift stations, the 8-inch and 10-inch mains immediately upstream of Momont No. 1 lift station, and 12-inch mains downstream of the Dickens Street lift station.

A small number of the 41 capacity deficient main segments are isolated single main segments. Two of these gravity main segments occur in the 30-inch and 42-inch interceptor main located within and adjacent to Clark Fork Road and another occurs in the 36-inch interceptor that crosses the Clark Fork River. Both are due to shallow slopes calculated from surveyed data.

Other single main segments that exceed q/Q of 0.75 typically occur after the intersection of multiple mains, or receive pumped discharge from lift stations or STEP mains.

Of the 41 capacity deficient main segments associated with the average day flow conditions, 15 mains are predicted to surcharge due to q/Q values in excess of 1.0. Surcharge depths are calculated at both ends of gravity main segments, with the higher of the two surcharge depths reported in Table 5-4. All of these main segments reported surcharge elevations less than 0.10 feet above the pipe crown with the exception of the main segments associated with the group upstream of Momont No. 1 lift station. These three main segments have surcharge levels that range from 0.15 feet to 2.91 feet and are due to a combination of diameter transition from 15-inch to 10- or 8-inch mains and shallow slopes. Mains that surcharge also put connected laterals and services in the vicinity at risk of surcharging.

The 16-inch Southern STEP collection main discharging to the 36-inch interceptor main that crosses the Clark Fork River immediately east of the WWTP at manhole P91-1-I reported an average day flow of 2,255 gpm, resulting in a q/Q = 0.99. Based on the loading assumption presented above, 2,213 gpm of the total flow is pumped and the remaining 42 gpm originate from flows collected and conveyed through STEG mains. Note that these flows conservatively assume that all STEP system pumps are operating simultaneously and do not account for flow attenuation throughout the system. If modeled without the flow contributed by the 16-inch STEP main, the q/Q value for this interceptor segment drops to 0.69, suggesting that actual conditions are likely near the design capacity of 75% pipe full or higher.

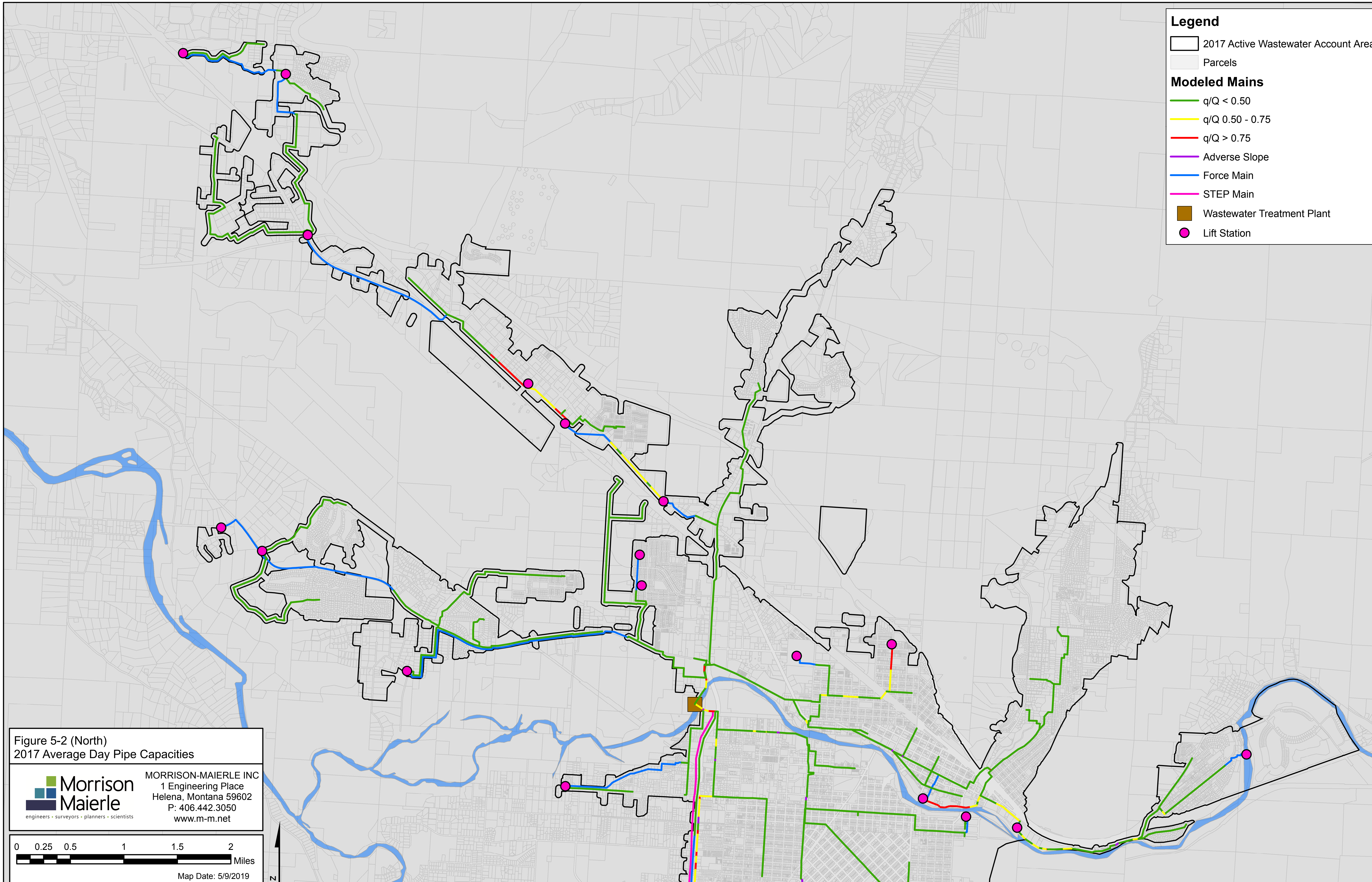

Figure 5-2 (North)
2017 Average Day Pipe Capacities

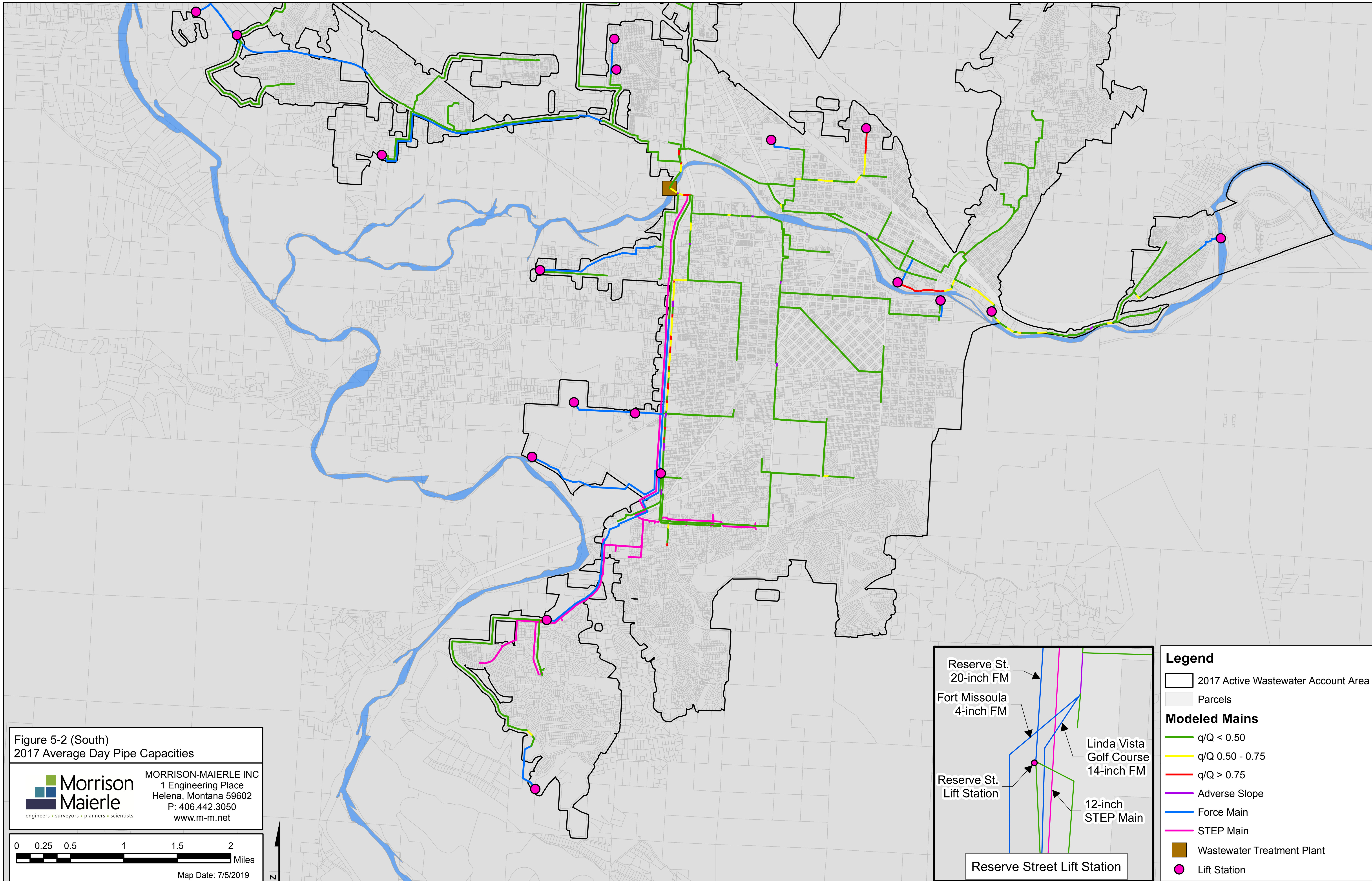

Figure 5-2 (South)
2017 Average Day Pipe Capacities

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5.2.4. Collection System Capacities for Maximum Day with Peak Diurnal Factor Flows

Similar to the average day loading conditions, analysis of the existing collection system maximum day conditions showed additional gravity main segments exceeding the criteria of q/Q > 0.75. Table 5-4 tabulates the additional main segments with capacity deficiencies associated with the maximum day flow conditions. Figure 5-3 graphically demonstrates the maximum day with peak diurnal factor gravity main capacities with break points at 50 percent and 75 percent. Main segments listed in Table 5-4 without values reported in the average day flow and capacity column do not exceed 75% full during average day flow conditions.

Overall, the existing system is also adequately sized to convey the maximum day with peak diurnal pattern flow conditions. The model reported 57 gravity main segments with q/Q values that exceed 0.75. These main segments include those reported for the average day flow conditions with 16 additional gravity main segments that are predicted to experience capacity deficiencies. The majority of the capacity deficient main segments are located in the same groups identified for the average day conditions. Some of these groups have additional main segments that experience deficiencies only during the maximum day conditions.

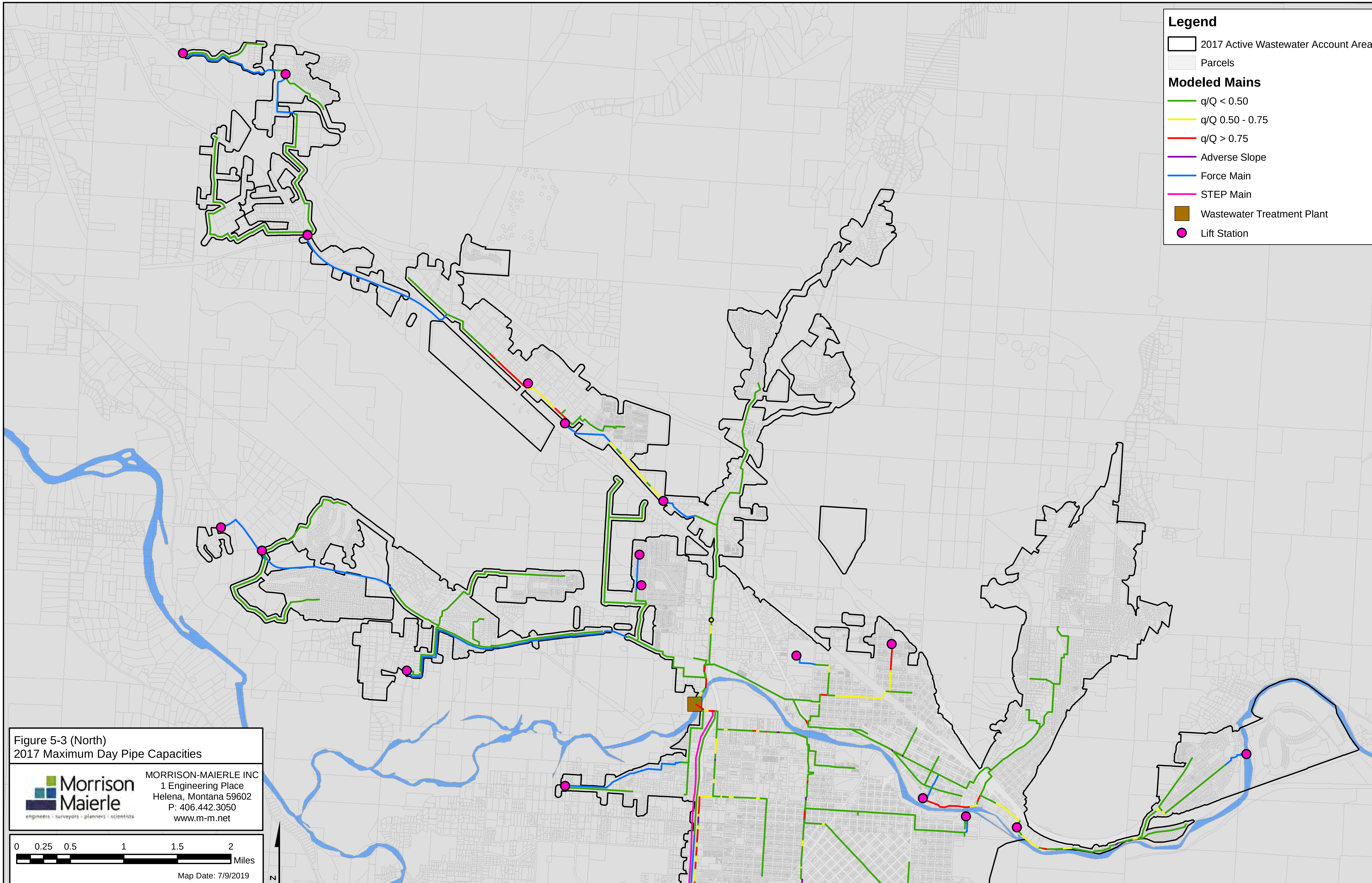
A relatively small number of the 16 additional main segments are isolated main segments in groupings of three or less. These isolated main segments are located at the intersection of Pattee Creek Drive and Bancroft Street, within Russell Street between Milwaukee Way and 6th Street S.W., within River Road between Hendricksen Drive and Missys Way, in the alley north of Philips Street between Byron Street and Burns Street, and at the intersection of Liberty Lane and Russell Street. All of these main segments have capacities less than 100% full with the exception of the 12-inch main at the intersection of Pattee Creek Drive and Bancroft Street. This 12-inch main capacity deficiency is primarily due to inadequate pipe slope.

Of the 57 capacity deficient main segments associated with the maximum day conditions, 27 mains are predicted to surcharge due to q/Q values in excess of 1.0. Similar to the average day flow conditions, the eight main segments associated with the group upstream of Momont No. 1 lift station report the highest surcharge levels ranging from 0.21 feet to 5.61 feet, again due to a combination of inverted diameter transition, inadequate diameter, and shallow slopes.

One additional main segment with a high surcharge elevation is associated with the flow split manhole R426-21 in Monroe Street between Vine Street and Poplar Street. During the average day flow conditions, wastewater is entirely conveyed through gravity main segment R426-21R426-20 which flows under Rattlesnake Creek. During the maximum day conditions, manhole R426-21 surcharges due to the flat slope of main segment R426-21R426-20. Due to the layout of the flow split manhole, a surcharge level of 1.76 feet above the manhole invert will allow wastewater to flow over a steel weir plate into gravity main segment R426-21C60-20 which is conveyed south to the Caras Park lift station. The surcharge level calculated in manhole R426-21 during maximum day flow conditions is 1.93 feet, with 259 gpm flowing across Rattlesnake Creek, and 243 gpm flowing to the Caras Park lift station.

The remaining 18 main segments that reported surcharge elevations during maximum day conditions are less than 0.41 feet above the pipe crown. The 16-inch Southern STEP collection main discharging to the

36-inch interceptor main reported a maximum day flow of 2,301 gpm with 2,213 gpm of the total flow pumped without peaking factors, and 88 gpm originating from STEG main flow which have peaking factors applied.


Legend
2017 Active Wastewater Account Area
Parcels
Modeled Mains
q/Q < 0.50
q/Q 0.50 - 0.75
q/Q > 0.75
Adverse Slope
Force Main
STEP Main
Wastewater Treatment Plant
Lift Station
Figure 5-3 (North)
2017 Maximum Day Pipe Capacities
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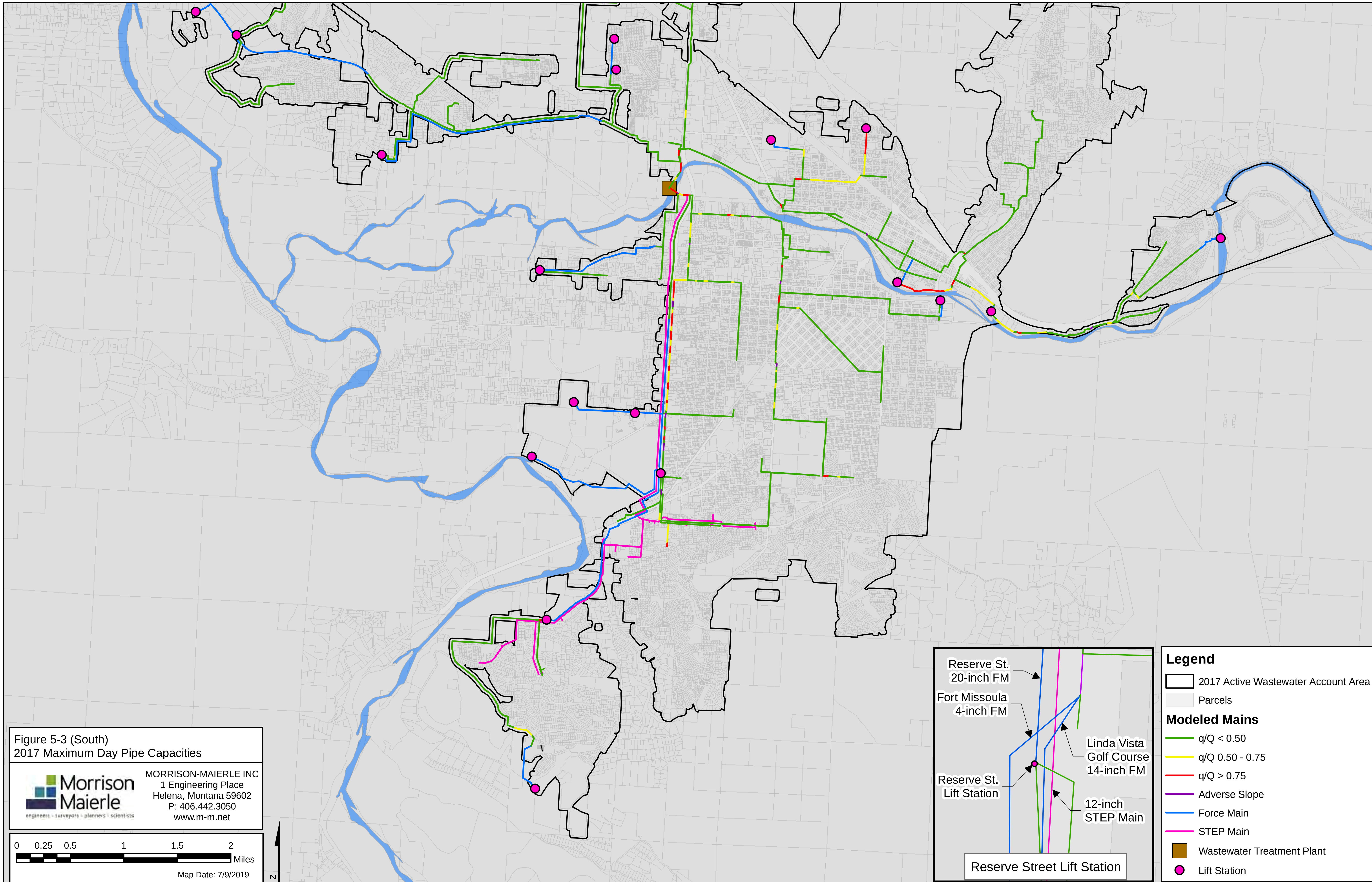
Reserve St.
20-inch FM
Fort Missoula
4-inch FM
Linda Vista
Golf Course
14-inch FM
Reserve St.
Lift Station
12-inch
STEP Main
Reserve Street Lift Station
Legend
2017 Active Wastewater Account Area
Parcels
Modeled Mains
q/Q < 0.50
q/Q 0.50 - 0.75
q/Q > 0.75
Adverse Slope
Force Main
STEP Main
Wastewater Treatment Plant
Lift Station
Figure 5-3 (South)
2017 Maximum Day Pipe Capacities
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Map Date: 7/9/2019

5.2.5. Gravity Main Existing Condition Evaluation Summary

Overall, the existing collection system is adequately sized to convey the average day and maximum day flows assuming all lift station and STEP system pumps are operating simultaneously. Of the total approximately 327,660 feet of gravity main modeled, the average and maximum flow scenarios result in 10,770 feet and 14,803 feet, respectively, of gravity main identified as capacity deficient using a q/Q > 0.75 threshold; 3,785 feet average day and 7,161 feet maximum day are identified to surcharge with q/Q values exceeding 1.0. Figure 5-4 shows gravity main capacities as percentage of the total mains that were modeled. Mains with q/Q ≥ 0.75 are considered deficient. None of the existing gravity main segments are predicted to surcharge higher than the adjacent manhole rims. However, these gravity main segments should be considered for field verification of capacity to adequately convey current flows, with more detailed study depending on field results.

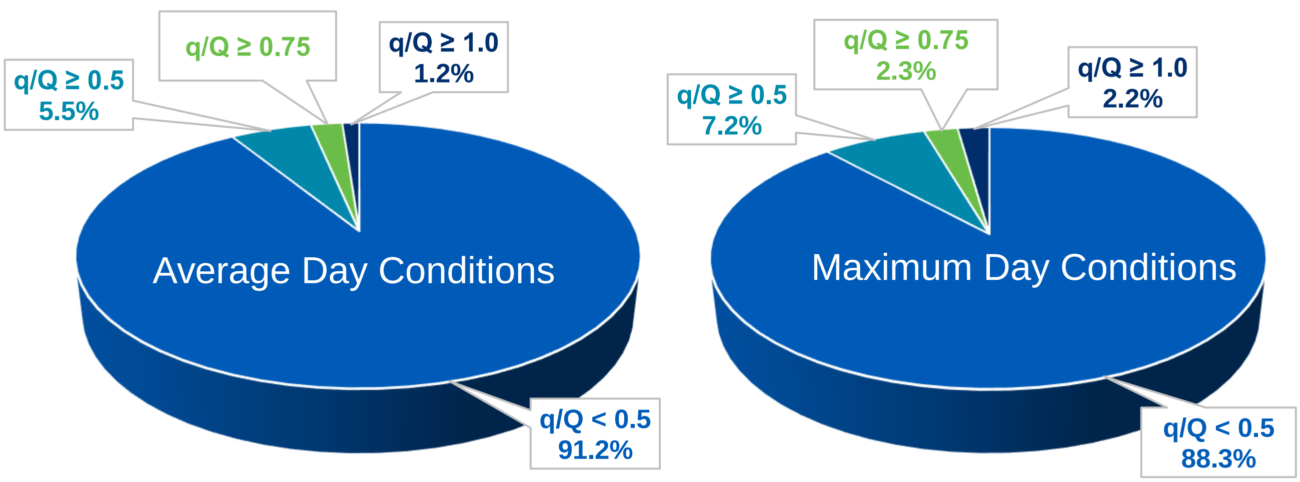

Figure 5-4: Gravity Main Capacities as Percentage of Total Modeled System Length

5.2.6. Existing System Lift Station Capacity Analysis

An analysis of lift stations that discharge to the gravity main collection system only (excluding STEP system discharging lift stations) was conducted to evaluate capacity for existing average day and maximum day conditions. The analysis calculated the ratio of modeled lift station influent flow to the single pump operating discharge capacity. This parameter is presented in Table 5-6, and is used to quantify the overall capacity of a lift station regardless of the wet well diameter and potential storage volume.

Table 5-6: Summary of Modeled Lift Station Capacities

Lift Station [1]	Modeled Capacity (gpm)	Average Day Influent Flow (gpm)	Average Day Lift Station In/Out Ratio	Maximum Day Influent Flow (gpm)	Maximum Day Lift Station In/Out Ratio
Grant Creek	500	1,057	**2.11**	1,126	**2.25**
Mastad	259	452	**1.75**	481	**1.86**
Momont No. 1	600	696	**1.16**	815	**1.36**
Linda Vista Golf Course [2]	700	575	0.82 [2]	675	0.96 [2]
University	361	143	0.4	325	**0.9**
Pleasant View 1	30	12	0.4	26	**0.87**
East Broadway	820	573	0.7	705	**0.86**
Kelly Island	705	555	0.79	596	**0.85**
Momont No. 2	1,000	685	0.69	777	0.78
Caras Park	1,585	867	0.55	1,165	0.74
Reserve St. [3]	2,150	928	0.43	1,561	0.73
Futurity	585	301	0.51	330	0.56
Fort Missoula	59	17	0.29	28	0.47
Mullan Rd.	500	160	0.32	180	0.36
Pleasant View 2	30	5	0.17	10	0.33
Kona Ranch	125	3	0.24	4	0.32
Community Hospital	300	41	0.14	86	0.28
Third St.	185	25	0.14	50	0.27
East Missoula	460	55	0.12	103	0.22
Dickens St.	489	49	0.1	103	0.21
Travois Village [4]	162	9	0.06	18	0.11
Big Sky	80	3	0.04	6	0.08
Lower Miller Creek [5]	480	13	0.03	22	0.05
Waldo	423	4	0.01	6	0.01

[1] All but two lift stations have two pumps (1 duty, 1 standby) and a single pump was modeled.
[2] aka Lower Miller Creek No. 2 lift station; only a single pump was modeled but the LS has 4 pumps (3 duty, 1 standby).
[3] Only a single pump was modeled but the LS has 3 pumps (2 duty, 1 standby).
[4] Travois Village lift station is a private facility that discharges to the City collection system and was included in model analyses due to the relative size and area served.
[5] aka Linda Vista 14 lift station

Eight of the 24 modeled lift stations exceed the 0.80 ratio. Lift stations with an In/Out ratio that exceeds 0.80 indicates that the current discharge capacity is at risk of not being adequate for conveying the non-attenuated modeled influent flows by needing to use reserve capacity. These 8 lift stations should be considered for more detailed study to verify current operational abilities to adequately convey current flows.

Three of the 24 lift stations have ratios greater than 1.00. A ratio in excess of 1.00 indicates that the current lift station pump capacity is not adequate to convey non-attenuated modeled influent flows and both available pumps may need to operate during peak flows, leaving the pump station without a redundant pump.

- The Grant Creek lift station is known to need a capacity upgrade by City staff and has been listed on previous capital improvement lists.
- The Momont No. 1 lift station was recently upgraded with variable frequency drives allowing City staff to increase the single pump capacity of 600 gpm up to 1,200 gpm with user interface at the station.
- The Mastad lift station should be reviewed for adequate capacity considering the extremity of the in/out ratio, and the relatively close proximity of the Mastad lift station wet well to the Waldo lift station force main discharge point. Approximately 600 feet of 12-inch gravity main separate the Waldo force main discharge point and the Mastad wet well. This relatively short distance limits the attenuation potential of Waldo pumped flows conveyed to the Mastad wet well.

5.3. NEAR-TERM COLLECTION SYSTEM MODEL EVALUATION

Specific areas identified by the City of Missoula are anticipated to experience significant development in the near-term future. This section details the predicted capacity impacts of these significant near-term developments to the existing collection system. Hydraulic model loading for six known near-term significant developments identified by the City are listed in Table 5-7.

Table 5-7: Near-Term Significant Developments and Loads

Development Name	Residential	Commercial	Average Day Demand (gpm)	Maximum Day Demand (gpm)
Riverfront Triangle	250 multifamily units	60,000 sf conference, 195-room hotel, 50,000-sf office, and 35,000 sf retail	76.3	174.9
ROAM Student Living	488 multifamily	6,500-sf retail	30.6	70.3
Millsite	700 units	150,000-200,000 sf	67.4	154.5
Mercantile/Residence Inn		24,000 sf retail/convention, 160,000 sf hotel with 154 rooms	21.7	49.8
Linda Vista Estates	976 single family and 444 multifamily	-	190.5 [1]	436.9 [1]
Hillview Way	610 units	-	59.5	136.4
[1] Hydraulic load value does not include additional 43 gpm of infiltration.				

5.3.1. Summary of Near-Term Model Loading

The residential portion of average day hydraulic loads listed in Table 5-7 were developed by identifying the total population associated with each near-term development and applying the residential per capita loading value of 60 gpcd reported in Chapter 3. Residential populations were calculated from the people-per-household values obtained from traffic analysis zone (TAZ) areas encompassing the near-term developments, and the estimated units listed in Table 5-7.

The commercial portion of average day hydraulic loads were developed by various methods. The first method was by applying the non-residential per capita loading value of 27 gpcd reported in Chapter 3 to the estimated populations associated with commercial offices. A second method was to apply the residential per capita loading value of 60 gpcd to populations identified for commercial hotels. Convention centers and retail uses had per capita loading values of 8 and 10 gpcd respectively applied as referenced by Wastewater Engineering Treatment and Reuse 4th Edition, Metcalf & Eddy. Populations per area for all commercial uses were obtained by reference from 2018 International Building Code, Table 1004.5. The maximum day loads were calculated using the same methodology and peaking factors as were used for the existing conditions analysis.

Infiltration was only included in the additional model loads for the Linda Vista Estates development since the associated development area was outside the existing active wastewater account area and requires a significant amount of new collection system infrastructure. A base infiltration of 68 gal/d-ac was applied to the 912-acre development area identified by the City, which resulted in an additional infiltration load of 43 gpm. The infiltration load was included in the model and distributed based on the portion of development area that is estimated to be conveyed to specific existing collection system manholes.

The six near-term development hydraulic loads identified in Table 5-7 were added to the existing conditions model. The locations of near-term development hydraulic loads are shown on Figure 5-5.

5.3.2. Near-Term Evaluation Criteria

Evaluation of the collection system with near-term significant developments was similar to that performed for the existing conditions. The analyses were performed to quantify the maximum potential flows within the system and identify resulting capacity deficiencies. Model analyses assumed that each lift station within the system was operating, and only maximum day loading conditions were evaluated. The model output was evaluated based on the same parameters as the existing conditions evaluation which included q/Q and lift station in/out ratios.

5.3.3. Near-Term Development Evaluation Results

The following sections summarize the model results for the near-term collection system for maximum day conditions and all modeled lift stations and STEP system pumps operating. Table 5-8 summarizes the gravity mains which are predicted to have flows that exceed 75 percent of total capacity (q/Q > 0.75) as a result of the additional near-term development loads. Only the gravity mains with deficient capacities located downstream of near-term development loads are reported in Table 5-8.

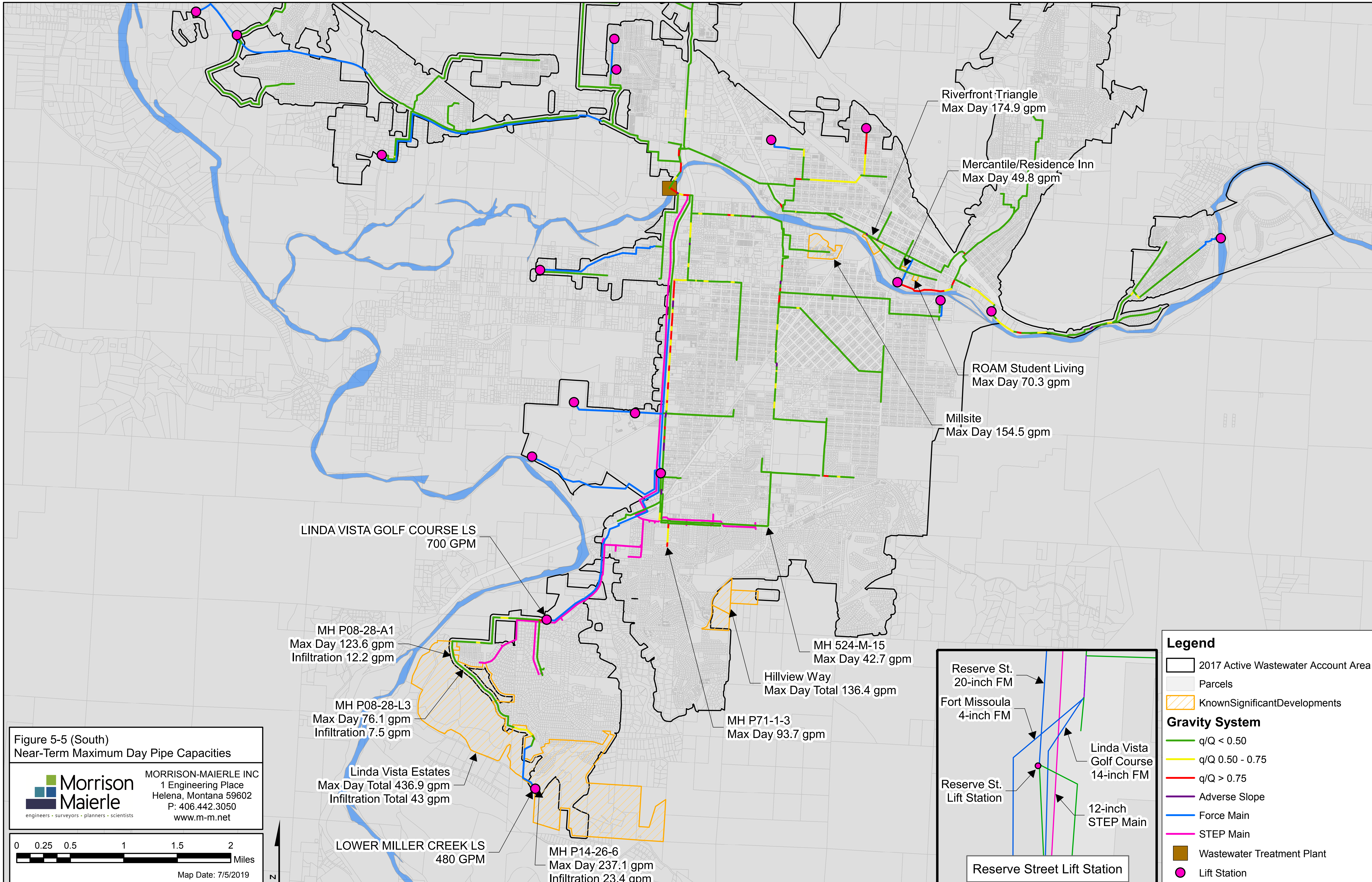

Figure 5-5 (South)
Near-Term Maximum Day Pipe Capacities

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Table 5-8: Summary of Modeled System Capacity Deficiencies – Existing Conditions with Near-Term Developments

Impacting Near-Term Development	Location	Gravity Main Segment Facility ID	Maximum Day Flow (gpm)	Maximum Day Capacity (q/Q)	Maximum Day Surcharge Depth (ft)	Full Flow (gpm)	Slope (ft/ft)	Diameter (in)	Length (ft)	Cause Summary
Riverfront Triangle Mercantile/Residence Inn ROAM Student Living	42-in. interceptor	P01-6-10P01-6-9	4,325	1.465	0.009	2,952	0.00003	42	318	Inadequate slope
Riverfront Triangle Mercantile/Residence Inn ROAM Student Living	30-in. interceptor	P76-8-1C74-1-4 C74-1-4C74-1-3	6,139 6,139	2.229 0.922	0.068	2,754 6,661	0.00013 0.00077	30 30	129 350	Receive flows from 30, 36, and 42-inch mains, inadequate slope and diameter
Millsite	36-in. interceptor	P09-48-2C61-3 C61-2C61-1	9,494 9,693	1.257 0.902	0.089	7,550 10,741	0.00037 0.00128	36 36	401 486	Receive 16-inch STEP main discharge and shallow slopes
Linda Vista Estates	Lower Miller Cr. Rd. & Linda Vista Blvd	P08-28-A21P08-28-A22	893	1.315	0.075	679	0.00032	15	316	Inadequate slope and diameter
Hillview Way	Briggs St. & 24th Ave.	P71-1-3P71-1-2A	876	1.201	0.135	729	0.00207	12	145	Receive STEP main discharge, inadequate slope
Millsite	River Rd. between Hendricksen Dr. & Missys Way	526-OO1C61-17 P00-16-AP86-3-1	2,604 2,593	1.018 1.014	0.001 0.001	2,558 2,557	0.00006 0.00007	36 36	226 110	Inadequate slope Inadequate diameter
Riverfront Triangle Mercantile/Residence Inn ROAM Student Living	Liberty Ln. and Russell St.	P97-1-9P97-1-8	2,831	0.895		3,164	0.00007	36	213	Inadequate slope
Hillview Way	Briggs St. & 39th St.	P71-1-1AC65-T42	891	0.836		1,066	0.00442	12	292	Inadequate diameter

Figure 5-5 graphically demonstrates the maximum day gravity main capacities with break points at 50 percent and 75 percent. The figure shows all deficient gravity mains in the southern portion of the collection system, not only those impacted by the near-term developments.

The addition of near-term development loads to the existing system results in 11 gravity main segments with q/Q values that exceed 0.75 for maximum day with peak diurnal factor flow conditions. Again, only the gravity mains with deficient capacities located downstream of near-term development loads are reported in Table 5-9. Of the 11 capacity deficient gravity main segments, 10 were reported previously as having capacity deficiencies associated with the existing conditions evaluation. Only one additional gravity main segment (P71-1-1AC65-T42) associated with the Hillview Way development reported a capacity deficiency compared with existing conditions maximum day flow evaluations. This main segment is at the junction with the 18-inch main in 39th Street, and has adequate slope, but inadequate diameter to convey the predicted near-term flows.

The near-term development flow conditions also impacted three existing lift stations. The analysis calculated the ratio of modeled lift station influent flow with the near-term developments to the existing single pump operating discharge capacity. This parameter is presented in Table 5-9 and is used to quantify the overall capacity of the three lift stations regardless of the wet well diameter and potential storage volume.

Table 5-9: Summary of Impacted Modeled Lift Station Capacities

Lift Station Name	Existing Conditions			Existing Conditions with Near-Term Developments	
	Modeled Capacity (gpm)	Maximum Day Influent Flow (gpm)	Maximum Day Lift Station In/Out Ratio	Maximum Day Influent Flow (gpm)	Maximum Day Lift Station In/Out Ratio
Linda Vista Golf Course [1]	700	675	***0.96***	894	**1.28**
Lower Miller Creek [2]	480	22	0.05	274	***0.57***
Reserve St.	2,150	1,561	0.73	1,697	***0.79***

[1] aka Lower Miller Creek No. 2 lift station. Existing lift station has four pumps installed with single pump capacity of 700 gpm.
[2] aka Linda Vista 14 lift station

Only the Linda Vista Golf Course lift station has a ratio greater than 1.0 indicating that lift station influent flows exceed the current single pump capacity of 700 gpm. However, the Linda Vista Golf Course lift station has four pumps as identified in Table 5-1, indicating that additional installed pump capacity exists while maintaining pumps in stand-by. The capacity of two pumps operating concurrently is unverified, and would need to convey a total of 1,120 gpm to achieve a lift station in/out ratio of 0.80. However, an increase to the Linda Vista Golf Course lift station capacity would impact the downstream 21-inch and 27-inch gravity main segments in Reserve Street.

A separate analysis was conducted in which the Linda Vista Golf Course lift station capacity was increased to 1,120 gpm. The increased lift station capacity increased the number of capacity deficient gravity mains in Reserve Street downstream of the force main discharge at Dixon Avenue from the current 15 to 28 main segments. The total length of main segments that would be capacity deficient is approximately 6,520 feet, compared to 3,430 feet of gravity main identified as currently capacity deficient for maximum day conditions. The highest surcharge level reported in the 28 capacity deficient mains was 0.26 feet.

Since the existing 21-inch and 27-inch mains have numerous lateral connections, replacement with significant slope changes would be problematic. Upsizing the 21-inch gravity mains to 27-inch, and the 27-inch gravity mains to 30-inch, along with minor slope modifications would result in capacities less than 75 percent. Due to the significant cost associated with gravity main replacement within the Reserve Street corridor to accommodate an increase in pump discharge capacity from the Linda Vista Golf Course lift station, manifolding the force main with the existing 20-inch Reserve Street lift station force main may be considered.

Utilizing the existing 20-inch force main unloads additional near-term pumped flows from the Reserve Street gravity mains, by instead using a separate 36-inch gravity main on the west side of the Reserve Street corridor closer to the wastewater treatment plant which has significant available capacity. Unloading the Linda Vista Golf Course lift station pumped flows from the existing Reserve Street gravity mains would result in only three main segments predicted to have q/Q values over 0.75 for maximum day conditions with near-term loading. This alternative is discussed in more detail in Chapter 6.

5.3.4. Near-Term Evaluation Summary

Overall, the existing collection system is adequately sized to convey the maximum day flow conditions with the addition of near-term developments assuming all lift station and STEP system pumps operating. Only one additional gravity main segment located at Briggs Street & 39th Street would have a q/Q value exceeding 0.75.

However, the Linda Vista Golf Course lift station is predicted to exceed the current single pump operating in/out ratio, necessitating increased pump capacity likely available by operating two of the four existing pumps. An increase in the pumped capacity from 700 gpm to 1,120 gpm would exacerbate current gravity main capacity deficiencies downstream of the existing force main discharge point and result in additional capacity deficient gravity mains in the Reserve Street corridor. Preliminary analysis indicates that the existing 21-inch gravity main in Reserve Street from Benton Avenue to the intersection of Davis Street and 3rd Street would need to be upsized to 27-inch diameter with minor slope modifications. The existing 27-inch gravity main in Davis Street between 3rd Avenue and River Road would need to be upsized to 30-inch diameter with minor slope modifications. Alternatively, conveying the pumped flows to the existing 36-inch gravity main originating near 9th Street S.W. and Reserve Street would unload the existing 21-inch and 27-inch Reserve Street gravity mains, and utilize available capacity in the 36-inch main closer to the treatment plant.

The impacts of the Linda Vista Golf Course lift station capacity should be considered for more detailed study to verify current operational alternatives to adequately convey current and near-term flows. Also, consideration should be given to the downstream impacts to the existing gravity mains due to pump capacity increases.

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WASTEWATER FACILITY PLAN

CHAPTER 6 - FUTURE COLLECTION SYSTEM ANALYSIS

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CHAPTER 6 FUTURE COLLECTION SYSTEM ANALYSIS

6.1. INTRODUCTION

Future collection system model analyses were performed to identify predicted capacity deficiencies within the collection system with year 2037 loading conditions. This section details the modeled 2037 maximum day load allocations and the resulting deficiencies within the collection system.

The 2037 residential and non-residential populations predicted within the planning area, including group quarters, were obtained from TAZ areas. The 2037 Wastewater Planning Area provided by the City of Missoula was approximately 34,000 acres, compared to the existing active wastewater account area of approximately 17,600 acres. The 2037 Wastewater Planning Area is shown on Figure 6-1 in relation to the 2017 Active Wastewater Account Area. All projected 2037 population and acreage within the planning area was assumed to contribute to the collection system for model loading. The existing STEP collection systems loads were assumed unchanged, with additional 2037 population and acreage contributing to the adjoining existing gravity collection system.

6.2. 2037 COLLECTION SYSTEM MODEL

The existing collection system skeletonized model network was expanded with new collection system elements to approximate collection and conveyance within large currently unsewered areas. The currently unsewered portions of the 2037 Wastewater Planning Area furthest from existing collection system infrastructure evaluated were west of Reserve Street in the South Central Basin and adjacent to Lower Miller Creek Road in the Southwest Basin.

6.2.1. 2037 Model Assumptions

Assumptions associated with potential future collection system elements are as follows:

- West of Reserve Street in the South Central Basin: Potential future 8-, 10-, and 12-inch diameter PVC gravity mains and one new 990 gpm lift station located near the end of Kenwood Drive. The potential future lift station discharges to existing manhole P01-11-D1. The existing Third Street lift station and Big Sky lift station were assumed abandoned. A potential future gravity main extension was included to the existing Community Hospital lift station.

- Lower Miller Creek Road in the Southwest Basin: Potential future 10-inch diameter PVC gravity main connections to existing 12-inch gravity main between Linda Vista Golf Course lift station and Lower Miller Creek lift station. Potential future 8-inch diameter main connections to existing Lower Miller Creek lift station.

- Potential future collection system elements were added to the model in these areas based on preliminary planning information acquired from previous studies. The potential future collection system elements were typically 12-inch and smaller gravity mains and manholes.

- Ground surface elevation data from City-provided digital terrain model (DTM) was used to define potential future manhole elements. Minimum ground cover over gravity mains was four feet with manhole spacing meeting the requirements of *MDEQ Circular-2*.

- The potential future collection system gravity mains were sized to adequately convey resulting flows based on a maximum main capacity of 75 percent (q/Q=0.75).

- The Thiessen polygons developed for the existing collection system were revised to accommodate loading of the potential future collection system elements developed for the 2037 skeletonized system. This was accomplished by defining additional polygons associated with potential future collection system components. The additional Thiessen polygons were developed using the same processes applied to the existing collection system elements described in Chapter 5 and using the 2037 Wastewater Planning Area boundary as the ultimate extents.

6.2.2. 2037 Conditions Maximum Day with Peak Diurnal Flow Allocation / Loading

Model loading for the 2037 collection system analysis was initiated with allocated average day loads to the skeletonized collection system network. This was accomplished by applying the per capita loading values of 60 gpcd and 27 gpcd reported in Chapter 2 to the residential and non-residential 2037 populations associated with individual TAZ areas. The same maximum day and peak diurnal factors used in the existing conditions model were applied to the distributed average day loads to obtain the maximum day loads.

The existing maximum day total infiltration load of 2.57 mgd associated with the existing active wastewater account area used in the existing conditions model analysis was maintained for the 2037 analysis. A baseline infiltration value of 68 gal/d-ac was applied to the additional acres associated with the 2037 Wastewater Planning Area that is currently without collection system infrastructure.

The near-term development loads identified in Chapter 5 Table 5-10 were included in the 2037 loads in lieu of the TAZ populations underlying the development areas. The near-term development maximum day total loading of 1,023 gpm was included, along with the 43 gpm infiltration load specific to the Linda Vista development area expansion.

The model runs assume that all lift stations and STEP pumps are operating, and existing lift stations maintain existing single pump capacities, and that attenuation of flow downstream of the lift stations is not considered.

Similar to the existing 2017 conditions hydraulic loading, residential, non-residential, and infiltration loads were replaced by individual STEP pump operational conditions that would occur after a City-wide power outage.

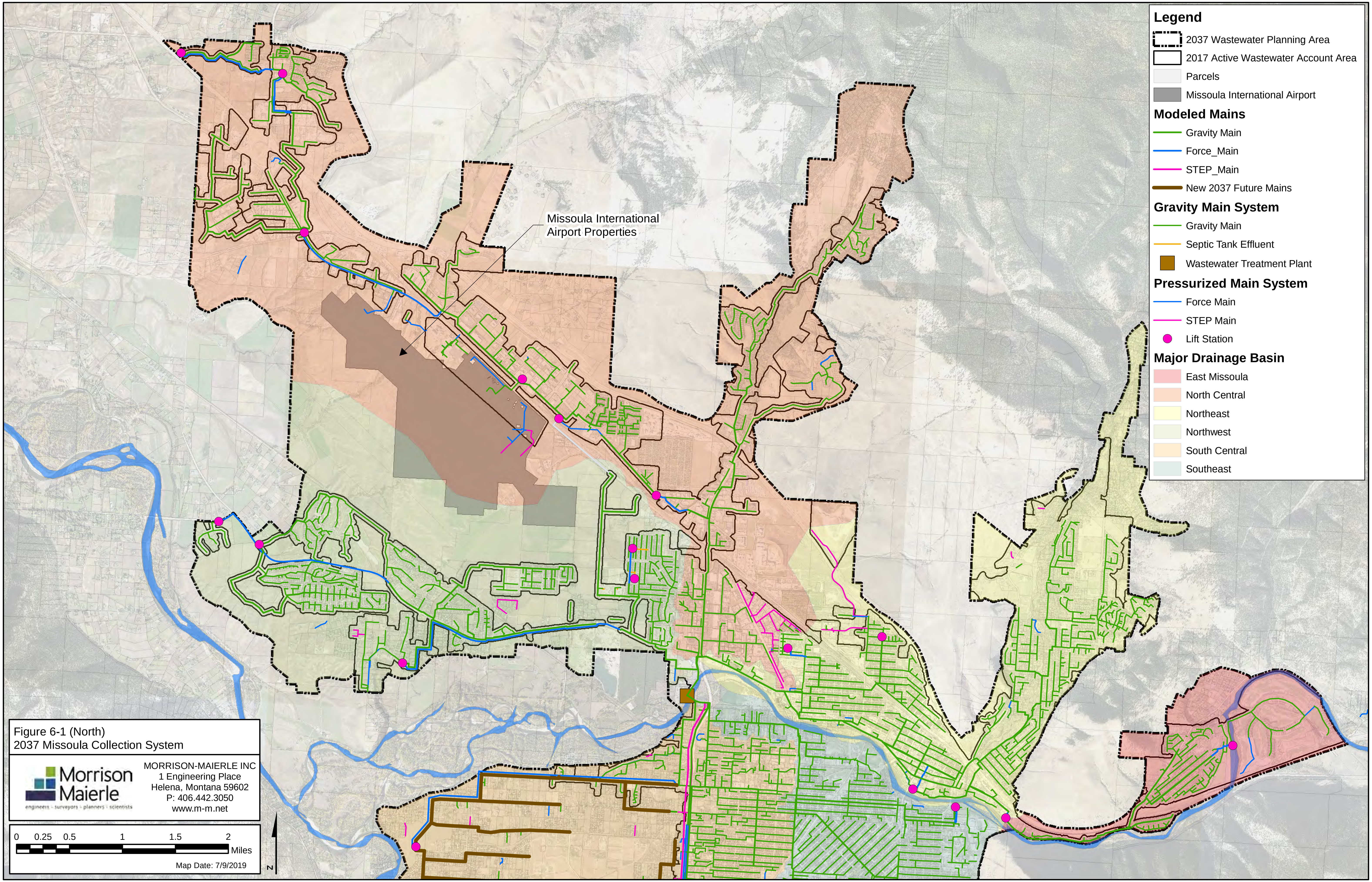
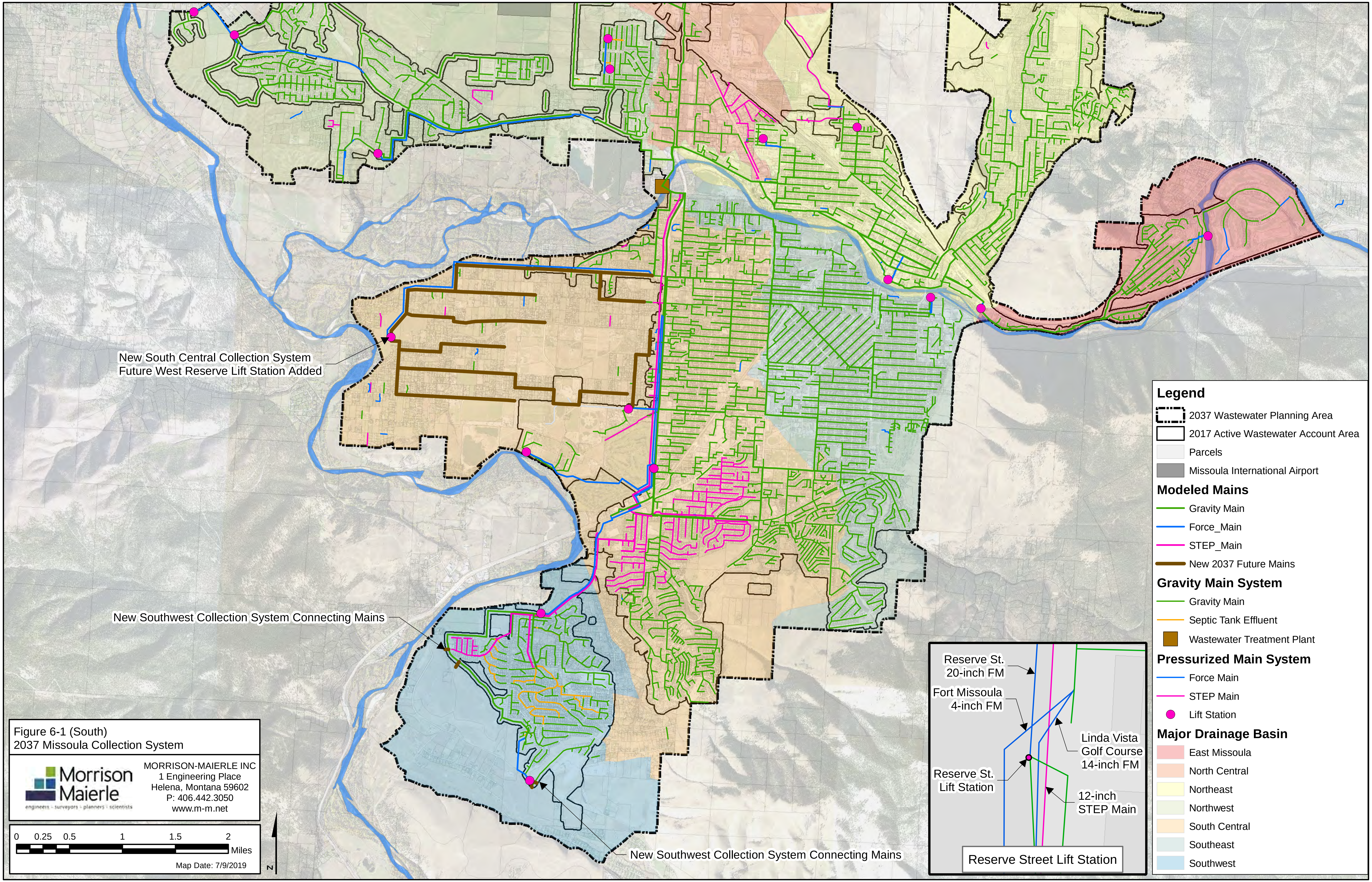


Figure 6-1 (North)
2037 Missoula Collection System



New South Central Collection System
Future West Reserve Lift Station Added
New Southwest Collection System Connecting Mains
New Southwest Collection System Connecting Mains
Legend
2037 Wastewater Planning Area
2017 Active Wastewater Account Area
Parcels
Missoula International Airport
Modeled Mains
Gravity Main
Force_Main
STEP_Main
New 2037 Future Mains
Gravity Main System
Gravity Main
Septic Tank Effluent
Wastewater Treatment Plant
Pressurized Main System
Force Main
STEP Main
Lift Station
Major Drainage Basin
East Missoula
North Central
Northeast
Northwest
South Central
Southeast
Southwest
Reserve St.
20-inch FM
Fort Missoula
4-inch FM
Linda Vista
Golf Course
14-inch FM
Reserve St.
Lift Station
12-inch
STEP Main
Reserve Street Lift Station
Figure 6-1 (South)
2037 Missoula Collection System
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6.3. SUMMARY OF 2037 MODEL FLOWS

A summary of the major drainage basin flows and southern STEP system flows is provided in Table 6-1 for 2037 loading conditions. The only lift station modification was the addition of the potential future 990 gpm lift station associated with the currently unsewered expansion area west of Reserve Street.

Table 6-1: Summary of 2037 Model Loading Flows

Trunk Main / Major Drainage Basin	Discharge	Existing 2017 Maximum Day Flow (gpm)	2037 Maximum Day Flow (gpm)	Percent Increase (%)	Notes
North Central	North Interceptor MH P01-6-12	1,149	1,694	47	
Northwest	North Interceptor MH P01-6-8	992	1,734	75	
Northeast	North Interceptor MH P01-6-10	3,661	4,995	36	
South Central	South Interceptor [1] 27-inch @ MH C61-6 [2] 30-inch @ MH P09-48-1 [3] 36-inch @ MH P98-38-1	2,409 2,156 198	2,824 2,189 1,053 [6]	17 2 432	
Southeast	South Interceptor MH C61-6	2,453	3,341	36	
Southern STEP System 16-inch Main Discharge	South Interceptor MH P09-48-2	2,301	2,301	0	
Summation equivalent to model WWTP influent		15,379	20,162	31	
Southwest	South Central Trunk / Drainage Basin [4] MH C65-T33C	700	700	0	Flow is conveyed through South Central Major Drainage Basin
East Missoula	Northeast Trunk / Drainage Basin [5] MH 376-9A	820	820	0	Flow is conveyed through Northeast Major Drainage Basin

[1] 27-inch trunk main conveys S. Reserve St. gravity main collection and Linda Vista Golf Course lift station flows.
[2] 30-inch short length trunk main primarily conveying S. Reserve St. lift station flows.
[3] 36-inch short length trunk main primarily conveying flows immediately south and east of WWTP.
[4] MH C65-T33C receives flow from Linda Vista Golf Course Lift Station.
[5] MH 376-9A receives flow from East Broadway Lift Station.
[6] Includes 990 gpm discharge from potential future West Reserve Lift Station.

Table 6-1 demonstrates a key point regarding flow distribution, conveyance, and operations within the collection system, like the existing system. The higher hydraulic loads associated with the 2037 maximum day conditions result in overall increases to the wastewater treatment plant. However, the numerous lift stations throughout the collection system influence the flows. The portion of the 2037 Wastewater

Planning Area that drains to the WWTP exclusively by gravity flow is approximately 35 percent. The remaining 65 percent of the 2037 Wastewater Planning Area is intercepted by lift stations and STEP systems with flows discharged downstream at a fixed rate, with variable frequency of pumps on or off.

Table 6-1 demonstrates the 2037 predicted flows within all major drainage basin trunk mains will increase, even when assuming the existing capacities of lift stations remain unchanged. However, numerous lift stations throughout the existing collection system would require capacity increases to adequately convey individual lift station influent flows. Instances in which existing lift stations require capacity increases will result in impacts to the downstream gravity main capacities. Therefore, suggested capacity modifications to existing lift stations need to be addressed first, followed by the resulting gravity main capacity modifications. These collection system modifications are discussed in detail in the following sections.

6.4. 2037 COLLECTION SYSTEM ANALYSIS

The following sections summarize the model results for the 2037 collection system for maximum day conditions and all modeled lift stations and STEP system pumps operating.

6.4.1. 2037 Evaluation Criteria

Evaluation of the 2037 collection system was similar to that performed for the existing conditions. The analyses were performed to quantify the maximum potential flows within the system with each lift station operating, and only maximum day loading conditions. The model output was evaluated based on similar parameters to the existing conditions and near-term development evaluations which included q/Q and lift station in/out ratios.

6.4.2. 2037 Evaluation Results

The West Reserve lift station is the only potential future modeled lift station required to provide service to the area west of Reserve Street. The addition of the West Reserve lift station negates the need for the existing Third Street lift station and Big Sky lift station which were modeled as abandoned. Note that all existing modeled lift stations retain their current discharge capacities. Figure 6-2 graphically demonstrates the impacts of future 2037 maximum day conditions on the existing collection system. The gravity main capacities are color coded with break points at 50 percent and 75 percent. Gravity mains flowing more than 75% full are considered capacity deficient. Lift stations are noted with in/out ratios.

The lift station in/out ratios noted in Figure 6-2 were calculated from the ratio of modeled lift station influent flow to the existing single pump operating discharge capacity. Table 6-2 compares the influent flows and in/out ratios of modeled lift stations for both the existing 2017 conditions and future 2037 conditions model results.

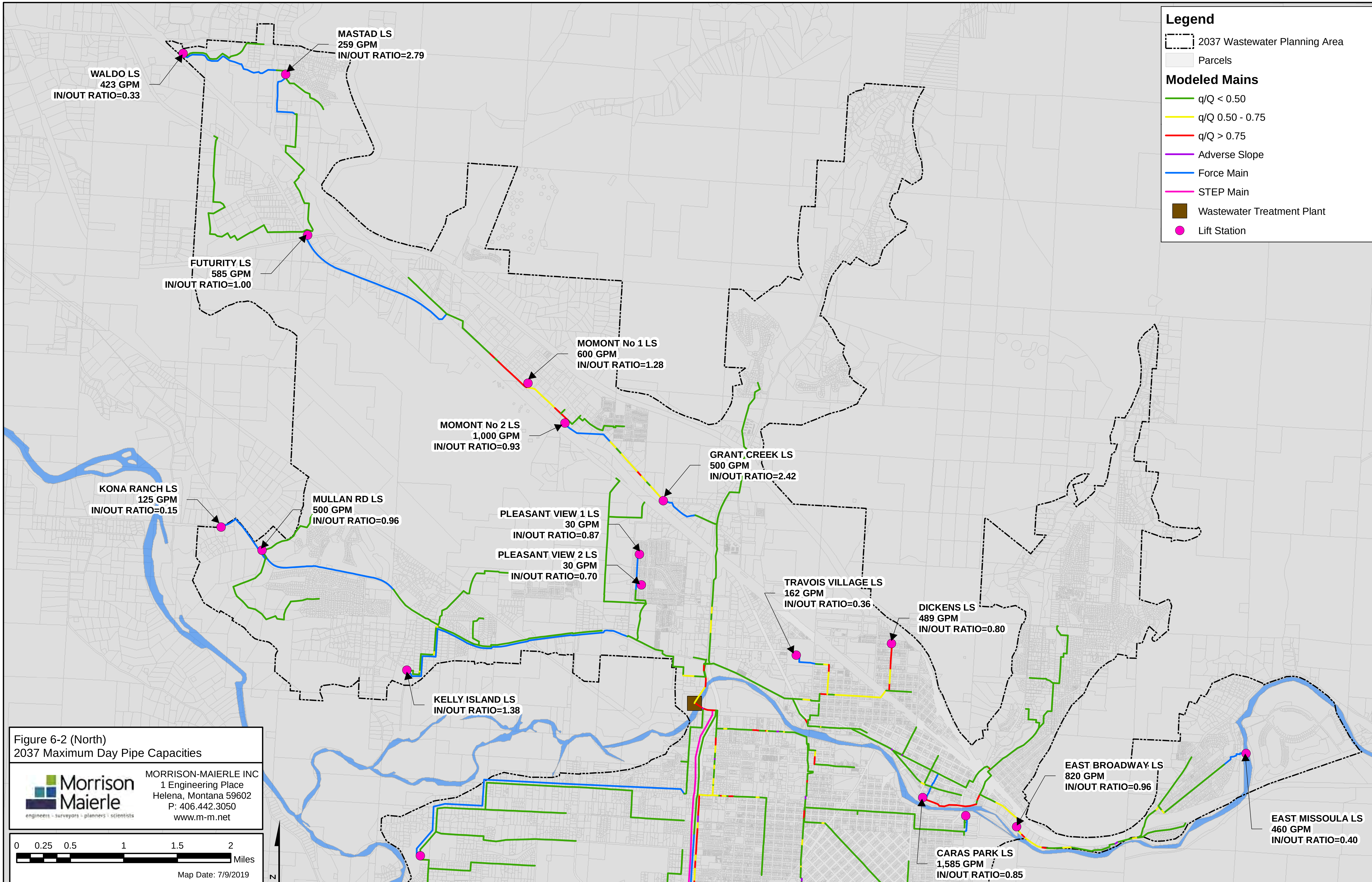

Figure 6-2 (North)
2037 Maximum Day Pipe Capacities

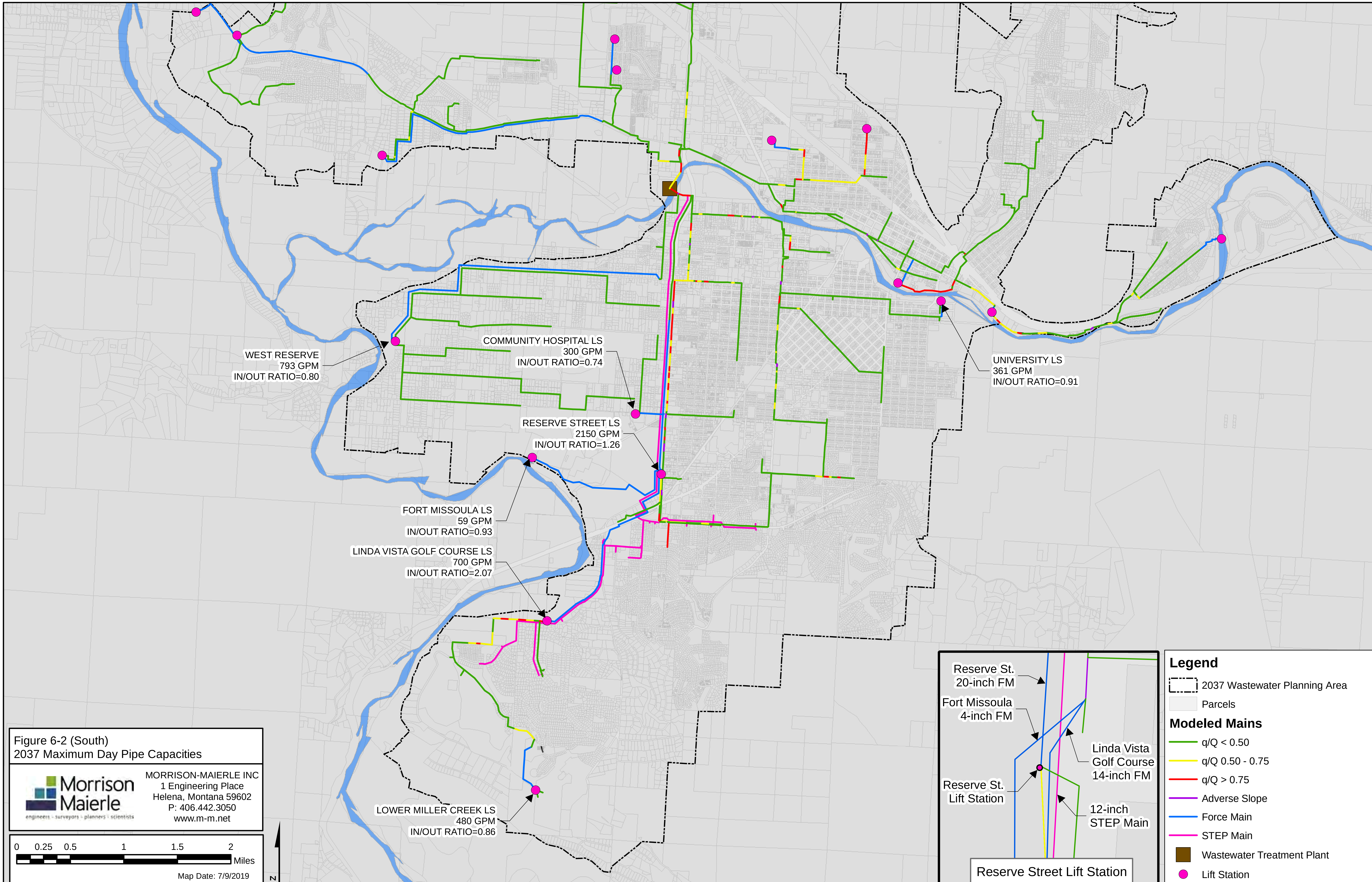

Figure 6-2 (South)
2037 Maximum Day Pipe Capacities

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Table 6-2: Summary of Modeled Lift Station Capacities

Lift Station [1]	Existing Pump Capacity (gpm)	2017 Maximum Day Influent Flow (gpm)	2017 Maximum Day Lift Station In/Out Ratio	2037 Maximum Day Influent Flow (gpm)	2037 Maximum Day Lift Station In/Out Ratio
Mastad	259	481	**1.86**	723	**2.79**
Grant Creek	500	1,126	**2.25**	1,211	**2.42**
Linda Vista Golf Course [2]	700	675	0.96 [2]	1,449	2.07 [2]
Kelly Island	705	596	**0.85**	972	**1.38**
Momont No. 1	600	815	**1.36**	769	**1.28**
Reserve St. [3]	2,150	1,561	0.73	2,701	1.26
Futurity	585	330	0.56	586	**1.0**
Mullan Rd.	500	180	0.36	479	**0.96**
East Broadway	820	705	0.86	784	**0.96**
Fort Missoula	59	28	0.47	55	**0.93**
Momont No. 2	1,000	777	0.78	930	**0.93**
University	361	325	0.9	328	**0.91**
Pleasant View 1	30	26	0.87	26	0.87
Lower Miller Creek [4]	480	22	0.05	415	0.86
Caras Park	1,585	1,165	0.74	1,343	0.85
Dickens St.	489	103	0.21	391	0.8
West Reserve [5]	990	-	-	793	0.8
Community Hospital	300	86	0.28	221	0.74
Pleasant View 2	30	10	0.33	22	0.73
East Missoula	460	103	0.22	186	0.4
Travois Village [6]	162	18	0.11	59	0.36
Waldo	423	6	0.01	139	0.33
Kona Ranch	125	4	0.32	19	0.15
Third St. [7]	-	50	0.27	-	-
Big Sky [7]	-	6	0.08	-	-

[1] All but two lift stations have two pumps (1 duty, 1 standby) and a single pump was modeled.
[2] aka Lower Miller Creek No. 2 lift station; only a single pump was modeled but the LS has 4 pumps (3 duty, 1 standby).
[3] Only a single pump was modeled but the LS has 3 pumps (2 duty, 1 standby).
[4] aka Linda Vista 14 lift station
[5] New lift station for future 2037 analysis to service area West of Reserve St.
[6] Travois Village lift station is a private facility that discharges to the City collection system, and was included in model analyses due to the relative size and area served.
[7] Existing lift station assumed abandoned for 2037 analysis

The table demonstrates that every existing lift station will experience an increased influent flow associated with the 2037 population loading, except the Momont No. 1 lift station which is predicted to experience a relatively small decrease in influent flow. As a result of increased influent flows, a total of 15 existing lift stations are predicted to exceed an in/out ratio of 0.80, and seven existing lift stations are predicted have a ratio of 1.0 or greater which indicates inadequate capacity, not accounting for wet well diameter and potential storage volume.

6.4.3. 2037 Lift Station Alternatives

Since 15 existing lift stations are predicted to exceed an in/out ratio of 0.80, and seven existing lift stations are predicted to not have adequate discharge capacities to convey the 2037 maximum day influent flows, improvements to increase lift station capacities are warranted. Increases to lift station pumped capacities would result in increased downstream gravity main flows, which would further exacerbate the gravity main capacity deficiencies shown in Figure 6-2.

By contrast, some lift stations are predicted to have excess pumped capacity for the maximum day influent flows based on 2037 population hydraulic loading including the Kona Ranch and Waldo lift stations with in/out ratios of 0.15 and 0.33 respectively. The relatively high discharge of these lift stations which are in series with other lift stations further downstream has a negative impact on the downstream lift stations and gravity main capacities.

Appreciating that existing lift station pump revisions are typically more economical than upsizing existing gravity mains between lift stations, a more targeted analysis was conducted to determine the required 2037 maximum day lift station capacities. The analysis started at the furthest upstream lift stations and worked downstream to determine the required pumped capacity to serve the predicted 2037 maximum day flows. Due to the full-build out assumption associated with the 2037 loading conditions, the assumed maximum lift station in/out ratio criteria used was 0.95. This ratio would allow lift stations to pass the 2037 maximum day flows with a 5% reserve capacity, without factoring pumped flow attenuation in gravity mains, storage volume in wetwells, pumping frequency/duration, and equally sized stand-by pumps.

The Waldo lift station in/out ratio was also modified since it generates an initial baseline pumped flow that must be conveyed by six consecutive lift stations arranged in series. The current Waldo lift station pump capacity is well in excess of both the 2017 maximum day and 2037 maximum day influent flows. Waldo lift station was assigned a minimum in/out ratio of 0.90 to reduce the negative impacts imparted on the downstream lift stations if the existing pump capacities were used.

Momont No. 2 Force Main Realignment

Lift station force main modification options were also identified specific to the Momont No. 2 and Grant Creek lift station force mains. In 2004, a new 24-inch gravity main was installed within the Southwest drainage basin that terminated at West Broadway Street approximately 3,700 feet from Momont No. 2 lift station. Also in 2004, a new 15-inch diameter gravity main was installed within the Southwest drainage basin that also terminated at West Broadway Street approximately 1,000 feet from Grant Creek lift station. A significant amount of available capacity is predicted within these two gravity mains which could receive and convey the Momont No. 2 or Grant Creek lift station pumped flows.

Installation of a new 12-inch 4,220-foot force main from Momont No. 2 to the 24-inch main by crossing the Montana Rail Link railroad tracks was identified to be most advantageous for the following reasons:

- Redirecting the Momont No. 2 pumped discharge away from the Grant Creek lift station influent would eliminate the pump capacity upgrades of Grant Creek lift station that would otherwise be required to accommodate 2037 flows.

- The Momont No. 2 modified pump capacity to accommodate 2037 maximum day influent flows is 1,500-gpm listed in Table 6-3. A 1,500-gpm pumped discharge to the existing 15-inch gravity main between Momont No. 2 and Grant Creek lift stations would result in approximately 3,300-ft of gravity main with future capacities exceeding q/Q values 0.90, with numerous manholes predicted to surcharge with q/Q values exceeding 1.00. Conveying the future Momont No. 2 pumped flow of 1,500-gpm to the 24-inch gravity main in West Broadway would negate upsizing the existing 15-inch main between Momont No. 2 and Grant Creek lift stations.

- Similarly, if the Momont No. 2 modified pump capacity of 1,500-gpm was ultimately conveyed to Grant Creek lift station, and the Grant Creek lift station inflows were redirected to the existing 15-inch diameter gravity main terminated in West Broadway, approximately 1,600-ft of this 15-inch main would also experience q/Q values exceeding 0.90, with additional manholes predicted to surcharge. The existing 24-inch main terminated in West Broadway could receive the Momont No. 2 future pumped capacity of 1,500-gpm without exceeding gravity main q/Q values of 0.75.

- Redirecting Momont No. 2 flows away from the Grant Creek lift station influent would reduce the number of lift stations that pump in series north of the Montana Rail Link railroad tracks to five instead of six lift stations in series. This reduces the resulting total horsepower and energy costs associated with re-pumping of previously pumped flow.

Based on this evaluation, the new 12-inch force main alternative was selected as the recommend system modification included in subsequent analysis. Results of the lift station alternatives analysis and recommended pump capacities are presented in Table 6-3.

6.4.4. 2037 Gravity Main Alternatives

Using the recommended lift station pump capacities listed in Table 6-3, the resulting pumped flow changes were accounted for within the gravity main network, and recommended capacity modifications to the gravity main network were accomplished. Some capacity deficient areas had multiple modification alternatives available for consideration and are detailed in this section.

South Reserve Street Gravity Mains and Linda Vista Golf Course Lift Station

The most significant capacity impacts to the system due to modified pump capacities was associated with the existing 21-inch and 27-inch gravity mains within the South Reserve Street corridor from the Linda Vista Golf Course lift station force main discharge to River Road. Fifteen existing gravity main segments in the South Reserve Street corridor were identified as capacity deficient under the existing 2017 conditions with the Linda Vista Golf Course lift station operating at current single pump capacity of 700 gpm. With the 2037 modified pump capacity of 1,525 gpm listed in Table 6-3, and additional flows due to 2037 population growth, the capacity deficient mains downstream of the force main discharge

Table 6-3: Summary of 2037 Modeled Lift Station Modified Capacities

Lift Station [1]	Existing Pump Capacity (gpm)	2037 Maximum Day Influent Flow (gpm)	Modified Pump Capacity (gpm)	Modified 2037 Maximum Day Influent Flow (gpm)	2037 Maximum Day Lift Station In/Out Ratio
Mastad	259	723	480	455	0.95
Grant Creek	500	1,211	-	211	0.42
Linda Vista Golf Course [2]	700	1,449	1,525	-	0.95
Kelly Island	705	972	1,025	-	0.95
Momont No. 1 [3]	600	544	1,090	1,034	0.95
Reserve St. [4]	2,150	2,701	2,845	-	0.95
Futurity	585	586	850	807	0.95
Mullan Rd.	500	479	-	-	0.96
East Broadway	820	784	-	-	0.96
Fort Missoula	59	55	-	-	0.93
Momont No. 2	1,000	930	1,500	1,420	0.95
University	361	328	-	-	0.91
Pleasant View 1	30	26	-	-	0.87
Lower Miller Creek [5]	480	415	-	-	0.86
Caras Park	1,585	1,343	-	-	0.85
Dickens St.	489	391	-	-	0.8
West Reserve [6]	-	-	990	793	0.8
Community Hospital	300	221	-	-	0.74
Pleasant View 2	30	21	-	-	0.7
East Missoula	460	186	-	-	0.4
Travois Village [7]	162	59	-	-	0.36
Waldo	423	139	155	-	0.9
Kona Ranch	125	19	-	-	0.15
Third St. [8]	-	-	-	-	-
Big Sky [8]	-	-	-	-	-

[1] All but two lift stations have two pumps (1 duty, 1 standby) and a single pump was modeled.
[2] aka Lower Miller Creek No. 2 lift station. Existing lift station has four pumps installed with single pump capacity of 700 gpm.
[3] Momont No. 1 LS is equipped with variable frequency drives. Single pump 100% speed would have 1,200 gpm capacity.
[4] Only a single pump was modeled but the LS has 3 pumps (2 duty, 1 standby).
[5] aka Linda Vista 14 lift station
[6] New lift station for 2037 analysis to service area West of Reserve St.
[7] Travois Village lift station is a private facility that discharges to the City collection system and was included in model analyses due to the relative size and area served.
[8] Existing lift station assumed abandoned for 2037 analysis

increased to 36 main segments exceeding q/Q values of 0.75 and 21 main segments exceeding q/Q of 1.00. The total length of these capacity deficient main segments was 8,380 feet, with deficiencies primarily due to inadequate slope of the mains which were installed circa 1965, and are comprised primarily of ACP, VCP, RCP, and DIP materials.

The alternatives to rectify the South Reserve Street corridor gravity main deficiencies included gravity main replacement with increased diameter, and modification of the Linda Vista Golf Course lift station force main discharge. The replacement alternative would require increasing the diameter of approximately 10,100 feet of existing 21-inch gravity main to 27-inch from the Linda Vista Golf Course force main discharge to the intersection of 3rd Street S.W. and Davis Street. Additional replacement of approximately 3,300 feet of existing 27-inch gravity main to 30-inch from 3rd Street S.W. to River Road along Davis St would be required.

Due to the significant costs and construction challenges to undertake gravity main replacements within the Reserve Street corridor, modification to the Linda Vista Golf Course force main was studied. A relatively simple force main revision was evaluated in which the existing 14-inch Linda Vista Golf Course force main is manifolded to the existing 20-inch Reserve Street lift station force main. This would unload the Linda Vista Golf Course lift station 2037 pumped capacity of 1,525 gpm from the Reserve Street gravity mains which would result in only four gravity main segments exceeding q/Q values of 0.75, with three exceeding q/Q of 1.00 compared to 36 and 21 respectively with the 1,525 gpm pumped flow conveyed through the Reserve Street gravity main. The maximum surcharge depth is predicted to be 0.03 feet, nearly identical to the existing 2017 maximum day condition.

Both the Linda Vista Golf Course and Reserve Street lift station would utilize the existing 20-inch force main which discharges to the existing 36-inch gravity main on the west side of Reserve Street. The existing 36-inch PVC gravity main has significant available capacity. The combined 2037 pumped flows of Linda Vista Golf Course and Reserve Street lift stations of 4,370 gpm would result in a velocity of 4.5 ft/s in the 20-inch force main. The combined 4,370 gpm would not result in any capacity deficiencies in the 36-inch gravity main up to the existing 36-inch interceptor crossing the Clark Fork River.

Since the Linda Vista Golf Course and Reserve Street force mains are in close proximity, manifolding the mains would be cost effective for the benefit of unloading the existing 21-inch and 27-inch gravity mains. Also the existing 36-inch gravity main has significant available capacity to convey the combined pumped flows of both lift station. This relatively simple and low-cost alternative was selected as the recommended system modification included in the final model.

Gravity Mains Upstream of Caras Park Lift Station and East Broadway Lift Station

The gravity main immediately upstream of the Caras Park lift station was another significant reach with capacity issues that was identified. A total of 10 gravity main segments were predicted to exceed q/Q of 1.00. These gravity mains were also identified as capacity deficient under the existing 2017 maximum day flow conditions, with nine main segments that exceed q/Q values of 0.75, eight of which exceed 1.00. Although the deficient gravity mains were not impacted by 2037 modified pump capacities, the additional flow due to 2037 population growth further impacts deficiencies. Alternatives evaluated to rectify the gravity main deficiencies included replacement with increased diameter pipe and modification of the East Broadway lift station force main discharge.

The replacement alternative would require replacing approximately 3,300 feet of existing 15-inch ACP and RCP mains with new 18-inch PVC gravity mains from the intersection of Jackson Street and E. Broadway to the Caras Park lift station. Increased diameter along with minor modifications to pipe slopes would result in all gravity mains having q/Q values of 0.71 or less.

Modification of the existing East Broadway lift station force main would require installation of approximately 3,500 feet of new force main to an existing 21-inch gravity main at existing manhole R246-11 located at the intersection of E. Broadway and Madison Street. This would reduce the number of capacity deficient gravity mains immediately upstream of Caras Park lift station that exceed q/Q of 1.00 from 10 to one. The single remaining main segment that exceeds q/Q of 1.00 has a flat slope. The additional 820 gpm that would be redirected to the 21-inch gravity main in E. Broadway would have some minor negative capacity impacts downstream with an additional two main segments exceeding q/Q of 1.00.

Both alternatives require approximately the same amount of new main installation. However, the gravity main would be a replacement of an existing pipe with increased diameter primarily outside of paved streets. The force main would be a new pipe that would need to be installed within E. Broadway alongside an existing 15-inch gravity main. Also, the force main realignment would create an increased load on the 21-inch gravity main in E. Broadway by collecting pumped discharges separately from both the Caras Park and East Broadway lift station at 1,585 gpm and 820 gpm, respectively. In the current series pumping configuration, the 21-inch main only conveys the Caras Park pumped flow of 1,585 gpm.

Given the age of the Caras Park lift station which was rehabilitated in 2016, and that it is sized appropriately to handle the East Broadway lift station pumped flows and is not predicted to need capacity modification to accommodate 2037 loading, the gravity main replacement alternative was selected as the recommend system modification included in the analysis.

6.5. SUMMARY OF 2037 MODEL WITH RECOMMENDED ALTERNATIVES

Figure 6-3 graphically depicts the 2037 maximum day gravity main capacities with the modified lift station pump capacities detailed in Table 6-3, a new realigned Momont No.2 force main, manifolding the Linda Vista Golf Course and Reserve Street force mains, and increasing the diameter of gravity main segments upstream of Caras Park lift station. Additional recommended gravity main modifications to address relatively short portions of capacity deficient mains are also included.

The gravity main capacities are displayed in Figure 6-3 with break points at 50 percent and 75 percent. Modified or existing lift station capacities are included in the figure, along with recommended gravity main revisions to adequately convey flows. Isolated gravity main segments that exceed q/Q of 75 percent but were not recommended for modification are also shown with an explanation of deficient capacity.

Table 6-4 details the recommended gravity main modifications shown in Figure 6-3. The isolated gravity main segments that exceed q/Q of 75 percent but were not recommended for modification are included in Table 6-5 with explanation of the capacity deficiencies. The recommended upgrades listed in Table 6-4 represent a significant undertaking for the City to plan and budget. The recommended upgrades were identified primarily based upon the predicted 2037 capacity deficiencies. Additional factors influencing the recommended upgrades included the age and material of the existing gravity mains, capacity deficiencies that currently occur, and capacity deficiencies that are predicted to occur as a result of near-term development flows.

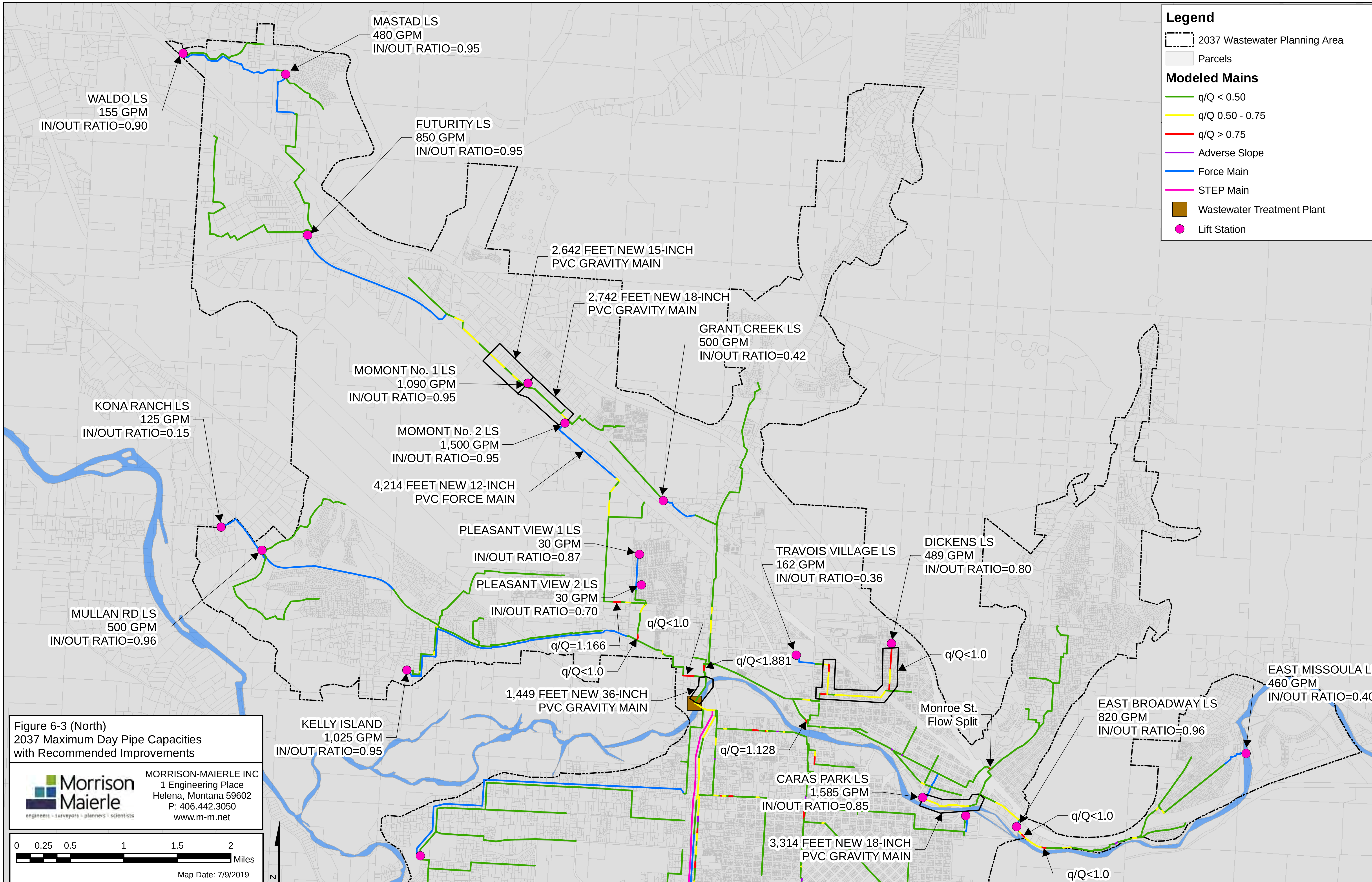

Figure 6-3 (North)
2037 Maximum Day Pipe Capacities
with Recommended Improvements

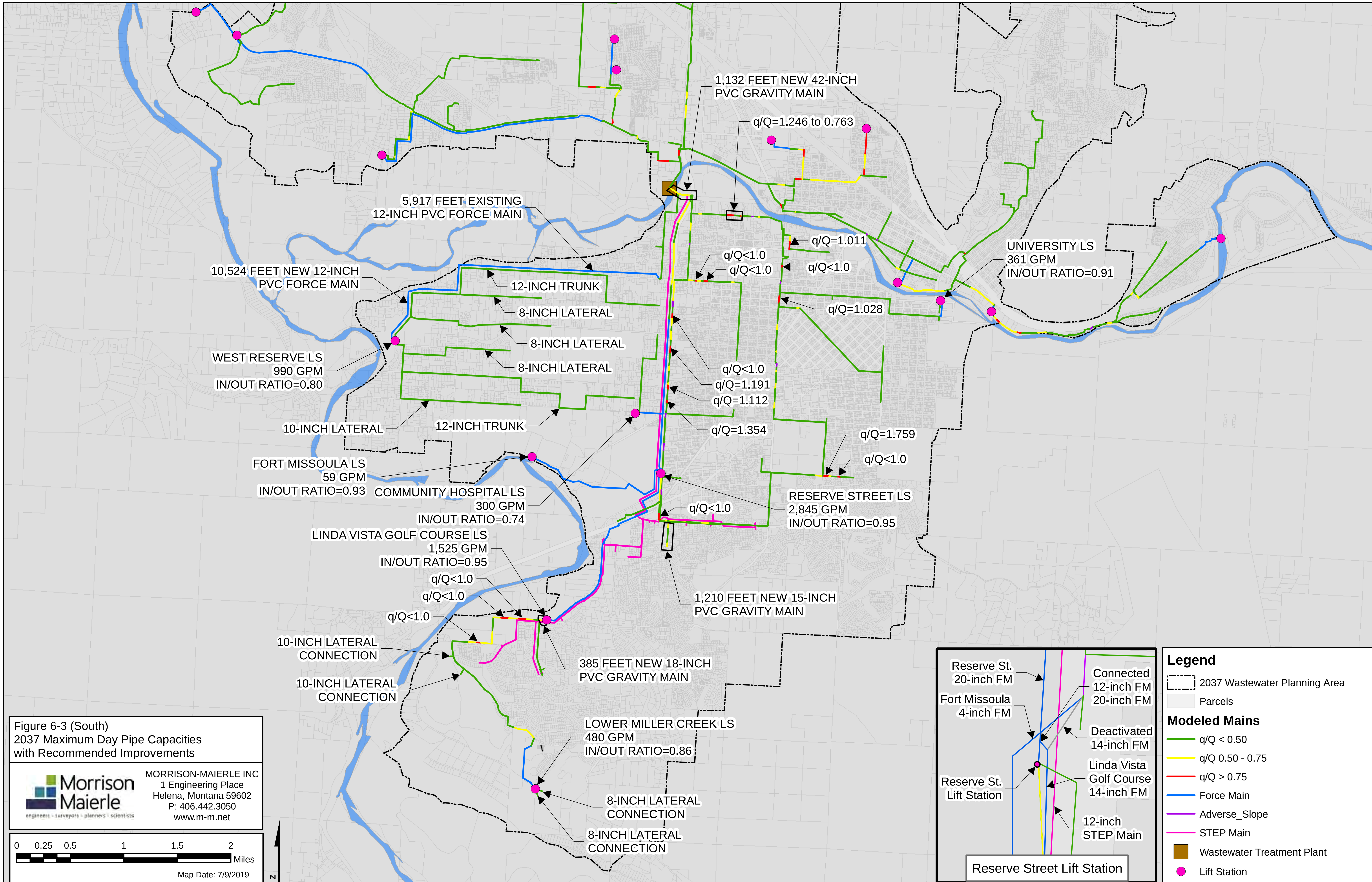
1,132 FEET NEW 42-INCH PVC GRAVITY MAIN
q/Q=1.246 to 0.763
5,917 FEET EXISTING 12-INCH PVC FORCE MAIN
q/Q=1.011
q/Q<1.0
q/Q<1.0
q/Q<1.0
UNIVERSITY LS 361 GPM IN/OUT RATIO=0.91
10,524 FEET NEW 12-INCH PVC FORCE MAIN
12-INCH TRUNK
8-INCH LATERAL
q/Q=1.028
8-INCH LATERAL
WEST RESERVE LS 990 GPM IN/OUT RATIO=0.80
8-INCH LATERAL
q/Q<1.0
q/Q=1.191
q/Q=1.112
10-INCH LATERAL
12-INCH TRUNK
q/Q=1.354
q/Q=1.759
q/Q<1.0
FORT MISSOULA LS 59 GPM IN/OUT RATIO=0.93
COMMUNITY HOSPITAL LS 300 GPM IN/OUT RATIO=0.74
RESERVE STREET LS 2,845 GPM IN/OUT RATIO=0.95
q/Q<1.0
LINDA VISTA GOLF COURSE LS 1,525 GPM IN/OUT RATIO=0.95
q/Q<1.0
q/Q<1.0
q/Q<1.0
1,210 FEET NEW 15-INCH PVC GRAVITY MAIN
10-INCH LATERAL CONNECTION
385 FEET NEW 18-INCH PVC GRAVITY MAIN
10-INCH LATERAL CONNECTION
LOWER MILLER CREEK LS 480 GPM IN/OUT RATIO=0.86
8-INCH LATERAL CONNECTION
8-INCH LATERAL CONNECTION
Reserve St. 20-inch FM
Connected 12-inch FM 20-inch FM
Fort Missoula 4-inch FM
Deactivated 14-inch FM
Linda Vista Golf Course 14-inch FM
Reserve St. Lift Station
12-inch STEP Main
Reserve Street Lift Station
Legend
2037 Wastewater Planning Area
Parcels
Modeled Mains
q/Q < 0.50
q/Q 0.50 - 0.75
q/Q > 0.75
Force Main
Adverse_Slope
STEP Main
Wastewater Treatment Plant
Lift Station
Figure 6-3 (South)
2037 Maximum Day Pipe Capacities with Recommended Improvements
Morrison Maierle
engineers · surveyors · planners · scientists
MORRISON-MAIERLE INC
1 Engineering Place
Helena, Montana 59602
P: 406.442.3050
www.m-m.net
0 0.25 0.5 1 1.5 2 Miles
Map Date: 7/9/2019
N

Table 6-4: Summary of Recommended Modifications included in the Model – 2037 Conditions

Modification Project	Alternative	Location	Number of Gravity Main Segments Upgraded	Year Installed	Existing Diameter (in)	Existing Material	Upgraded Diameter (in) [1]	Maximum Flow (gpm)	Upgraded Maximum Capacity (q/Q)	Upgraded Minimum Full Flow (gpm)	Upgraded Average Slope (ft/ft)	Upgraded Length (ft)	Cause Summary
Momont No. 2 Force Main		Realigned Momont No. 2 LS Force Main to MH P02-3-N31A in West Broadway		1980	10	DIP	12					4,214	Realigned to reduce series lift station pumping and impacts to Grant Creek LS
Caras Park	Preferred	Upsize Gravity Mains Upstream of Caras Park LS	12	NA	15	ACP/RCP	18	1,343	0.690	1,853	0.00267	3,314	Inadequate diameter
	Secondary	Realigned East Broadway LS Force Main to MH R426-11 in East Broadway		1973	10	DIP	10					3,490	Realigned to reduce series lift station pumping and impacts to mains upstream of Caras Park LS
South Reserve St. Corridor	Preferred	Connect Linda Vista Golf Course FM to Reserve St. FM											Unload the existing 21-inch and 27-inch Reserve St. gravity mains
	Secondary	Reserve St. between Linda Vista Golf Course LS force main discharge & South Ave. W.	11	1965	21	ACP/VCP	27	1,713	0.482	3,307	0.00083	2,810	Inadequate slope and diameter, 2 adverse slope mains fixed
	Secondary	Reserve St. between South Ave. W. & Davis St.	32	1965	21	ACP	27	2,845	0.744	3,395	0.00082	7,266	Inadequate slope and diameter, 1 adverse slope main fixed
	Secondary	Davis St. between 3rd St. S.W. & River Rd.	14	1965	27	ACP/RCP/ DIP	30	3,649	0.651	5,586	0.00111	3,300	Inadequate slope and diameter, 1 adverse slope main fixed
Momont No. 1 to No. 2 Mains		Between Momont No. 1 LS & No. 2 lift stations	10	1980	12 & 15	PVC	18	1,165	0.523	2,227	0.00203	2,742	Inadequate diameter
Momont No. 1 Mains		Upstream of Momont No. 1 Lift station	9	1968	8 & 12	ACP/VCP	15	1,034	0.679	1,509	0.00249	2,642	Inverted diameter transitions with Inadequate slope and diameter
30-inch Interceptor		30-in. interceptor in Clark Fork Dr.	5	1974	30	ACP/PVC	36 [2]	9,972	0.489	20,400	0.00273	1,449	Receive flows from 30, 36, and 42-inch mains, inadequate diameter
36-inch Interceptor		36-in. interceptor crossing Clark Fork River	4	1965-2010	36	ACP/PVC	42	13,116	0.744	16,230	0.00098	1,132	Receive 16-inch STEP main discharge, inadequate diameter, 1 adverse slop main fixed
Lower Miller Creek Rd. Mains		Lower Miller Cr. Rd. & Linda Vista Blvd	3	2011	15	PVC	18	1,449	0.590	2,451	0.00422	385	Inadequate slope and diameter
24th Avenue Mains		24th Ave. between Briggs St. & 39th St.	6	1971-1991	12	ACP/PVC	15	1,290	0.735	1,719	0.00592	1,210	Receive STEP main discharge, inadequate slope and diameter
										Total Force Main (preferred options)		4,214	
										Total Gravity Main (preferred options)		12,874	

[1] All recommended materials are PVC

[2] 33-inch diameter upgrade sufficient to achieve q/Q values less than 75 percent full. 36-inch diameter upgrade presented since 33-inch PVC D3034 & F679 not a standard manufactured size.

Table 6-5: Summary of Additional Gravity Main Capacity Deficiencies – 2037 Conditions

Location	Gravity Main Segment Facility ID	Maximum Day Flow (gpm)	Maximum Day Capacity (q/Q)	Maximum Day Surcharge Depth (ft)	Full Flow (gpm)	Slope (ft/ft)	Diameter (in)	Length (ft)	Cause Summary
Reserve St. between Brooks St. and Old Highway 93	P91-1-8P91-1-7	2385	0.876		2,724	0.00042	24	26	Inadequate slope
	P91-1-7P91-1-6	2445	0.918		2,663	0.00041	24	304	
Downstream of Dickens St. lift station	371-A334-EX	528	0.855		617	0.00148	12	403	Inadequate diameter
	371-1A371-A	492	0.944		522	0.00280	10	284	
	371-3A371-2A	490	0.94		522	0.00280	10	282	
	371-2A371-1A	491	0.942		522	0.00280	10	280	
	89-6089-52	636	0.792		803	0.00251	12	359	
42-in. interceptor	P01-6-10P01-6-9	5,553	1.881	0.021	2,952	0.00003	42	318	Inadequate slope
36-in. trunk main in Clark Fork Way	P01-6-14P01-6-13	3,266	0.964		3,388	0.00008	36	584	Inadequate slope
Old Bitterroot Rd.	P08-28-A18P08-28-A19	1,291	0.765		1,688	0.00200	15	389	Inadequate slope
Old Bitterroot Rd.	P08-28-A15P08-28-A16	1,232	0.788		1,563	0.00171	15	385	
Showdown Ln.	P08-28-A7P08-28-A8	1,223	0.828		1,476	0.00153	15	214	
Monroe St. flow split	R426-21R426-20	293	1.000		293	0.00000	21	72	Flat slope
Upstream of E. Broadway LS	376-16P95-22-A1	717	0.986		727	0.00037	15	89	Inadequate slope
	P95-22-A1376-15	738	0.839		880	0.00092	15	143	
	376-12376-11	762	0.795		959	0.00109	15	240	
Pattee Creek Dr. between Bancroft St. & Hollis St.	AC60-37AC60-36	292	1.759	0.072	166	0.00011	12	317	Inadequate slope
	AC60-39281-128	279	0.904		309	0.00037	12	170	
Russell St. between 6th St. S.W. and 4th St. S.W.	AC60-4AC60-3	1,077	1.028		1,048	0.00011	24	359	Inadequate slope
Russell St. between 1st St. S.W. and Milwaukee Way	N354-CN354-B	2,372	0.937		2,530	0.00019	30	112	Inadequate slope
River Rd. between Hendricksen Dr. & Missys Way	526-OO1C61-17	3,188	1.246	0.008	2,558	0.00006	36	226	Inadequate slope
	P00-16-AP86-3-1	3,168	1.239	0.004	2,557	0.00007	36	110	
	P97-20-1A526-OO1	3,178	0.763		4,166	0.00016	36	171	
Liberty Ln. and Russell St.	P97-1-9P97-1-8	3,569	1.128	0.004	3,164	0.00007	36	213	Inadequate slope
Alley North of Philips St. between Byron St. & Burns St.	89-10889-109	1,170	0.937		1,249	0.00185	15	379	Inadequate Slope
Burns St. between Turner & Stoddard St.	N260-DP00-13-A1	270	0.786		344	0.00400	8	157	Not surveyed, based on record drawings
	P00-13-A1290-3	288	0.838		344	0.00400	8	219	
North of Mullan Rd. 500-ft east of Flynn Ln.	P02-3-N3P02-3-N2	1,850	0.807		2,293	0.00009	30	230	Inadequate slope
	P02-3-N2P02-3-A6	1,853	0.919		2,016	0.00007	30	127	
Siren's Rd.	P02-3-N16P02-3-N15	1,636	1.166	0.006	1,403	0.00003	30	439	Inadequate slope
Prince St. between Wyoming St. and Montana St.	505-9505-7	504	1.011	0.014	499	0.00151	10	357	Inadequate diameter
3rd St. S.W. between Schilling St. and Davis St.	C61-13C526-DD1	524	0.947		553	0.00036	15	174	Inadequate slope
	526-BB1C61-13A	566	0.875		647	0.00050	15	147	
				Maximum Day Flow Conditions Total Deficient Length				8,279	

6.5.1. Recommended Lift Station Upgrades

The recommended lift station capacity upgrades listed in Table 6-3 to accommodate the predicted 2037 flows include abandoning the Third Street and Big Sky lift stations. These two lift stations with relatively small pumped capacities would be replaced by the proposed future West Reserve lift station. The proposed West Reserve lift station would discharge through a new 12-inch force main for approximately 2.0 miles and connect to an existing 12-inch force main installed in Third Street S.W. The total force main length would be approximately 3.1 miles.

In addition to these lift station abandonments, a total of eight existing lift stations are recommended for pumped capacity modifications. The pumped capacity modifications are recommended to improve overall efficiency for in-series pumping instances, and to adequately convey the predicted influent flows by lift stations that do not currently have adequate single pump capacities.

6.5.2. Recommended Force Main Upgrades

One of the most significant recommended upgrades in Table 6-4 involves unloading the South Reserve Street gravity mains by manifolding the 14-inch Linda Vista Golf Course force main to the 20-inch Reserve Street force main. By unloading the Linda Vista Golf Course lift station pumped flow from the gravity main, a total of approximately 10,100 feet of diameter increase of existing 21-inch and 27-inch main can be avoided.

Additional study is recommended to verify adequate pressure ratings of the existing 20-inch force main and the required discharge heads of the contributing pumps. Similarly, the Community Hospital lift station pumped flow of 300-gpm could also be unloaded from the Reserve Street corridor gravity main by manifolding with the 20-inch force main, and would also require additional study to verify force main pressure ratings and pump discharge heads.

Another significant force main alignment change is that which was recommended for the Momont No. 2 lift station. The recommended force main alignment redirects the Momont No. 2 discharge away from the Grant Creek lift station influent to an existing 24-inch gravity main in West Broadway Street to the south. The realigned 12-inch force main would have an estimated length of 4,220 feet.

6.5.3. Recommended Gravity Main Upgrades

The most significant recommended gravity main upgrade is the replacement of approximately 3,300 feet of existing 15-inch gravity main immediately upstream of Caras Park lift station with new 18-inch PVC main.

The remaining gravity main upgrades listed in Table 6-4 are associated with six identified potential projects with a total length of gravity main replacement of approximately 9,600 feet. The two largest diameter upgrades are associated with the existing 30-inch interceptor in Clark Fork Drive and the 36-

inch interceptor crossing the Clark Fork River. Both existing gravity mains are immediately upstream of the wastewater treatment plant.

The 30-inch interceptor conveys all flows north of the river, and is recommended for upgrade to 36-inch along with minor slope modifications to accommodate 2037 maximum day flow conditions. Without the diameter upgrade, main segments of the existing 30-inch interceptor would have q/Q values ranging from 3.07 to 0.61, with a maximum surcharge depth 0.31 feet. The furthest upstream main segments are the most capacity deficient where the 30-inch interceptor collects flow from three gravity mains with diameters of 30-inch, 36-inch, and 42-inch.

The 36-inch interceptor conveys all flows south of the river, and is recommended for upgrade to 42-inch to accommodate 2037 maximum day flow conditions. Without the diameter upgrade, main segments of the existing 36-inch interceptor would have q/Q values ranging from 1.40 to 0.79, with a maximum surcharge depth 0.18 feet. All 4 main segments are predicted to be exceed q/Q of 0.75 with the main segment crossing the Clark Fork River having a q/Q value of 1.080, indicating slightly pressurized conditions under the river.

The gravity main segments presented in Table 6-5 that exceed q/Q of 75 percent, but were not recommended for modification were selected based on the significance of capacity deficiencies, low surcharge potential in manholes, and the number of deficient mains nearby. Of the 33 main segments that exceed q/Q of 75 percent, 28 are predicted to have q/Q values less than 1.0, meaning the mains are not completely full and surcharging of manholes will not result. The remaining seven main segments are predicted to have q/Q values in excess of 1.0, indicating the mains are full and manholes would surcharge. The highest calculated surcharge level in manholes for these 11 main segments is 0.07 feet above the pipe crown. Due to the relatively minor surcharge levels predicted, these main segments were not recommended for upgrade.

In addition to the recommended upgrades described above, it is recommended that the City implement an annual gravity main rehabilitation and replacement program to stay abreast of problems associated with an aging collection system. At a minimum, one percent of the total collection system or about 5.2 miles should be addressed every year. If piping is replaced/rehabilitated at this rate, it will take about 100 years to rehabilitate the entire system, not including additional projects completed to address specific deficiencies or service area growth. A rehabilitation rate higher than 1.0 percent per year is advisable to keep the average collection system pipe age to less than 100 years. When selecting sewer mains to be included in the annual rehabilitation program, a number of factors should be considered:

- Age and material of the existing pipe – preference should be given to piping over 50 years old using asbestos cement and vitrified clay piping.
- Depth and proximity to the Clark Fork River – rehabilitation in this area is anticipated to be most effective in combating infiltration.
- Other street projects – combining work below the surface with street resurfacing projects keeps overall project costs lower.
- Capacity concerns identified in this report – as verified by field measurements.

6.6. SUMMARY OF RECOMMENDED IMPROVEMENTS AND COSTS

The following table summarizes all recommended improvements in order of priority. In some cases, further study may be beneficial to better determine local flow conditions and develop additional site specific solutions to noted deficiencies. The cost estimates associated with the recommendations are based on high level budgetary cost estimates from manufacturers and known scaled costs from past construction projects. Estimates correspond to the AACE International definitions for Class 3 and Class 4/5 cost estimates. Class 3 estimates include a 25 percent contingency and Class 4/5 estimates include a 65 percent contingency. More detail on the AACE International definitions for cost estimating is presented in Chapter 8. Detailed cost estimates are included in Appendix 6-1.

Table 6-6: Summary of Recommended Improvements and Associated Costs

Recommendation	Cost [1]
Near-Term	
Momont No. 2 Force Main Realignment [2]	$1,301,000
Gravity Main Upsizing Upstream of Caras Park Lift Station	$994,000
Gravity Main Upsizing Upstream of Momont No. 1 Lift Station	$889,000
Gravity Main Upsizing on Lower Miller Creek Rd. and Linda Vista Blvd.	$193,000
Long-Term	
Connect Linda Vista Lift Station Force Main to Reserve Street Lift Station Force Main [3]	$39,000
Gravity Main Upsizing between Momont No. 1 and No. 2 Lift Stations	$903,000
30-inch Interceptor Upsizing	$927,000
36-inch Interceptor Upsizing	$2,198,000
24th Avenue Gravity Main Upsizing	$502,000
Mastad Lift Station Capacity Modification (480 gpm) [4]	$531,000
Kelly Island Lift Station Capacity Modification (1,025 gpm) [4]	$648,000
Momont No. 2 Lift Station Capacity Modification (1,500 gpm) [5, 3]	$520,000

[1] Construction costs only, based on 2019 dollars and Class 5 cost estimates that include a 65% contingency except as noted.
[2] Grant Creek Lift Station Capacity Modification is negated by recommended improvement
[3] Cost based on Class 3 cost estimates that include a 25% contingency.
[4] Cost for replacement pumps, lining wet well, new generator, upgraded mechanical, electrical, and controls.
[5] Based on preliminary construction cost estimate associated with Momont No. 2 design proposal. Cost includes engineering, bidding, and construction administration fee.

APPENDIX 6-1

COLLECTION SYSTEM RECOMMENDED ALTERNATIVE COST ESTIMATES

MISSOULA WASTEWATER FACILITY PLAN - COLLECTION SYSTEM
Probable Capital Cost for Recommended Alternatives
July 2019

Re-Align Momont #2 Force Main to MH P02-3-N31A in West Broadway

Item				Cost
12" C-900 PVC Pipe	4,214	LF	$	316,000
20" Steel Case, Bore & Jack	430	LF	$	258,000
Road Approach Crossing	2	EA	$	20,000
Asphalt Pavement Surface Restoration	1,054	LF	$	79,000
Other Surface Restoration	3,161	LF	$	47,000
Connection to Existing Manhole	1	LS	$	3,000
Subtotal			**$**	**723,000**
General Conditions			$	108,000
Contingency [1]			$	470,000
Total			**$**	**1,301,000**

Upsize Gravity Mains Upstream of Caras Park Lift Station

Item				Cost
18" ASTM F679 PVC Sewer Pipe	3,314	LF	$	331,000
48" Manholes	13	EA	$	78,000
Asphalt Pavement Surface Restoration	663	LF	$	50,000
Other Surface Restoration	2,651	LF	$	40,000
Dewatering	1	LS	$	10,000
Bypass Pumping Setup/Teardown	1	LS	$	8,000
Bypass Pumping (800 gpm)	7	WKS	$	35,000
Subtotal			**$**	**552,000**
General Conditions			$	83,000
Contingency [1]			**$**	**359,000**
Total			**$**	**994,000**

Upsize Gravity Mains Upstream of Momont #1

Item				Cost
15" ASTM F679 PVC Sewer Pipe	2,642	LF	$	198,000
48" Manholes	10	EA	$	60,000
Asphalt Pavement Surface Restoration	2,642	LF	$	198,000
Bypass Pumping Setup/Teardown	1	LS	$	8,000
Bypass Pumping (600 gpm)	6	WKS	$	30,000
Subtotal			**$**	**494,000**
General Conditions			$	74,000
Contingency [1]			**$**	**321,000**
Total			**$**	**889,000**

Upsize Lower Miller Creek Road and Linda Vista Boulevard Gravity Mains

Item				Cost
18" ASTM F679 PVC Sewer Pipe	385	LF	$	39,000
48" Manholes	4	EA	$	24,000
Asphalt Pavement Surface Restoration	385	LF	$	29,000
Bypass Pumping Setup/Teardown	1	LS	$	10,000
Bypass Pumping (1,500 gpm)	1	WKS	$	5,000
Subtotal			**$**	**107,000**
General Conditions			$	16,000
Contingency [1]			**$**	**70,000**
Total			**$**	**193,000**

Connect 14" Linda Vista Golf Course Force Main to 20" Reserve Street Force Main

Item				Cost
20" MJ Ductile Iron Wye	1	EA	$	5,000
20"x14" Reducer	1	EA	$	2,000
14" Buried Plug Valve	2	EA	$	10,000
14" MJXMJ Ductile Iron Wye	1	EA	$	4,000
20" Flexible Coupling	1	EA	$	2,000
14" Flexible Coupling	1	EA	$	2,000
14" C-900 PVC Pipe	1	LS	$	2,000
Landscape Surface Restoration	1	LS	$	1,000
Subtotal			**$**	**28,000**
General Conditions			$	4,000
Contingency [2]			**$**	**7,000**
Total			**$**	**39,000**

Upsize Gravity Sewer Main between Momont #1 and Momont #2

Item				Cost
18" ASTM F679 PVC Sewer Pipe	2,742	LF	$	274,000
48" Manholes	11	EA	$	66,000
Asphalt Pavement Surface Restoration	1,371	LF	$	103,000
Other Surface Restoration	1,371	LF	$	21,000
Bypass Pumping Setup/Teardown	1	LS	$	8,000
Bypass Pumping (1,000 gpm)	6	WKS	$	30,000
Subtotal			**$**	**502,000**
General Conditions			$	75,000
Contingency [1]			**$**	**326,000**
Total			**$**	**903,000**

1. This is a Class 5 cost estimate and a 65% contingency was applied to account for unknown future bidding climates, changes in material costs, details not included, and other unknowns. Costs are expressed in 2019 dollars.
2. This is a Class 3 cost estimate and a 25% contingency was applied to account for unknown future bidding climates, changes in material costs, details not included, and other unknowns. Costs are expressed in 2019 dollars.

Upsize 30-inch Interceptor in Clark Fork Drive

Item				Cost
36" ASTM F679 PVC Sewer Pipe	1,449	LF	$	290,000
60" Manholes	6	EA	$	45,000
Asphalt Pavement Surface Restoration	725	LF	$	54,000
Other Surface Restoration	725	LF	$	11,000
Dewatering	1	LS	$	25,000
Shoring (Deep Excavation)	1	LS	$	15,000
Bypass Pumping Setup/Teardown	1	LS	$	15,000
Bypass Pumping (10,000 gpm)	6	WKS	$	60,000
Subtotal			**$**	**515,000**
General Conditions			$	77,000
Contingency [1]			**$**	**335,000**
Total			**$**	**927,000**

Upsize 36-inch Interceptor Crossing Clark Fork River

Item				Cost
42" ASTM F679 PVC Sewer Pipe	1,132	LF	$	283,000
48" Jack and Bore/Directional Drill	500	LF	$	625,000
72" Manholes	5	EA	$	55,000
Asphalt Pavement Surface Restoration	113	LF	$	8,000
Other Surface Restoration	1,019	LF	$	15,000
Dewatering	1	LS	$	100,000
Shoring (Deep Excavation)	1	LS	$	50,000
MDT Coordination (Reserve Street)	1	LS	$	20,000
Bypass Pumping Setup/Teardown	1	LS	$	15,000
Bypass Pumping (13,000 gpm)	5	WKS	$	50,000
Subtotal			**$**	**1,221,000**
General Conditions			$	183,000
Contingency [1]			**$**	**794,000**
Total			**$**	**2,198,000**

Upsize 24th Avenue Gravity Mains between Briggs Street and 39th Street

Item				Cost
18" ASTM F679 PVC Sewer Pipe	1,210	LF	$	121,000
48" Manholes	7	EA	$	42,000
Asphalt Pavement Surface Restoration	1,210	LF	$	91,000
Bypass Pumping Setup/Teardown	1	LS	$	10,000
Bypass Pumping (1,300 gpm)	3	WKS	$	15,000
Subtotal			**$**	**279,000**
General Conditions			$	42,000
Contingency [1]			**$**	**181,000**
Total			**$**	**502,000**

Mastad Lift Station Capacity Modification (480 gpm)

Item				Cost
Site Work	1	LS	$	40,000
Bypass Pumping	1	LS	$	30,000
Concrete	1	LS	$	5,000
Replacement Pumps	1	LS	$	80,000
Pipe and Mechanical	1	LS	$	40,000
Wet Well Lining	1	LS	$	20,000
Generator	1	LS	$	30,000
Electrical and Controls	1	LS	$	50,000
Subtotal			**$**	**295,000**
General Conditions			$	44,000
Contingency [1]			$	192,000
Total			**$**	**531,000**

Kelly Island Lift Station Capacity Modification (1,025 gpm)

Item				Cost
Site Work	1	LS	$	50,000
Bypass Pumping	1	LS	$	40,000
Concrete	1	LS	$	5,000
Replacement Pumps	1	LS	$	100,000
Pipe and Mechanical	1	LS	$	50,000
Wet Well Lining	1	LS	$	24,500
Generator	1	LS	$	35,000
Electrical and Controls	1	LS	$	55,000
Subtotal			**$**	**359,500**
General Conditions			$	54,000
Contingency [1]			$	234,000
Total			**$**	**648,000**

Momont No. 2 Lift Station Capacity Modification (1,500 gpm)

Item				Cost
Site Work	1	LS	$	56,500
Bypass Pumping	1	LS	$	10,000
Concrete	1	LS	$	45,000
Replacement Pumps	1	LS	$	120,000
Pipe and Mechanical	1	LS	$	55,000
Wet Well Lining	1	LS	$	24,500
Generator	1	LS	$	-
Electrical and Controls	1	LS	$	60,000
Subtotal			**$**	**371,000**
General Conditions			$	56,000
Contingency [2]			$	93,000
Total			**$**	**520,000**

1. This is a Class 5 cost estimate and a 65% contingency was applied to account for unknown future bidding climates, changes in material costs, details not included, and other unknowns. Costs are expressed in 2019 dollars.
2. This is a Class 3 cost estimate and a 25% contingency was applied to account for unknown future bidding climates, changes in material costs, details not included, and other unknowns. Costs are expressed in 2019 dollars.

THE GARDEN CITY
MISSOULA
HUB OF FIVE VALLEYS

WASTEWATER FACILITY PLAN

CHAPTER 7 - EXISTING TREATMENT PLANT DESCRIPTION AND CAPACITY ASSESSMENT

Morrison Maierle
engineers • surveyors • planners • scientists

CHAPTER 7 EXISTING TREATMENT PLANT DESCRIPTION AND CAPACITY ASSESSMENT

7.1. INTRODUCTION

The Missoula wastewater treatment plant (WWTP) is located in the northern part of the City on the banks of the Clark Fork River. It receives the City's wastewater via two gravity trunk mains. A 36-inch main crosses the river immediately adjacent to the WWTP and a 30-inch main reaches the plant from the north on the same side of the river. Treated and disinfected effluent is discharged to the Clark Fork River. Figure 7-1 shows an overall site plan noting the locations of all major structures.

This chapter presents a history of the WWTP, process flow diagram and hydraulic profile, and descriptions of the existing unit processes. An assessment of each unit process is provided for condition of the existing equipment, performance of the unit process, and its capacity.

7.2. HISTORY OF THE EXISTING WASTEWATER TREATMENT FACILITY

The Missoula WWTP was initially designed in 1961 to provide grit removal, primary clarification, disinfection (chlorination), anaerobic sludge digestion and sludge drying facilities. In 1973, the plant was upgraded to a conventional activated sludge plant with a design capacity of 9.0 mgd. In 1984, the headworks facility was replaced and a new final clarifier was added. In 1986, the aeration system was upgraded from a mechanically aerated, coarse-bubble diffusion system to a fine-bubble aeration system employing ceramic disk diffusers. In 1995 several pump stations were modified and in 1996, a new belt filter press was added to the two existing units. The design flow was 8.99 mgd.

In 1994, the operating staff began operating the activated sludge basins with an anaerobic zone to encourage biological phosphorus removal. The anaerobic zone (25 percent of process volume) was created by turning off the air supply to the first of the four cells, in each of the two bioreactors. Return activated sludge (RAS) continued to be pumped to this first anaerobic cell with the remainder of the existing tanks operating as aerobic zones to provide BOD reduction and limited nitrification.

In 2003 and 2004, the WWTP was improved and expanded to provide biological nutrient removal (BNR) and increase capacity. Additions included construction of a new treatment train consisting of two bioreactor trains and three secondary clarifiers, as well as modifications to the existing bioreactor basins that would allow for BNR in all bioreactors. Other process and equipment upgrades were completed to replace aging systems, provide added capacity, or take advantage of newer technology and better efficiencies. The upgraded average day plant capacity was and still is 12.0 mgd with a maximum month design flow of 13.8 mgd.

In 2006, one of the existing three belt filter presses was replaced with a high volume centrifuge dewatering system. Ten years later, a volute press was purchased to replace the remaining belt filter presses for solids dewatering. The most recent major upgrades in 2011 replaced the Headworks Building for the second time in the history of the plant, added influent flow metering, and improved effluent flow metering.

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DATE: 10/2018

MISSOULA WASTEWATER FACILITY PLAN

MISSOULA MONTANA

OVERALL SITE PLAN

PROJECT NO. 1657.039.800

FIGURE NUMBER 7-1

R:\1657\039 Wastewater Facility Plan\ACAD\Exhibits\FIG 7-1.dwg Plotted by jerry a. chambers on Apr/22/2019

In a process initiated in 2007, the City explored and subsequently invested in a hybrid poplar plantation that takes up to 1.5 mgd of effluent flow during the growing season, effectively removing the associated nutrient load from the river. Another environmentally friendly investment included the installation of a co-generation facility that can convert digester gas to electricity and help supply the WWTP.

In 2018, the City finalized its purchase of EKO Compost, a composting facility located to the west of the plant. The plant has sent all of its biosolids to the facility for decades. City ownership will ensure that cost effective solids handling continues to provide full treatment by a combination of digestion, aerated storage and composting. The following graphic summarizes the history of the WWTP from its original construction to the present.

1961

Engineer: Clark and Groff

ORIGINAL PLANT
- Headworks
- Primary clarifiers (2)
- Digesters (2) and control building
- Sludge drying beds

Most facilities modified or replaced with subsequent projects

1973

Engineer: Clark and Groff

SECONDARY PLANT UPGRADE/EXPANSION
- Secondary lift pump station
- Aeration tanks/tunnels
- Blower building
- DAFs
- Lab
- Secondary clarifiers (2)
- Chlorine contact tank
- Primary clarifier (1 new)

Complete mix AB basins. Coarse bubble/turbine mixer aeration system– subsequently retrofitted with fine bubble. Secondary clarifiers modified with subsequent project

1982

Engineer: James M. Montgomery

SOLIDS HANDLING UPGRADES
- New primary digester
- Belt filter presses (2)
- Existing digester mods

1983

Engineer: James M. Montgomery

BOILER SYSTEM MODIFICATIONS

1984

Engineer: James M. Montgomery

HEADWORKS REPLACEMENT
- New influent pumps, screens & grit
- Secondary Clarifier (1 new)
- Replaced Generator

1986

Engineer: James M. Montgomery

AERATION SYSTEM UPGRADES

- Aeration basin mods
- Fine bubble diffusers
- Blower System modifications

Converted AB to current plug-flow configuration. Installed fine bubble diffusers

1988

Engineer: James M. Montgomery

SECONDARY CLARIFIER MODIFICATIONS

Converted secondary clarifiers to center feed

1995

Engineer: James M. Montgomery

DIGESTER SYSTEM MODIFICATIONS

Interior and exterior mods. Insulation

1995

Engineer: Robert Peccia Assoc.

PUMP STATION MODIFICATIONS

- Secondary lift station mods.
- Primary pump mods
- DAF pump mods

1995

Engineer: James M. Montgomery

CHLORINE SYSTEM MODIFICATIONS

Interior and exterior mods. Insulation

1996

Engineer: Robert Peccia Assoc.

BELT PRESS UPGRADE

- New belt press (1)

Added unit within existing building. Total of 3 units

1998

Engineer: OZ Architects

ADMINISTRATION BUILDING ADDITION AND MODIFICATIONS

Building upgraded. New offices and Laboratory

2002

Engineer: Morrison-Maierle

DRY POLYMER FEED EQUIPMENT UPGRADES

- Replacement of existing equipment
- New pumps for digested sludge (4)

2002

Engineer: Morrison-Maierle

PRIMARY EFFLUENT PIPE REPLACEMENT

Installation of new piping between the primary effluent lift station and the aeration basin influent box

2004

Engineer: Morrison-Maierle

HEADWORKS MODIFICATIONS

- Screenings washer/compactor
- Grit pumps (4)
- Grit classifier/washer

Grit removal and screenings processing improvements

2004

Engineer: Morrison-Maierle

WASTEWATER TREATMENT PLANT UPGRADE

- Primary effluent lift station expansion
- Elutriation water pumps (3)
- Secondary treatment modification and expansion
  - Bioreactors (2 new, 2 existing)
  - Aeration system improvements
- Clarifiers (3 new)
- Primary sludge fermenter
- Dewatering facilities modifications
- UV disinfection system

Conversion and expansion of secondary treatment bioreactors and associated equipment, to achieve biological nutrient removal at an average day design capacity of 12 mgd

2006

Engineer: Morrison-Maierle

SOLIDS HANDLING IMPROVEMENTS

- Centrifuge (1)
- Polymer feed pumps (4)
- Sludge transfer pumps (2)

Replacement of one belt filter press with a high capacity centrifuge. Existing pumps were replaced with higher capacity units to accommodate the higher capacity of the centrifuge

2006

Engineer: Rocking M Design

TRUCK BARN

Building for vehicle and equipment storage

2010

Engineer: Morrison-Maierle/Stantec/Rocking M Design

NEW HEADWORKS BUILDING

- Headworks building
- Screens
- Influent pumps
- Exterior grit removal
- Effluent flow metering improvements

2007-2013

Engineer: Morrison-Maierle

HYBRID POPLAR PLANTATION

- Pilot Project and Expansion

65 lb/d of TN and 4.7 lb/d of TP diverted from Clark Fork River to poplar plantation

2016

Engineer: HDR

VOLUTE PRESS

Replacing existing belt filter presses and allowing for better side stream equalization

2016-2018

Engineer: Anderson-Montgomery

PURCHASE OF EKO COMPOST AND TRANSITIONAL FACILITY

This will allow solids handling to continue to provide full treatment by a combination of digestion, aerated storage and composting

2018

Engineer: Morrison-Maierle

CO-GENERATION FACILITY

Use of digester gas for generation of electricity

7.3. WASTEWATER TREATMENT PLANT

7.3.1. Description

The Missoula WWTP is a conventional secondary treatment facility with biological nutrient removal (BNR) capability that utilizes an activated sludge process. Treatment of the wastewater generated by the City begins in the headworks facility where the wastewater is screened and degritted before flowing to the primary clarifiers for initial settling. Following primary clarification, primary effluent is pumped to two sets of bioreactors. A sequence of anaerobic, anoxic, and fully aerated cells in the bioreactors facilitate removal of BOD, nitrogen, and phosphorus. Mixed liquor from the bioreactors flows to secondary clarifiers where the suspended solids settle to the bottom and treated effluent flows over the weir and to the UV disinfection system. Disinfected effluent is discharged to the Clark Fork River.

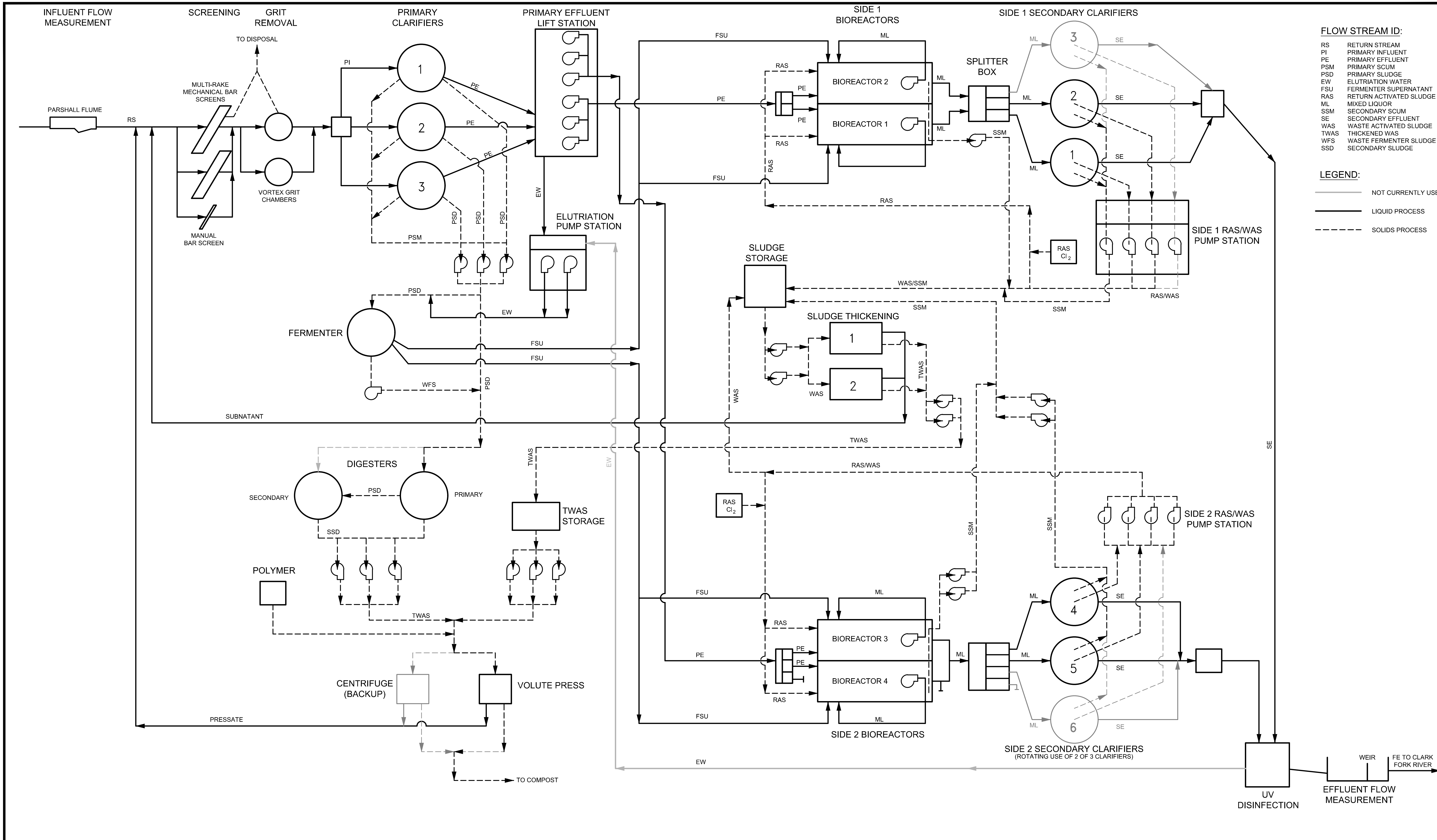
The solids process train in the treatment plant starts with the sludge that is collected from the primary clarifiers. Primary sludge is pumped directly to the fermenter and then to anaerobic digesters. The fermenter provides volatile fatty acids (VFAs) for the biological phosphorous removal process and reduces sludge volume. The anaerobic digesters further reduce sludge volume and provide sludge stabilization and pathogen reduction.

A portion of the sludge collected from the secondary clarifiers is pumped as return activated sludge (RAS) to the bioreactors while a smaller portion, the waste activated sludge (WAS) is pumped to a dissolved air floatation thickening unit (DAF). The thickened WAS (TWAS) is pumped to an aerated storage tank. TWAS and digested primary sludge are pumped in batches to the dewatering facility. The dewatered solids are transported via a conveyor belt to the neighboring composting facility for further processing. Figure 7-2 shows the process flow diagram for all liquid and solids handling process streams.

7.3.2. Condition, Performance, and Capacity

The WWTP is well-maintained and consistently operating as intended. The following list presents an overall summary of the current condition, performance, and capacity of the plant. The following sections present descriptions and condition, performance, and capacity assessments of all major plant process steps and equipment. General conclusions are:

- Missoula WWTP staff has an excellent record of proactive plant maintenance resulting in equipment longevity.
- The service life for most existing equipment is expected to last through the planning period or beyond with good maintenance and service.
- Plant staff monitor equipment and evaluate replacement needs on an ongoing basis.
- Plant performance overall has been very consistent and in compliance with MPDES permit limits. In the five years of data used for this analysis, only four occasions of permit violations were noted a WET test, pH, *E. coli*, and ammonia.
- Current design capacity of the plant is 12.0 mgd for average day.
- 2037 projected average day flow is 11.2 mgd.



NOTE: PROCESS FLOW SCHEMATIC IS FOR ILLUSTRATION ONLY AND IS NOT TO BE USED FOR CONSTRUCTION. ONLY PRIMARY AND MAJOR SECONDARY PROCESS FLOW PATTERNS ARE SHOWN.

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DATE: 10/2018

MISSOULA WASTEWATER FACILITY PLAN

MISSOULA MONTANA

PROCESS FLOW DIAGRAM

PROJECT NO. 1657.039.800

FIGURE NUMBER 7-2

R:\1657\039 Wastewater Facility Plan\ACAD\Exhibits\FIG 7-2.dwg Plotted by jerry a. chambers on Apr/8/2019

- Projected maximum day and peak hour flows are higher than those projected in 2001 when the plant design criteria were developed.
- Treatment for nutrients at current permit limits would become difficult at average flows over 9.2 mgd.
- Projected permit limits for nutrients are lower than current limits which may affect overall plant treatment capacity.
- Hydraulically, the plant could likely handle 2037 average and maximum month flows but would be challenged at projected 2037 maximum day and peak hour flows.

7.4. HYDRAULIC PROFILE

Figures 7-3 – 7-5 show the hydraulic profile through the plant. The hydraulic profile follows the liquid stream from influent to effluent and lists water surface elevations for 2037 maximum month and peak hour flow conditions. Elevations shown are based on the plant datum that has been used for design/construction projects over the past 20 years. An Excel-based hydraulic profile calculator was used to model water surface elevations throughout the plant. The calculator uses a number of conservative assumptions and results in a slightly conservative approach appropriate for planning and design of new projects.

7.4.1. Hydraulic Profile Calculator Input

Assumptions made for the calculation of the hydraulic profile include the following:

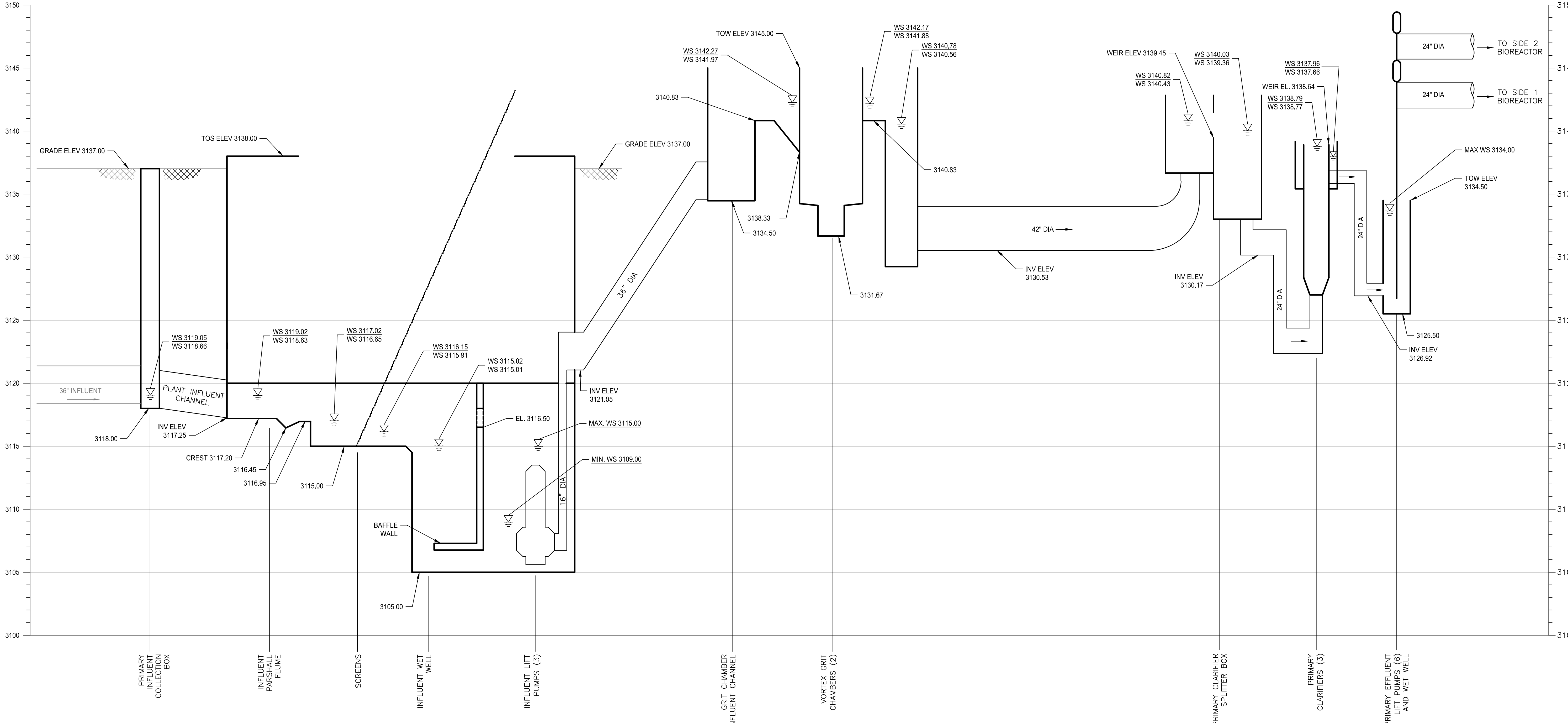
- Plant design flows for average day and peak hour taken from 2002 WWTP Record Drawings
- Elevations based on 2002 WWTP Record Drawings and 2012 Headworks Record Drawings
- Starting water surface elevation: 100-yr flood elevation per 2002 WWTP As-Bid Drawings
- Current equipment, pipe sizes, and plant configuration used for all modeled scenarios
- Maximum wet well level used for all pump stations
- Pump capacity evaluated separately; no use of redundant pumps and associated piping
- Two of three Side 1 secondary clarifiers in use, including largest unit
- Two Side 2 secondary clarifiers in use for 2017 flows; three secondary clarifiers in use for 2037 flows
- RAS flow: 0.95Q, capped at RAS pump station firm capacities of 3.7 mgd and 6.4 mgd for Side 1 and Side 2 bioreactors, respectively
- Flow split between Side 1 and Side 2 bioreactors: 37% and 63%, respectively, based on data; up to maximum flows for the Side 1 bioreactor: 5.5 mgd average, 6.0 mgd maximum month, 6.8 mgd maximum day, 8.3 mgd peak hour; all flows in excess of Side 1 bioreactor maximum capacity routed to Side 2 bioreactor
- Three primary clarifiers in use for all scenarios
- Two influent screens in service for all scenarios

ASSUMPTIONS:

- CLARK FORK RIVER AT 100 YEAR FLOOD ELEVATION
- FLOW SPLIT AT PRIMARY EFFLUENT LIFT STATION: 63% TO NEW SECONDARY PROCESS
  37% TO EXISTING SECONDARY PROCESS
- RAS FLOW RATE = 95%, CAPPED AT 3.7 mgd FOR SIDE 1 AND 6.4 mgd FOR SIDE 2.
- 2017: 2 SIDE 2 CLARIFIERS
- 2037: 3 SIDE 2 CLARIFIERS

KEY:

WS 3144.43	WS ELEVATION FOR 2037 PEAK HOUR FLOW (26.6 mgd)
WS 3144.58	WS ELEVATION FOR 2017 PEAK HOUR FLOW (18.2 mgd)



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MISSOULA WASTEWATER FACILITY PLAN

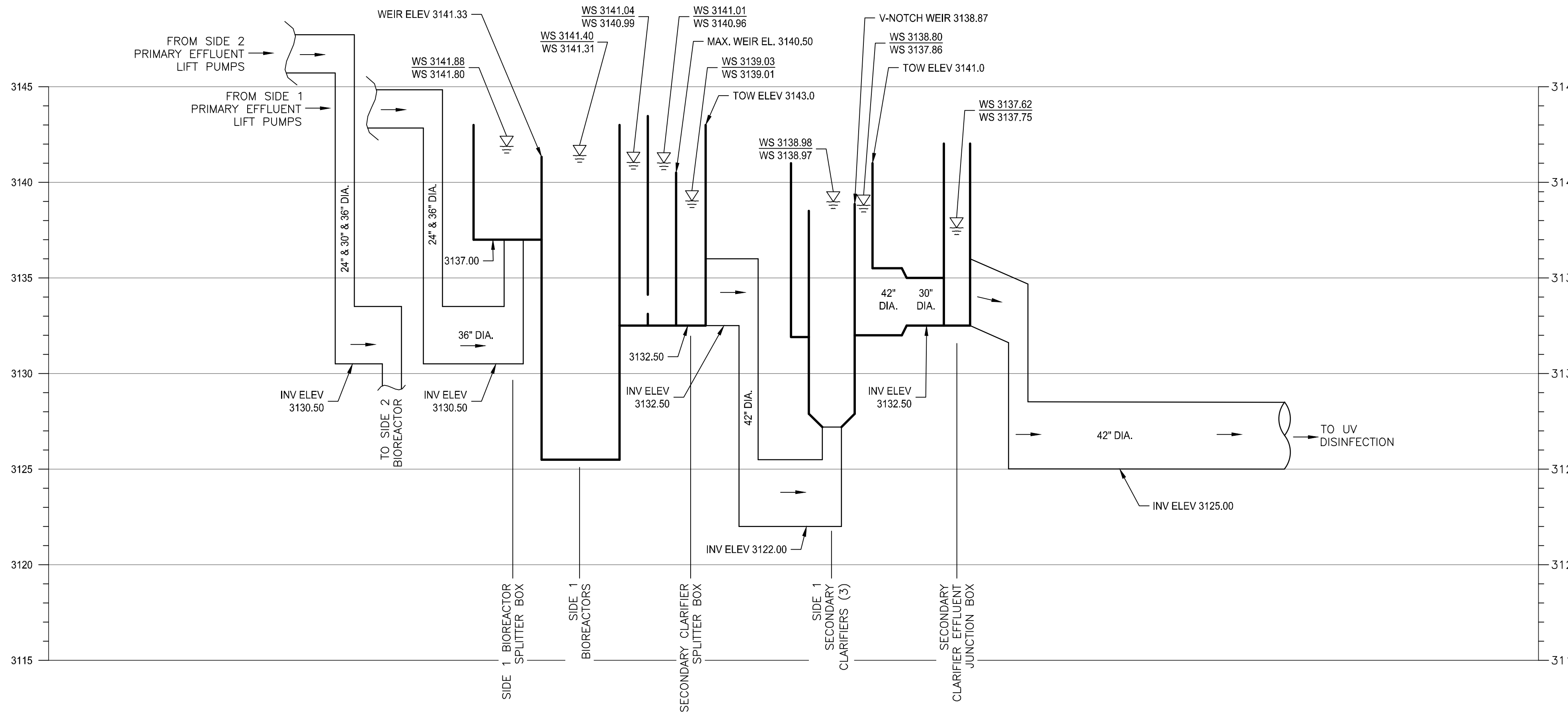
MISSOULA MONTANA

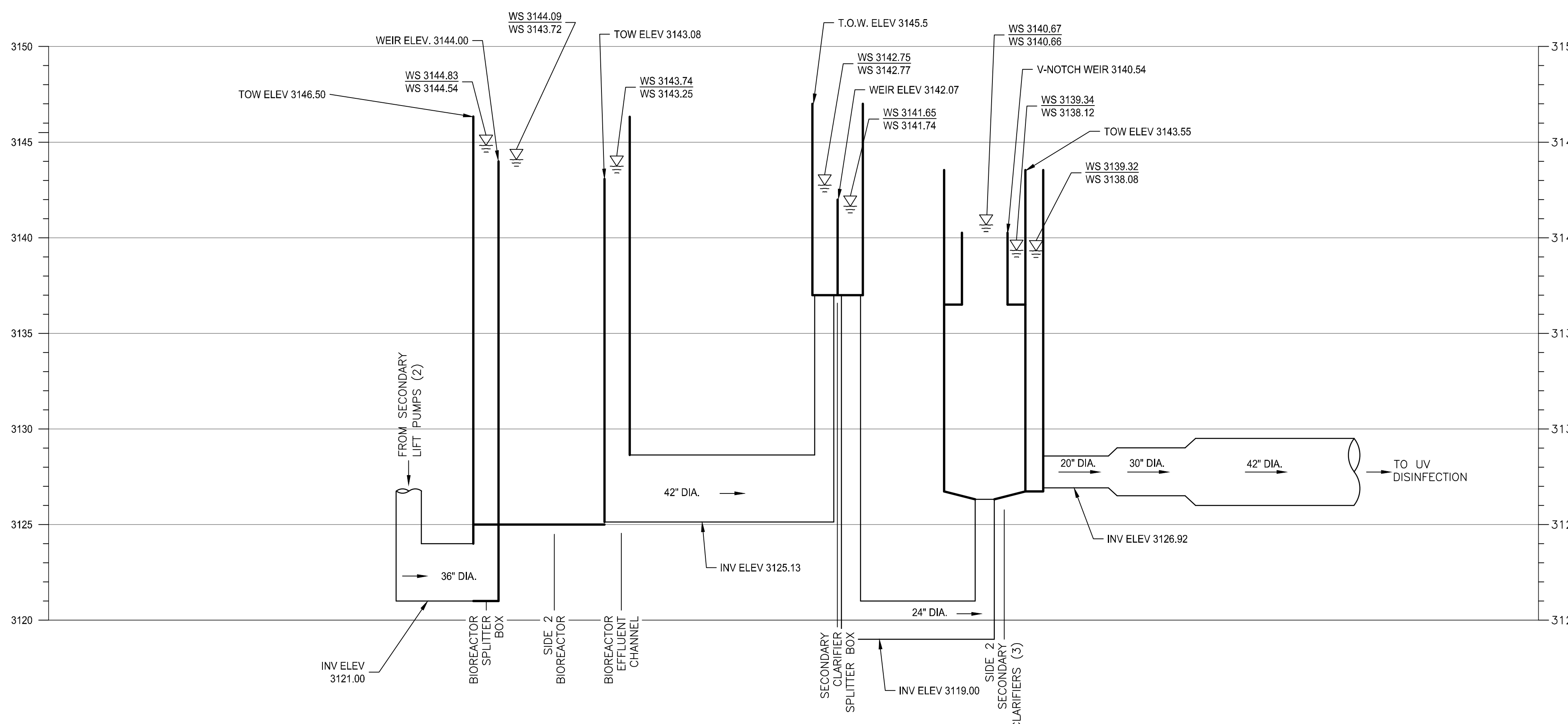
HYDRAULIC PROFILE - INFLUENT TO GRIT BASIN EFFLUENT

PROJECT NO. 1657.039.800

FIGURE NUMBER 7-3

R:\1657\039 Wastewater Facility Plan\ACAD\Exhibits\FIG 7-3.dwg Plotted by jerry a. chambers on Apr/8/2019





ASSUMPTIONS:

- CLARK FORK RIVER AT 100 YEAR FLOOD ELEVATION
- FLOW SPLIT AT PRIMARY EFFLUENT LIFT STATION: 63% TO NEW SECONDARY PROCESS 37% TO EXISTING SECONDARY PROCESS
- RAS FLOW RATE = 95%, CAPPED AT 3.7 mgd FOR SIDE 1 AND 6.4 mgd FOR SIDE 2.
- 2017: 2 SIDE 2 CLARIFIERS
- 2037: 3 SIDE 2 CLARIFIERS

KEY:

WS 3144.43	WS ELEVATION FOR 2037 PEAK HOUR FLOW (26.6 mgd)
WS 3144.58	WS ELEVATION FOR 2017 PEAK HOUR FLOW (18.2 mgd)

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MISSOULA WASTEWATER FACILITY PLAN

MISSOULA MONTANA

HYDRAULIC PROFILE - PRIMARY EFFLUENT SPLITTER BOX TO SECONDARY CLARIFIER EFFLUENT

PROJECT NO. 1657.039.800

FIGURE NUMBER 7-4

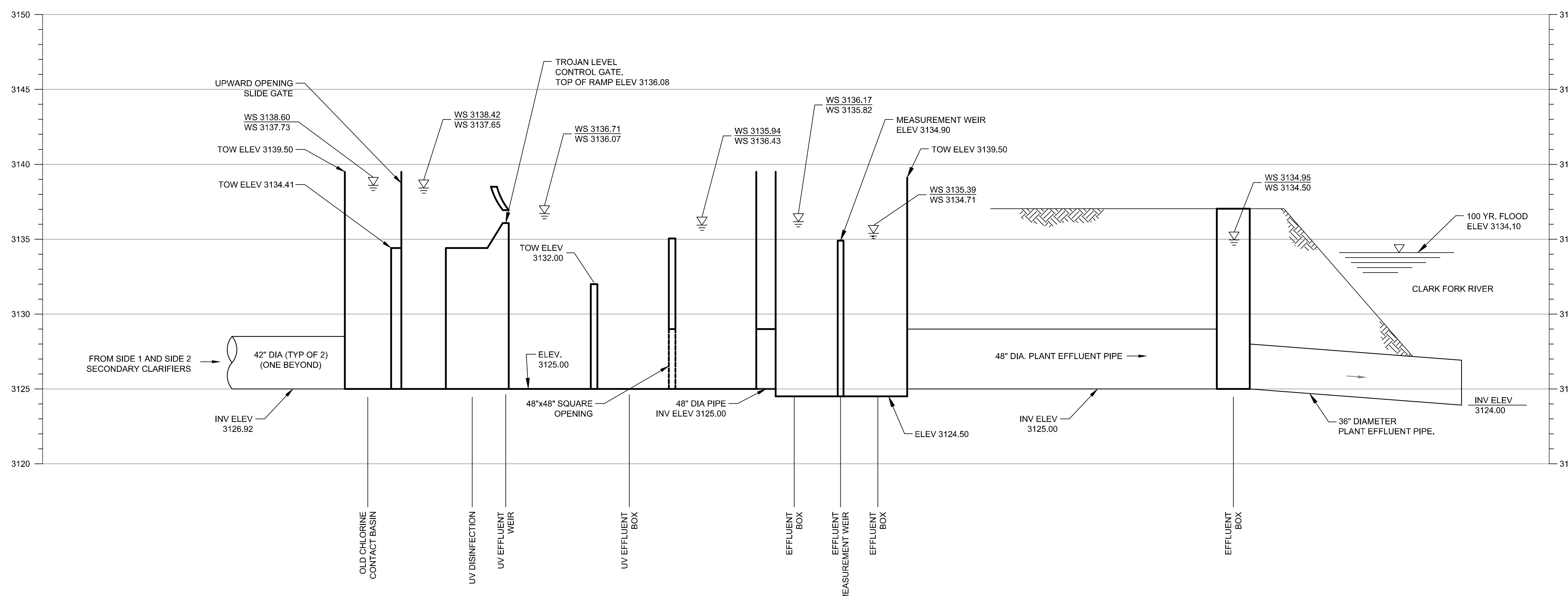
R:\1657\039 Wastewater Facility Plan\ACAD\Exhibits\FIG 7-4.dwg Plotted by jerry a. chambers on Apr/9/2019

ASSUMPTIONS:

- CLARK FORK RIVER AT 100 YEAR FLOOD ELEVATION
- FLOW SPLIT AT PRIMARY EFFLUENT LIFT STATION: 63% TO NEW SECONDARY PROCESS
  37% TO EXISTING SECONDARY PROCESS
- RAS FLOW RATE = 95%, CAPPED AT 3.7 mgd FOR SIDE 1 AND 6.4 mgd FOR SIDE 2.
- 2017: 2 SIDE 2 CLARIFIERS
- 2037: 3 SIDE 2 CLARIFIERS

KEY:

WS 3144.43	WS ELEVATION FOR 2037 PEAK HOUR FLOW (26.6 mgd)
WS 3144.58	WS ELEVATION FOR 2017 PEAK HOUR FLOW (18.2 mgd)



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MISSOULA WASTEWATER FACILITY PLAN

MISSOULA MONTANA

PROJECT NO. 1657.039.800

HYDRAULIC PROFILE - UV SYSTEM TO OUTFALL

FIGURE NUMBER 7-5

R:\1657\039 Wastewater Facility Plan\ACAD\Exhibits\FIG 7-5.dwg Plotted by jerry a. chambers on Apr/9/2019

The following flow scenarios were calculated:

Table 7-1: Plant Flow Summary

Condition	2017 [1]	2037 [1]	Design[2]
Average, mgd	7.27	11.2	12.0
Peak Hour, mgd	18.22	26.6	19.2

1. See Chapter 2.
2. As listed in 2002 As-Bid Drawings.

The calculations of the hydraulic profile were performed in three segments, as follows:

- Segment 1: Outfall to UV Disinfection System
- Segment 2A: UV System to Side 1 Primary Lift Pumps
- Segment 2B: UV System to Side 2 Primary Lift Pumps
- Segment 3: Primary Lift Station Wet Well to Plant Influent

7.4.2. Overall Hydraulic Capacity

The existing plant's overall hydraulic design capacity is about 20 mgd with exact capacities of individual processes and piping segments varying slightly. This overall capacity was tested in the spring and early summer of 2018 during the highest peak flows experienced during the tenure of current plant staff. Only very few plant areas showed capacity issues during this high flow event and are described further below.

Hydraulic profile calculator output for all scenarios is included in Appendix 7-1. Results confirm that the plant is expected to have adequate capacity for flows up to about 20.0 mgd but may experience hydraulic capacity restrictions in several locations at 2037 peak hour flows (26.6 mgd). The graphic depiction of current, 2037, and original design flows in Figure 7-6 illustrates how the projected 2037 peak flows would exceed known design capacities. Note that even though projected 2037 average flows are less than the currently stated design capacity of the plant, projected maximum day and peak hour flows are higher than current design values. As explained in Chapter 2, the 2037 maximum month and maximum day flows are data-based projections, while the peak hour flow is a calculated value strictly based on the projected 2037 population. If population growth in Missoula is slower than projected, resulting actual peak hour flows may be lower; however, the recent high flows of 2018 and general predictions connected to climate change and associated extreme weather events may present good reasons for conservative planning with respect to hydraulic capacity.

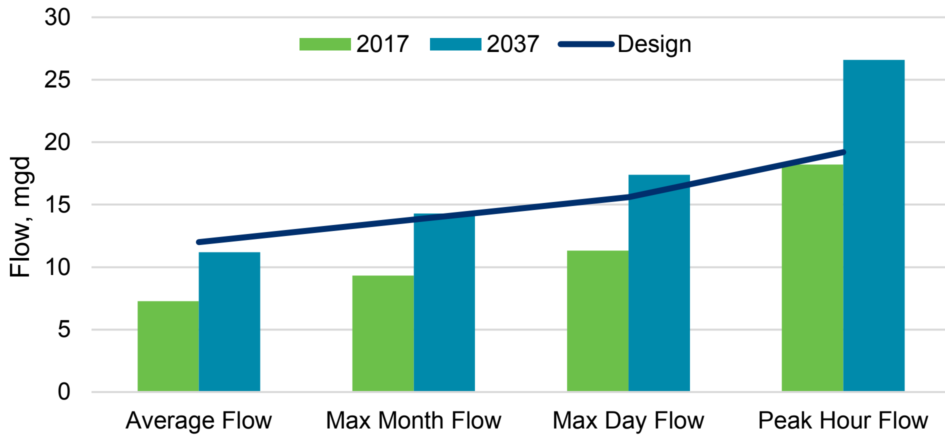


Figure 7-6: Plant Flows

It should be noted that hydraulic restrictions identified for 2037 flows are based solely on the mathematical approximation of flow through the plant. The calculator does not take into account limitations introduced by existing top of wall elevations of surrounding channels or basins or other features particular to the plant that limit allowable water levels or may cause spills; therefore, spot elevations were checked for all major basins and channels using the 2002 WWTP Record Drawings. Based on the calculations, the following restrictions will require further analysis and potential modification before flows reach the projected 2037 magnitudes.

Effluent Piping

The 36-inch effluent pipe from the effluent structure to the headwall is undersized for flows exceeding 21 mgd and introduces excessive headloss with potential to submerge the weir at the final effluent measurement structure (FEMS). Submergence of this weir would cause erroneous effluent flow readings. At the 2037 peak hour flow of 26.6 mgd, the pipe introduces seven inches more headloss than a 48-inch diameter pipe of the same material and almost 9 inches more than a 60-inch diameter pipe.

Side 1 Bioreactor and Secondary Clarifiers

Since the capacity of the Side 1 bioreactor side is set and increased flows would only be allowed to this bioreactor in an emergency, there are no significant capacity concerns. The calculations show the bioreactor splitter box weir submerged at the peak hour flow of 8.3 mgd. This submergence is based on the plan elevation for the weir. Raising the weir gate by as little as 1 inch would resolve the submergence without causing upstream hydraulic issues.

Side 2 Bioreactor and Secondary Clarifiers

Calculation results show that the bioreactor splitter box would be submerged at 2037 peak hour flows. Flow split would still be acceptable at these conditions, especially since by definition, it would only occur for one hour before receding. No other hydraulic restrictions were identified for the Side 2 bioreactor and clarifiers. As noted above, it was assumed that WWTP staff would bring all three clarifiers into routine service when flows approach the proposed 2037 magnitudes. Two clarifiers would not be able to adequately pass these higher flows.

Headworks, Primary Clarification, and Primary Lift Station

The only limited spot for this segment of the plant is the primary clarifier splitter box weir, which becomes submerged at 2037 peak hour flows. If the weir was raised about 6.5 to 7.0 inches, it would no longer be submerged and the model does not show upstream hydraulic issues with a higher weir elevation. Note that the modeled weir elevation is based on plan drawings and the actual weir elevation may have been changed already to accommodate higher flows. No other hydraulic pinch points were revealed in this segment of the plant.

Items Identified During Spring 2018 High Flows

During the spring of 2018, flood-related high influent flows spiked at over 20 mgd and stayed near the current plant design peak hour flow for days. During this time, the flow split into the Side 1 bioreactor was increased slightly to take some of the additional flow off the Side 2 bioreactor; however, the third Side 1 clarifier was still not needed. Otherwise, the only hydraulic limitation observed was the overflow wall between the primary effluent overflow basin (old chlorine contact chamber) and the UV system approach channel. Plant staff was forced to place sandbags on top of this wall to prevent disinfected effluent from flowing back into the overflow basin. Installation of an adjustable weir gate in this location may help manage extreme events like the 2018 flood but is not considered essential by plant staff at this time.

Another hydraulic issue identified during the 2018 high flows was in the UV disinfection system channel where water levels overflowed the UV ballast boxes of the upstream UV bank even with the level control gates lifted out of the flow. This was likely due to a combination of the hydraulic loss through the downstream bank and a hydraulic restriction in the channel geometry and level control gate width. Replacement of these gates with gates that have a higher hydraulic capacity and associated changes in channel geometry would help limit the water level rise in the UV channel at higher flows and better protect the ballast boxes.

7.4.3. Conclusions

- Overall, the existing plant configuration is expected to be able to hydraulically handle the projected 2037 influent flows.
- A few locations would require modification to prevent hydraulic issues at the 2037 peak hour flow.
- Adjustment of splitter box gates is a simple fix and may already have happened.
- Replacement of the effluent pipe and UV level control gates would require bypass pumping of the entire plant flow and potentially finding an alternate means of effluent disinfection during construction.
- If flows are expected to continue to increase past the 2037 planning horizon, a more thorough hydraulic analysis would be recommended to identify options for expanding the plant hydraulic capacity beyond 20 mgd.

7.5. FLOW MEASUREMENT

7.5.1. Description

Influent

- Parshall flume located in Headworks Building; 48-inch throat width,
- Ultrasonic flow element/indicator/ transmitter
- Installed in 2010

Effluent

- Rectangular weir, 11 feet wide with end contractions located in the final effluent flow measurement structure,
- Ultrasonic flow element/indicator/transmitter
- Installed in 2010

Table 7-2: Flow Measurement Equipment Summary

Unit Process and Parameter Description	Design Criteria or Capacity
Influent Flow Meter	
Type Size Flow Range	Parshall Flume 48" Throat width 0.9 – 43.9 mgd
Effluent Flow Meter	
Type Size Flow Range	Rectangular Weir, Contracted 11 ft 2.0 – >40 mgd

7.5.2. Condition, Performance, and Capacity

- Eight years old in good condition and performing as designed
- Expected design life of physical structures: in excess of 20 years
- Expected design life of electronics: typically 10 to 15 years
- Flow ranges of the physical structures: adequate beyond the planning period
- Flow ranges of the ultrasonic meters: may need adjustment for increased flow

7.5.3. Conclusions

- The equipment is relatively new and in good condition.
- The physical structures will not require improvements through the planning period and beyond.
- The condition of the electronic meters should be evaluated within the next 5 to 7 years to determine if replacement will be necessary.

- Capacity is not a concern – settings on ultrasonic equipment may need to be checked to verify that it can cover expected flow increases, especially for peak flow events.

7.6. HEADWORKS

7.6.1. Description

Raw wastewater that reaches the plant through the 30-inch and 36-inch diameter gravity sewers is combined in the influent collection box and routed to the influent screens via a buried plant influent channel and Parshall flume. The influent collection box also collects septage dumped at the adjacent septage receiving station and plant process return streams. Flow proceeds through two mechanically cleaned traveling rake bar screens prior to being pumped to the grit removal system. Washer/compactors rinse and compact the screenings before discharging to a dumpster. The bar screens, washer/compactors, dumpsters, and influent pumps are located in the Headworks Building; the grit removal system is located outside. The following summarizes the headworks facilities:

- All systems installed in 2011 – eight years old
- Two influent collection mains: 30-inch and 36-inch, combined in influent collection box
- Two mechanically cleaned, traveling rake bar screen: ¼-inch bar spacing
- One bypass channel with manually cleaned bar screen: 1-inch bar spacing
- Two dedicated washer/compactors
- Three equally sized influent pumps (2 duty, 1 standby)
- Two vortex grit chambers
- Two dedicated grit pumps plus one spare
- Two grit washers
- One screenings and grit dumpster
- Odor control system

Table 7-3 lists the headworks equipment and design criteria. The following paragraphs further describe major headworks components.

Table 7-3: Headworks Equipment Summary

Unit Process and Parameter Description	Design Criteria or Capacity
Mechanical Bar Screens	
Number of Screens	2
Type of Screens	Traveling Rake, Stainless Steel Bar Screens
Manufacturer	Duperon
Channel Width (each)	4.0 ft
Bar Spacing	3/8 in
Motor Size (each)	½ hp
Rated Screen Capacity (each)	12.5 mgd

Unit Process and Parameter Description	Design Criteria or Capacity
Washer/Compactors	
Number of Units	2
Type	Washer / Compactor
Manufacturer	Duperon
Capacity	60 ft^3/hr
Motor Size	0.75 hp
Dry Solids Content under Max Flow	60%
Volume Reduction	80%
Wash Water Requirements	3-5 gpm @ 40 psi
Influent Pump Station	
Number of Pumps	3 (2 duty, 1 standby)
Type of Pumps	End suction centrifugal
Manufacturer	ABS
Capacity (each)	7,500 gpm (observed ~6,000 gpm)
Static Lift	32-36 ft
Motor Size (each)	90 hp
Pump Station Firm Capacity (2 pumps)	20 mgd (observed ~14 mgd)
Grit Chambers	
Number of Basins	2
Type	Vortex
Diameter	12.0 ft
Capacity (each)	13 mgd
Grit Pumps	
Number of Pumps	2 duty, 1 spare
Type of Pumps	End Suction Centrifugal
Capacity (each)	250 gpm
Head (each)	16 ft
Horsepower (each)	10 hp
Grit Washer/Classifiers	
Number	2 (dedicated to each grit chamber)
Motor Size (each)	1 hp
Odor Control System	
Type	Photoionization
Manufacturer	Neutralox
Peak Inlet Concentration	10 ppm H_2S
System Performance	99% H_2S removal
Rated Capacity	4,550 cfm

Bar Screens and Washer/Compactors

The influent screening arrangement consists of two mechanically cleaned traveling rake screens with ¼-inch bar spacing. Screening the influent prior to lifting it to the grit removal system is protective of the influent pumps by reducing solids load to the pumps.

The traveling rake screens are made of vertical stainless steel bars evenly spaced across the channel. As flow passes through the bar screen, solids are deposited against the bars. The screen may be cleaning continuously or at intervals by multiple rake arms or scrapers mounted on a rotating chain drive. The rakes travel up against the screen and deadplate and dump the screenings behind the screen into dedicated screenings washer/compactor hoppers. The washer/compactors introduce water to wash the screenings in the hopper zone. An auger transports the screenings to the compacting/dewatering zone from which they discharge to a screenings dumpster.

Influent Pumps

The influent channel and influent screens are located over 20 feet below grade and screened influent is lifted to the grit removal system by three flooded suction centrifugal pumps. The pumps operate on variable speed drives (VFDs) to accommodate an influent flow range from 3.0 mgd to about 20 mgd. 16-inch pump discharge pipes manifold together into a 36-inch header discharging to the grit system.

Grit Removal System

The Missoula WWTP has two vortex grit chambers located adjacent to the Headworks Building. The design incorporated provisions for adding a third chamber when flows reach the design capacity of these two chambers. Vortex grit separators provide separation of organics from grit (<2 mm) particles by keeping the organics in suspension and allowing the grit to settle. The grit is captured in the settling compartment and accumulates in the storage hopper at the bottom of the chamber.

Grit pumps are used to transport grit from the grit storage hopper to the grit washer. Missoula uses two flooded suction, recessed impeller grit pumps with suction lines running from the bottom of the grit chamber to the pump vault adjacent to the grit chambers. Utility water (UW) is used to suspend the grit during pumping. The vault contains three pumps; two of them dedicated to one grit chamber each plus one additional pump serving as a shelf spare or to be used with a future grit chamber. Pump suction piping is dedicated to a single grit chamber for each pump. Discharge piping from the grit pump vault to the grit washers in the Headworks Building is also dedicated, with the piping for the future pump already installed.

The two direct feed grit washers located in the Headworks Building accept grit directly from the vortex grit chambers via the grit pumps. The grit slurry is fed through a vortex chamber where a fast spinning rotational movement is generated. Gravity and inertia cause grit and heavier organic particles to settle out of the flow and sink to the lower section of the washer. The lighter organic matter is carried with the wash water over the weir and returned to the influent channel. Separated grit is washed in the bottom of the washer which is separated from the grit washer tank by a perforated plate and perforated rubber diaphragm. The resulting grit material has an organic content typically less than 5% and a water content less than 10%.

7.6.2. Condition, Performance, and Capacity

All components of the headworks system were installed in 2011 and most are still in good condition. The typical design life for wastewater treatment equipment is at least 20 years. Therefore, the headworks equipment is expected to perform through well over half of the planning period or longer. The influent pumps are the only exception as their design life has been shortened by pre-mature impeller wear. In addition, system conditions including total dynamic head (TDH) at high pumped flows and lower than designed minimum influent flows experienced since the headworks construction are cause for evaluating the purchase of new and different influent pumps. Further information on the influent pumps is presented in Chapter 8.

With the exception of the influent pumps, no particular concerns exist regarding the headworks equipment conditions and plant staff will need to evaluate condition and potential need for replacement as the equipment ages. The headworks facility is operator friendly, providing comfortable access to all equipment, and is equipped with an odor control system that minimizes odors in the building and eliminates odors escaping the building.

The headworks facility has increased screenings and grit capture compared to the previous facility, which helps protect downstream equipment. The capacity of the headworks facility was generally designed for a peak hour flow of about 20 mgd. This was demonstrated during the high flows experienced during May and June of 2018. Flows were as high as 20 mgd and did not present a problem for the screens or grit removal system. As pump capacity has been lower than designed, three influent pumps were operated to keep up with the high influent flows. The following summarizes major condition, performance, or capacity issues:

- At eight years of age, most components in good shape
- Influent pumps have experienced problems with premature failure and reduced capacity – currently being addressed
- Typical design life for this equipment: about 20 years, longer with good maintenance
- Rated hydraulic capacity:
    - 12.5 mgd for each screen
    - With both screens in service, just short of 2037 peak hour flow of 26.6 mgd
    - Good screen performance during high flows in spring of 2018
    - 26.0 mgd with both grit chambers in service; almost meets 2037 peak hour flows
- Odor control successful; reduced complaints and noted by plant staff and visitors

7.6.3. Conclusions

- Continue to maintain equipment and replace wear parts.
- The expected equipment life is at least 20 years.

- An evaluation of influent lift pumping improvements is included in Chapter 8.
- No other immediate upgrade or major maintenance needs currently exist.
- Monitor peak flow events and screen performance over the next 10 years; if flows are on track to meet 2037 peak hour flow projections, then re-evaluate screen capacity and potential upgrade needs.

7.7. PRIMARY CLARIFIERS

7.7.1. Description

Flow from the grit system flows by gravity to the primary clarifier splitter box located in the Primary Effluent Lift Station Building and continues to the primary clarifiers. The Missoula WWTP currently has three primary clarifiers that operate in parallel. All three clarifiers are in operation unless major maintenance is required. The following summarizes primary clarifier equipment:

- Three primary clarifiers operating in parallel
- Clarifiers No. 1 and No. 2: 65-ft diameter, 9-ft side water depth (SWD) constructed in 1961
- Clarifier No. 3: 75-ft diameter, 9-ft SWD, constructed in 1973
- All three in operation at all times

7.7.2. Condition, Performance, and Capacity

In spite of their age and due to regular maintenance and service, the primary clarifiers are generally in good condition. The following summarizes clarifier condition:

- Clarifier No. 3: major maintenance of drive mechanism performed in 2008
- Clarifiers No. 1 and 2 scheduled to receive major maintenance in next five years
- Catwalks on clarifiers No. 1 and 2: in need of replacement
- Weir and launders: configuration causes water spray and mild release of odors

Surface Overflow Rates

Design surface overflow rates (SORs) typically range from 800 to 1200 gpd/sq. ft. on an average daily flow basis and 2,000 to 3,000 gpd/sq. ft. on a peak hourly flow basis. Missoula's primary clarifier SORs are currently within these ranges but projected 2037 flows will begin to exceed typical design values. Table 7-4 summarizes the primary clarifier SORs for 2017 and 2037 flow conditions and lists Circular DEQ-2 (Mt. Dept. of Env. Quality, 2016) and typical design values. Circular DEQ-2 lists the maximum average daily SOR at 1,000 gpd/sf, which would limit the existing primary clarifier rated capacity to 11 mgd. The peak hour SOR prescribed by Circular DEQ-2 would accommodate the 2037 peak hour flow without redundancy. DEQ-2 does not require redundancy.

Generally, as long as existing infrastructure is not modified and plant performance is adequate, MDEQ does not require equipment upgrades or expansions to increase capacity to meet Circular DEQ-2 requirements. The following summarizes the overflow rate evaluation:

- The average Circular DEQ-2 SOR of 1,000 would limit primary clarifier capacity to 11 mgd. At the upper range of typical primary clarifier design SOR values, the capacity limit would be 13.2 mgd.
- 2037 peak hour flow would exceed DEQ-2 peak hour requirement.
- If no infrastructure or equipment modifications are made, MDEQ typically does not require upgrading older infrastructure to meet DEQ-2 requirements.
- Performance requirements would be a more convincing driver for consideration of upgrades than meeting DEQ-2 requirements.
- At this time, no adverse effects of gradually decreasing primary clarifier removal efficiency on the overall treatment process have been identified.

Table 7-4: Primary Clarifier Surface Overflow Rates

2017 Conditions (2013-2017)	Surface Area	Avg Daily Flow (7.27 mgd)	Peak Hourly Flow (18.2 mgd)
All Clarifiers in Service, gpd/sf	11,050	658	1,646
Largest out of Service, gpd/sf	6,640	1,095	2,742
2037 Conditions	**Avg Daily Flow (11.2 mgd)**	**Avg Daily Flow (11.2 mgd)**	**Peak Hourly Flow (26.6 mgd)**
All Clarifiers in Service, gpd/sf	11,050	1,013	2,406
Largest out of Service, gpd/sf	6,640	1,688	4,008
MDEQ and Typical Values		**Avg Daily Flow**	**Peak Hourly Flow**
Circular DEQ-2, gpd/sf		1,000	1,500-3,000
Typical Design Criteria, gpd/sf		800-1,200	2,000-3,000

Removal Efficiency

Figures 7-7 and 7-8 show primary clarifier removal performance from 2013-2017 with an average removal efficiency of 30% for BOD and 57% for TSS. The scatter of the plant data for the five-year period does not show any trends; however, when compared to data from the late 1990s and 2007, removal efficiency appears to have decreased.

Metcalf and Eddy (Metcalf & Eddy, 2003) offer a calculation of typical removal efficiencies based on data from many actual primary clarifiers. This calculation uses the clarifier detention time to predict associated removal efficiencies for BOD_5 and TSS. When applying this calculation to the Missoula clarifier volumes and influent flows, the blue trend line shown in Figures 7-7 and 7-8 is generated. The 2013-2017 Missoula data reveals a lower than typical removal efficiency for BOD_5. Since the primary clarifiers remove particulate matter, only $cBOD_5$ in solid form can be removed. If current influent $cBOD_5$ includes a larger portion of soluble organics than influent 20 years ago, the removal efficiency through the primary clarifiers will be lower. However, a detailed influent characteristics analysis was not performed for this evaluation and reasons for declining performance are speculative only. Performance for TSS removal appears to be in line with the 2001-2006 data and may support an assumption of changing influent $cBOD_5$ characteristics.

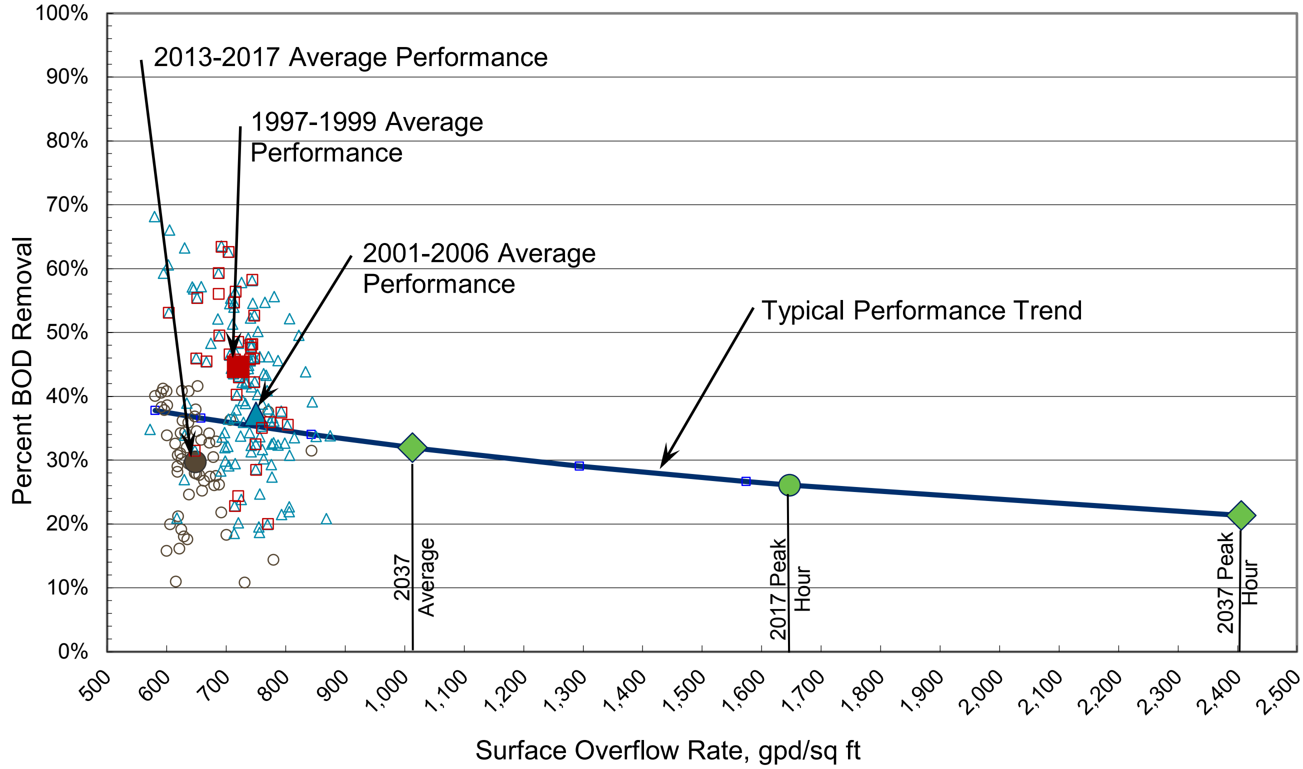


Figure 7-7: BOD_5 Removal versus Overflow Rate for Primary Clarification, 2013-2017

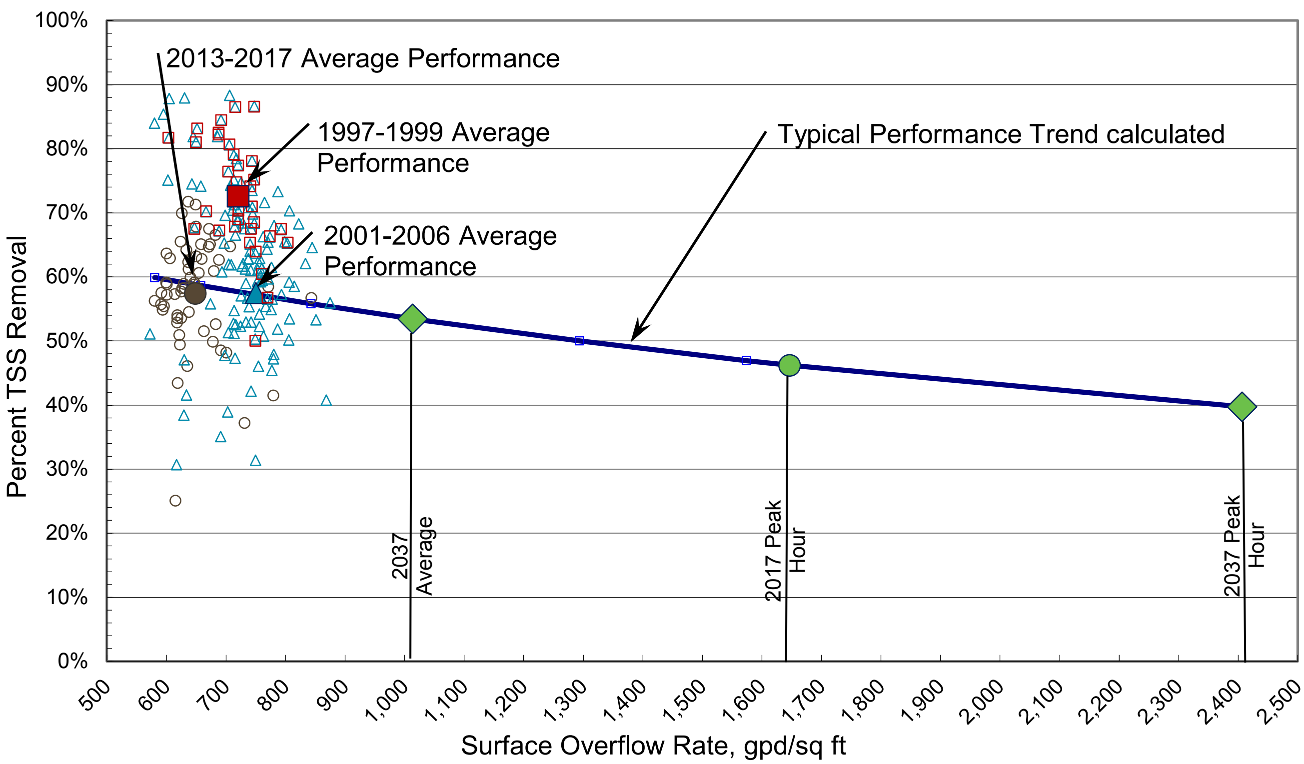


Figure 7-8: TSS Removal versus Overflow Rate for Primary Clarification, 2013-2017

In the absence of data for 2017 peak hour flow or 2037 projected flows, the calculated trend can be used to estimate expected removal efficiencies at higher flows. In the case of BOD_5, caution is needed, as future removal efficiencies may also lie below the calculated performance trend. Actual and projected removal efficiencies are further listed in Table 7-5.

Table 7-5: Actual and Projected BOD_5 and TSS Removal Efficiencies for the Primary Clarifiers

Condition	BOD_5 Removal [1]	TSS Removal [1]
2017 Average Flow	30%	57%
2017 Max. Month Flow	23%	50%
2017 Max. Day Flow [2]	35%	55%
2017 Peak Hour Flow	*26%*	*46%*
2037 Average Flow	*32%*	*53%*
2037 Max. Month Flow	*29%*	*50%*
2037 Max. Day Flow	*27%*	*47%*
2037 Peak Hour Flow	*21%*	*40%*

1. Removal in italics is calculated based on clarifier retention time. Plant data was not available for these conditions.
2. Values for June 13, 2017 for TSS and June 16, 2017 for BOD. BOD values for June 13, 2017 were not available.

Primary clarification serves to reduce load to the bioreactors and to supply the material used in the fermenter to produce VFAs used in the biological phosphorous removal process. Evaluation of the biological processes will need to take into consideration the changing performance of the primary clarifiers with increasing flows. Further analysis will be needed to show if additional primary clarification volume would be beneficial or not needed.

Primary Sludge

Average primary sludge production over the past five years is showing an increasing trend with the average daily primary sludge production in 2016 and 2017 about 20% higher than sludge production in 2013. This trend is shown both in volume and mass of sludge due to a consistent solids percentage in the removed sludge. Without a corresponding increase in influent solids and other constituents, this increase is difficult to explain. About half of the primary sludge is fermented to produce VFAs needed for biological phosphorous removal.

The primary clarifiers at the Missoula WWTP provide some thickening of the primary sludge within the clarifiers. A separate sludge thickening process is not used. The final solids concentration depends on sludge detention time, agitation as a result of sludge collectors, space in the tank, and the characteristics of the solids. Control of the process can be accomplished by providing deep sludge hoppers and flexible sludge pumping arrangements. The hoppers at the Missoula clarifiers are not very deep which somewhat hinders in-clarifier thickening. Primary sludge concentrations over the past five years were in the range of between two and seven percent solids, with a fairly consistent average of about 3.8 percent. This average is down from the five year average of 4.3 percent for the 2001-2006 period even though the hydraulic loading is lower now than it was for the 2001-2006 period. Operational changes such as pumping frequency, pumping duration, and sample collection relative to pump cycles, can influence sampling results. No conclusive evidence exists to explain the lower solids concentrations. However, both concentrations are considered typical for thickening done in a circular clarifier without a deep hopper and are considered satisfactory for overall plant performance. Table 7-6 summarizes primary sludge attributes for the period of record.

Table 7-6: Primary Sludge Production

Condition	Average Daily Volume, gal	Solids Percentage	Mass, lb
2015-2017 Average	29,400	3.8	9,300
2015-2017 Maximum [1]	52,300	7.5	19,600
2013 Average	25,300	3.8	8,000
2014 Average	30,000	3.4	8,500
2015 Average	27,900	3.9	9,100
2016 Average	30,100	3.7	9,300
2017 Average	30,299	3.8	9,600

1. Values in this row are not from a single occurrence.

7.7.3. Conclusions

- Major maintenance for Clarifiers No. 1 and 2 is planned to occur within the next five years.
- Covering the clarifiers for odor control was considered in the past and discarded. As there have not been further odor complaints since the operational changes at the composting facility have been implemented, primary clarifier covers will not be evaluated further.
- Primary clarifier capacity is finite and increasing flows and loads will cause reduced performance.
- The reduced removal efficiency at higher flows will be considered when evaluating different treatment alternatives.
- Expansion of primary clarifier capacity will not be considered.

7.8. PRIMARY SLUDGE PUMPS

7.8.1. Description

The existing primary sludge pump station is located in the center of the three primary clarifiers below the primary effluent lift pump station. Three pneumatic diaphragm primary sludge pumps were added in 1998 to replace the original sludge pumps. High pressure air at 110 psi, provided by two 40 horsepower compressors located in the basement of the dissolved air flotation thickener building is used to operate the pneumatic pumps. The following summarizes primary sludge pumping equipment:

- Original pump installation in 1998
- Identical pumps used for pumping of scum and thickened waste activated sludge (TWAS); efficiency in spare part management and operator maintenance and service capabilities
- All pneumatic diaphragm pumps in the plant are served by one compressed air system

- Two Atlas Copco Model GA22-100 air compressors; installed in 2004
- One GA22 replaced with GA22-VFD in 2012
- Fitted with a larger receiver in 2012

Table 7-7: Primary Sludge Pump Summary

Parameter	Value
Pumps	
Number	3
Type	Pneumatic diaphragm
Capacity (each)	200 gpm
Compressors	
Number	2
Manufacturer	Atlas Copco
Air Pressure	110 psi
Power	40 hp

7.8.2. Condition, Performance, and Capacity

- Pumps are serviceable; replacement of wear parts occurring as needed
- Pumps are operating well; no plans for replacement for foreseeable future
- Capacity: adequate; pumps operate approximately six minutes of every hour
- Compressor capacity: more than adequate for the diaphragm pumps

Depending on the level of thickening desired, the pumps can be operated using different time intervals. For current average influent flows and loads and a primary solids concentration of 3.7%, one pump would need to operate just over 6 minutes per hour. Pump capacity for 2037 would also be more than adequate as declining removal efficiencies with increasing hydraulic loading would not increase primary sludge volumes beyond the pump capacities.

7.8.3. Conclusions

- Plant staff regularly replace wear parts and perform maintenance and are currently not planning on replacing the pumps or compressed air system.
- The overall capacity is more than adequate for current and future flows and does not present a reason for replacement planning.
- It is recommended to plan for replacing the older single speed compressor with a variable speed capable compressor, matching the unit installed in 2012.

7.9. PRIMARY SCUM PUMPS

Three new submersible non-clog centrifugal pumps were installed during the 2004 upgrade. An older manual valve system that allows operators to manually open a valve and use the primary sludge pumps to empty the scum box is still in place and can be used in case of pump failure. Primary scum is pumped to the primary anaerobic digester. With regular maintenance as factory specified, the pumps are expected to have a useful life of 15 to 20 years. The following summarizes description, condition, performance, and capacity.

- Installed in 2004
- Pump directly to digester
- In good condition
- No capacity concerns; no replacement plans

Table 7-8: Primary Scum Pump Summary

Parameter	Value
Number of Pumps	3
Type	Submersible, non-clog, centrifugal
Capacity	250 gpm
Head	75 feet
Motor Size	20 hp

7.10. PRIMARY EFFLUENT LIFT PUMPS AND ELUTRIATION WATER PUMPS

7.10.1. Description

Due to hydraulic grade limitations at the Missoula WWTP, a lift station is necessary to lift the primary clarifier effluent to the secondary treatment process bioreactor basins. The primary effluent lift station includes two hydraulically connected wet wells located in the basement of the Primary Effluent Lift Station Building, each with three variable speed, vertical, mixed flow pumps mounted above the wet wells.

The elutriation water pumps are also located in the basement of the Primary Effluent Lift Station Building in a wet well hydraulically connected to the primary effluent lift pump wet well. These pumps pump primary effluent to the fermenter to elutriate VFAs produced in the fermenter and returned to the bioreactors for biological phosphorous removal. Following is a summary of the primary effluent pump system. The design criteria for these pump stations are presented in Table 7-9.

- Two sets of primary effluent lift pumps
  - Located in basement of Primary Effluent Lift Station Building
  - Side 1 pumps: installed in 1998 and rebuilt prior to 2008; three equally sized pumps
  - Side 2 pumps: installed in 2004; two larger and one smaller pump

- Side 1 and Side 2 pumps operate independently of each other; not able to cross feed to the two bioreactors
- Two elutriation water pumps installed in 2004
  - Elutriation pumps located in Primary Effluent Lift Station Building

Table 7-9: Primary Effluent Lift Station and Elutriation Water Pumps

Parameter	Value
Older Primary Lift Pumps	
Number of Pumps	3 (2 duty, 1 standby)
Type	Vertical, Mixed Flow
Capacity (each)	5,800 gpm
Pump Station Firm Capacity	~16 mgd
TDH at Design Capacity	21 ft
Speed	Variable, 1175 RPM Max.
Motor Size (each)	50 hp
Newer Primary Lift Pumps	
Number of Pumps	3 (2 duty, 1 standby)
Type	Vertical, Mixed Flow
Capacity (each)	2 @ 5,800 gpm; 1@ 2,250 gpm
Pump Station Firm Capacity	~9.3 mgd
TDH at Design Capacity	37 ft
Speed	Variable, 1175 RPM Max.
Motor Size (each)	2 @ 75 hp; 1 @ 25 hp
Elutriation Water Pumps	
Number of Pumps	2
Type	Vertical, Turbine
Capacity (each)	500 gpm
Pump Station Firm Capacity	0.8 mgd
TDH at Design Capacity	60 ft
Speed	Variable, 1760 RPM Max
Motor Size (each)	15 hp

7.10.2. Condition, Performance, and Capacity

All system components are in good condition and with regular maintenance expected to have a useful life beyond the 20-year planning period. The pumps have been performing well, including during the high flows in the spring of 2018. The flow split between Side 1 and Side 2 is typically operated at a ratio of 1:2 with about 37 percent of flow routed to Side 1 and the remainder to Side 2. The Side 1 bioreactor is a fixed capacity system and increasing plant flows will be routed to Side 2 with the Side 1 flows remaining steady. Exceptions may be made during peak flows to distribute the additional flow between both bioreactor sides. The following summarizes major condition, performance and capacity items:

- All equipment in overall good condition
- Flow split between Side 1 and Side 2 bioreactors: 33% and 67%, respectively, up to maximum capacity of Side 1 bioreactor; during spring 2018, split closer to 45% and 55%
- Side 1 pumps:
    - Side 1 bioreactor is fixed capacity system with maximum flow of 8.3 mgd
    - Side 1 pumps: no capacity issues; two of three pumps meeting all required flows

Table 7-10: Primary Effluent Lift Station Flows

Parameter	Total Secondary System Flow	Flow Split to Side 1 Bioreactors [1]	Flow Split to Side 2 Bioreactors
2017 Flow			
Average Daily Flow, mgd	7.27	2.69	4.58
Maximum Monthly Flow, mgd	9.32	3.45	5.87
Maximum Day Flow, mgd	11.32	4.19	7.13
Peak Hour Flow, mgd	18.2	6.73	11.47
2037 Flow			
Average Daily Flow (ADF), mgd	11.2	4.2	7.0
Maximum Monthly Flow (MMF), mgd	14.3	5.3	9.0
Peak Daily Flow (PDF), mgd	17.4	6.4	11.0
Peak Hourly Flow (PHF), mgd	26.6	8.3	18.3

1. The split is generally 33% to the old bioreactors and 67% to the new bioreactors. The old bioreactors and final clarifiers have a rated biological nutrient removal (BNR) capacity of 6.0 mgd on a maximum monthly basis and 8.3 mgd for peak hour flow.

- Side 2 pumps:
    - Low flows accommodated with one larger pump – small pump not needed for low flows
    - For Side 2 flows greater than 9.3 mgd, two larger pumps needed
    - For Side 2 flows greater than 13.4 mgd all three pumps needed (necessary during high flows of 2018)
    - Estimated flow for three pumps operating is approximately 15 mgd – less than 2037 peak hour flows
- Combined maximum pump capacity with full redundancy (and as limited by Side 1 bioreactor capacity) = approximately 18 mgd, which is just below current design peak hour flows
- Combined maximum pump capacity without redundancy (and as limited by Side 1 bioreactor capacity) = approximately 24 mgd, which is less than the 2037 peak hour flow

7.10.3. Conclusions

- Plant staff should continue to monitor Side 1 pumps and determine when replacement will be needed; pump age suggests that planning for replacement would be prudent.
- Side 2 pumps are currently at capacity; firm capacity is less than current peak hour flows. Replacement of the smaller pump with an equal sized pump would increase firm capacity to about

13.4 mgd, which would meet current and near-term peak flows but would still be below projected 2037 peak hour flows.

7.11. BIOREACTORS

7.11.1. Description

The Missoula WWTP has two separate bioreactors with two trains in each bioreactor. The Side 1 bioreactor was originally designed as a conventional activated sludge process in 1973. During the last plant upgrade in 2004, the basin was retrofitted to accommodate two trains configured according to the Modified Johannesburg Process, allowing for nutrient removal. The Side 2 bioreactor was added to increase overall plant capacity. Due to the given configuration of the existing Side 1 bioreactor basin and the freedom to design the new Side 2 bioreactor to optimal dimensions, the number of cells within each train differ between the Side 1 and 2 bioreactors. Table 7-11 lists the bioreactor cells and dimensions. Individual bioreactor equipment components are described separately in following sections.

Table 7-11: Summary of Bioreactor Cells and Nominal Dimensions

Bioreactor	No. of Trains	Cell No.	Dimensions (l x w x d)	Volume (gal)	Percent of Total Volume	Type of Cell
Old Bioreactor	2	1	19'6" x 29'6" x 15'7"	67,040	7	Pre-Anoxic
		2	22'5" x 29'6" x 15'7"	77,080	8	Anaerobic
		3	28' x 29'6" x 15'7"	96,260	10	Anoxic
		4	28" x 29'6" x 15'7"	96,260	10	Anoxic / Aerobic "Swing"
		5	42" x 29'6" x 15'7"	144,390	15	Aerobic
		6	59'11" x 34'6" x 15'7"	240,910	25	Aerobic
		7	59'11" x 34'6" x 15'7"	240,910	25	Aerobic
Total				**962,850**		
New Bioreactor	2	1	37' x 22' 15'6"	94,380	7	Pre-Anoxic
		2	42' x 22' x 15'6"	107,130	8	Anaerobic
		3	50' x 22' x 15'6"	127,530	9.5	Anoxic
		4	51' x 22' x 15'6"	130,080	10	Anoxic / Aerobic "Swing"
		5	91' x 42' x 15'6"	443,120	33	Aerobic
		6	91' x 42' x 15'6"	443,120	33	Aerobic
Total				**1,345,360**		

The plant has been operated with the swing zone in anoxic (non-aerated) mode throughout the year. Since the plant is currently still somewhat underloaded compared to its ultimate capacity, the aeration provided in the dedicated aerobic cells is sufficient to provide adequate BOD and ammonia removal. The aerobic volume of the bioreactors comprises about 66 percent of the total treatment volume, with the remainder is divided between anoxic volume (26%) and anaerobic (8%).

A mixed liquor recycle (MLR) to the anoxic zone facilitates denitrification (removal of nitrate), especially during the summer months when permit limits are in place. An MLR rate of 100 to 150 percent has been used year-round with reportedly better results at the lower end of this range.

The mixed liquor suspended solids (MLSS) concentration in the bioreactors has been fairly steady at about 1,900 mg/L for Side 1 and 2,500 mg/L for Side 2. These concentrations are maintained year-round and result in solids retention times (SRTs) of about ten days.

7.11.2. Condition, Performance, and Capacity

Condition

The overall condition of both bioreactor basins, walkways, stairs, grating, guardrails, and process equipment is good and with regular maintenance the basins are not expected to require major retrofit or equipment replacement within the planning period. The only exception are the Side 2 bioreactor scum skimmers, which either require adjustment in order to function properly or may be removed entirely. The plant reportedly does not experience significant foaming events and regular scum could be allowed to pass through to the clarifiers. More details on individual equipment components is provided in following sections.

Performance

Effluent data from the past five years shows very consistent performance of the WWTP with respect to treatment of $cBOD_5$, TSS, ammonia, nitrate + nitrite, total nitrogen, total phosphorous, and *E. coli*. Effluent concentrations fluctuate slightly with season as caused by changing wastewater temperatures, but no significant long-term trends were observed. Plant staff operate the plant very consistently and have not needed to significantly change process parameters like aeration, recycle rates, or handling of process return streams for years. Barring any sudden changes in influent quality or MPDES permitting requirements, the plant is expected to keep performing consistently until influent flows and loads near the capacity of the plant. Table 7-12 lists annual average effluent concentrations for a number of measured parameters and the corresponding current permit limit. On average, the plant has no problem complying with current permit limits.

Table 7-12: Annual Average Effluent Quality for Select Parameters

Parameter	2013	2014	2015	2016	2017	2015-2017 Average	MPDES Permit
$cBOD_5$, mg/L	1.98	2.24	2.08	2.30	2.26	2.21	19
TN, mg/L	9.30	8.88	9.79	10.13	9.69	9.87	15 [1]
Ammonia, mg/L	0.22	0.36	0.57	0.74	0.35	0.56	3.4 [2]
Nitrate + Nitrite, mg/L	7.58	7.00	7.57	7.63	7.73	7.64	-- [3]
TP, mg/L	0.68	0.75	0.47	0.77	0.85	0.65	1.7 [4]

1. Calculated based on load limit of 910 lb/d and average flow of 7.27 mgd.
2. Average monthly limit.
3. No reasonable potential to exceed the human health standard of 10 mg/L, therefore no limit in permit.
4. Calculated based on load limit of 101 lb/d and average flow of 7.27 mgd.

The following graphs show effluent quality for $cBOD_5$, Ammonia, TN, and TP. Concentrations are plotted for individual sample results (daily), monthly averages, and annual averages. The graphs illustrate the variations in individual sampling results, as well as the overall healthy averages. Only one MPDES permit exceedance occurred within the period of record. Ammonia exceeded the monthly and weekly averages by 6 percent and 1 percent during November 2016. As the graph shows, performance was back in compliance within a week. Other exceedances were discussed in Chapter 2 and included one instance each of toxicity, pH, and *E. coli*. These exceedances were individual instances and do not indicate a trend or overall performance issues with the plant. Exceedances can occur even in well-operated plants due to unforeseen upsets caused by toxins in the influent, sudden changes in flow, slug loads in the influent, equipment failure, and temperature-related changes in bacterial composition in the bioreactors. The fact that all exceedances were mitigated within short time periods demonstrates that the process is indeed healthy to allow it to bounce back quickly. Otherwise, the process has been stable without major upsets or changes that would have required adjustment of recycle rates, biomass concentrations, or other process parameters.

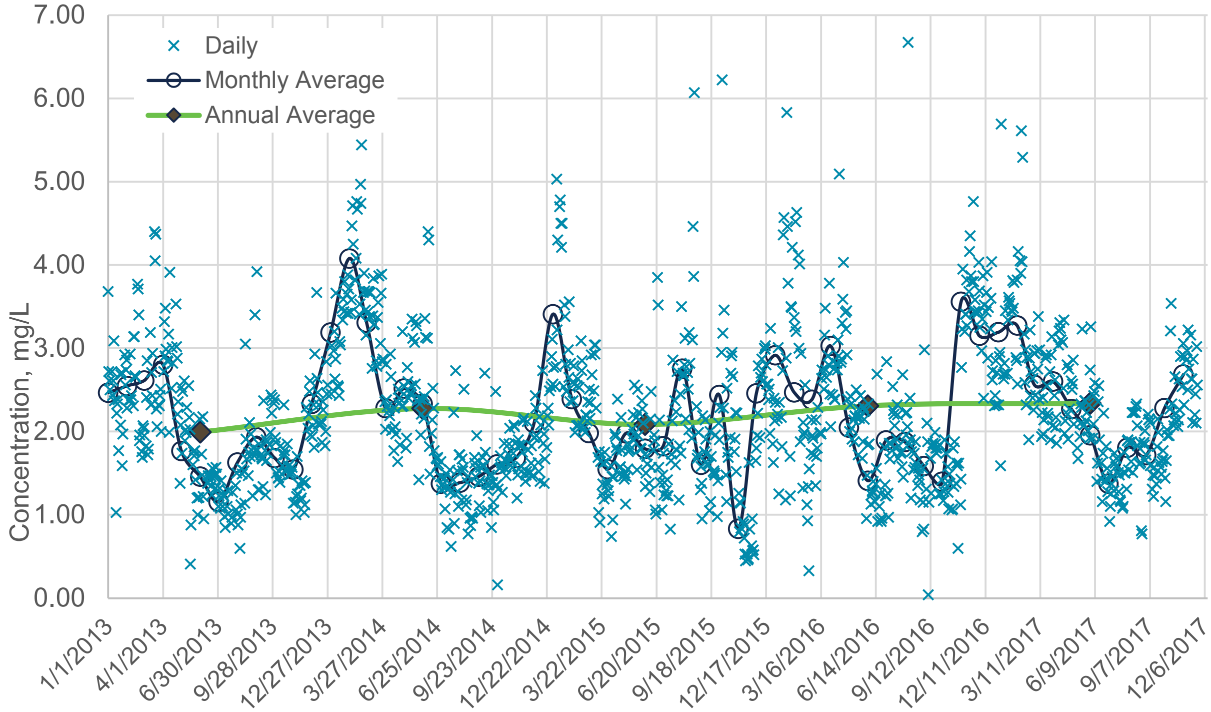


Figure 7-9: Effluent $cBOD_5$ Concentrations, 2013-2017

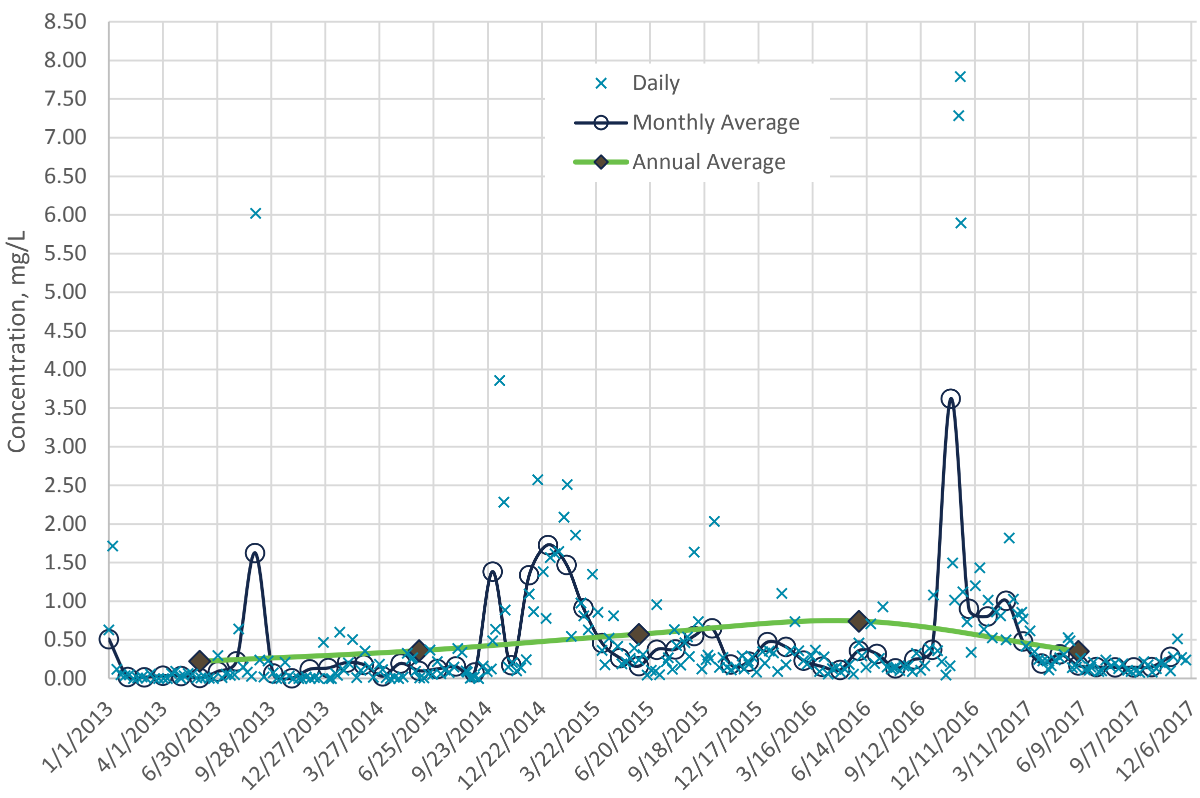


Figure 7-10: Effluent Ammonia Concentrations, 2013-2017

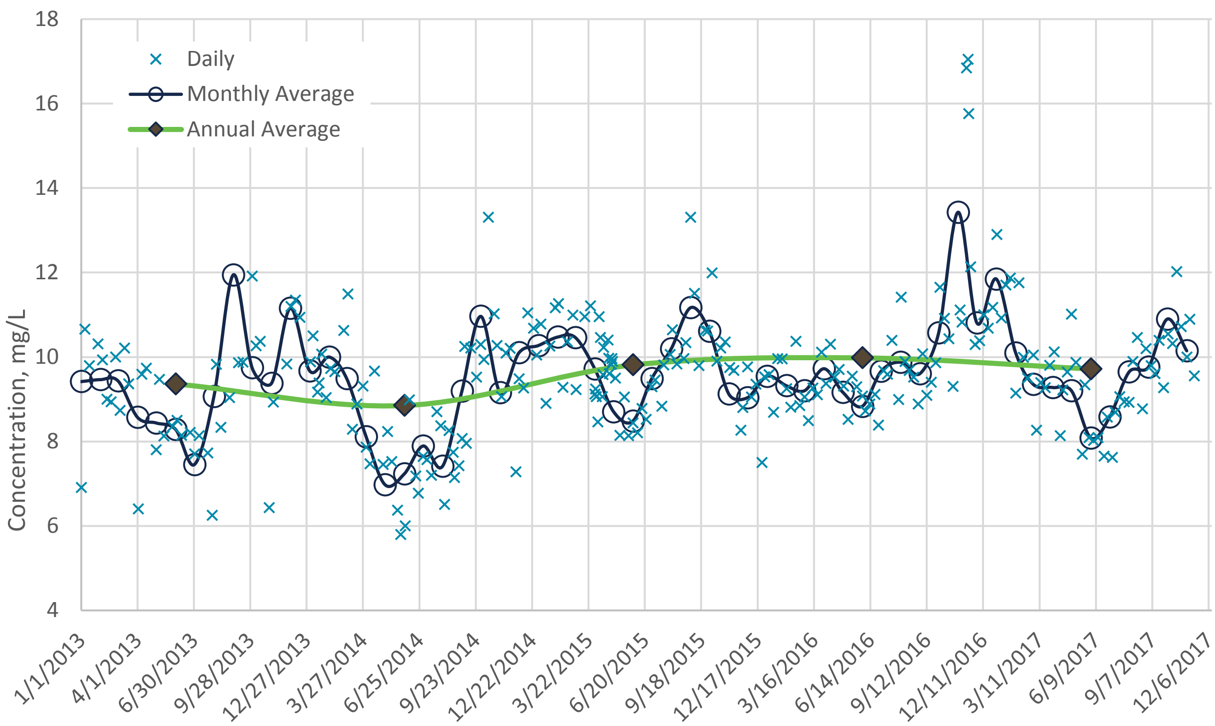


Figure 7-11: Effluent Total Nitrogen Concentrations, 2013-2017

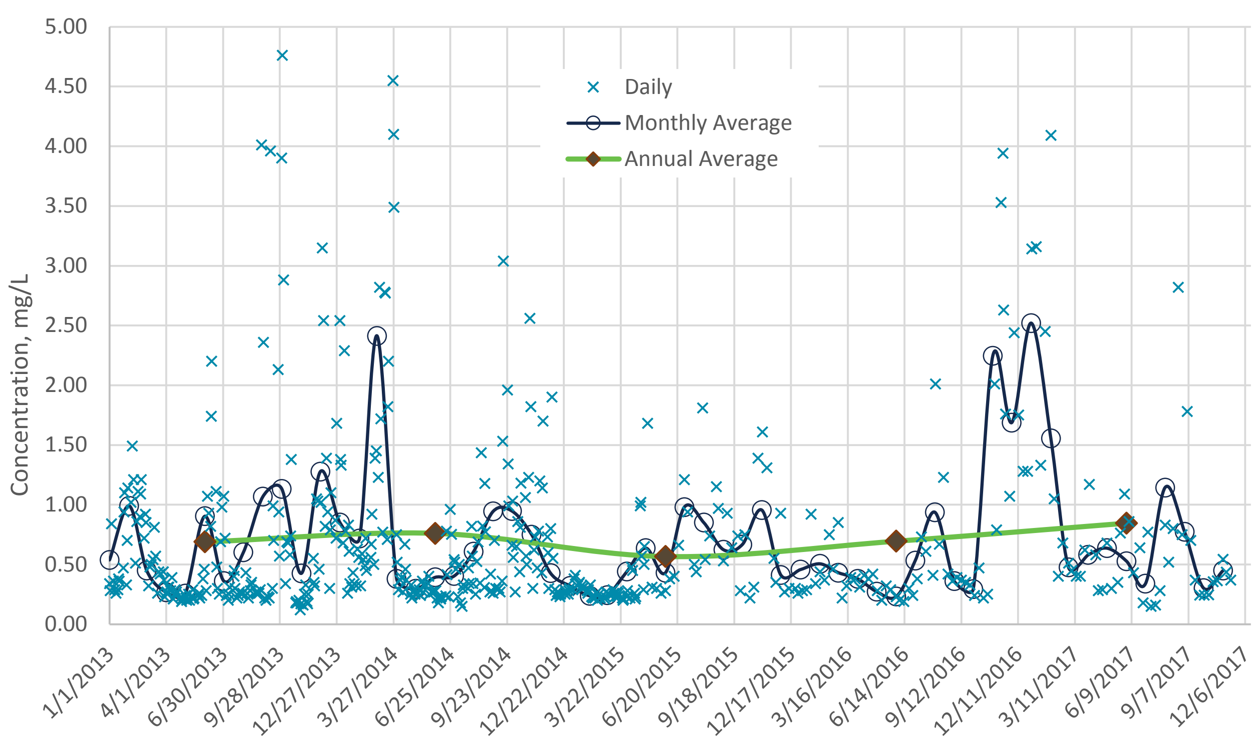


Figure 7-12: Effluent Total Phosphorous Concentrations, 2013-2017

Treatment Capacity

The two Side 1 bioreactor trains are considered capable of treating 6.0 mgd at maximum month loads. In order to meet the 2004 design capacity of 13.8 mgd (max month flow), the Side 2 bioreactor with two trains was added, providing an additional 7.8-mgd process capacity. This capacity refers to treatment capacity and the bioreactor system is capable of hydraulically passing higher flows as discussed above. The current operational scheme does not allocate the full design flow to the Side 1 bioreactor but rather underloads it. The flow split shown in Table 7-13 was developed by plant staff based on experience with hydraulic constraints and treatment results. Table 7-13 lists bioreactor capacities as designed, as currently operated, and as modeled for 2037 flows and loads. The current (2017) flow split between the two bioreactor sides was retained for the future scenarios.

Table 7-13: Summary of Bioreactor Design Capacities by Flows

Condition	Total Flow	Side 1 Bioreactor	Side 2 Bioreactor
2001 Design Flows[1]			
Flow Split		43%	57%
Average, mgd	12.0	5.2	6.8
Max Month, mgd	13.8	6.0	7.8
Max Day, mgd	15.6	6.8	8.8
Peak Hour	19.2	8.3	10.9
2017 Flows			
Flow Split		37%	63%
Average, mgd	7.27	2.69	4.58
Max Month, mgd	9.32	3.45	5.87
Max Day, mgd	11.32	4.19	7.13
2037 Flows			
Flow Split		37%	63%
Average, mgd	11.2	4.2	7.0
Max Month, mgd	14.3	5.3	9.0
Max Day, mgd	17.4	6.4	11.0

1. Design capacity of bioreactors as listed in the 2001 Predesign Report for the new biological nutrient removal (BNR) treatment system.

Figure 7-13 illustrates the information presented in the above table. The Side 2 graph shows that at 2037 maximum day and peak hour flows, the Side 2 bioreactor (right) would be taxed beyond its design treatment capacity.

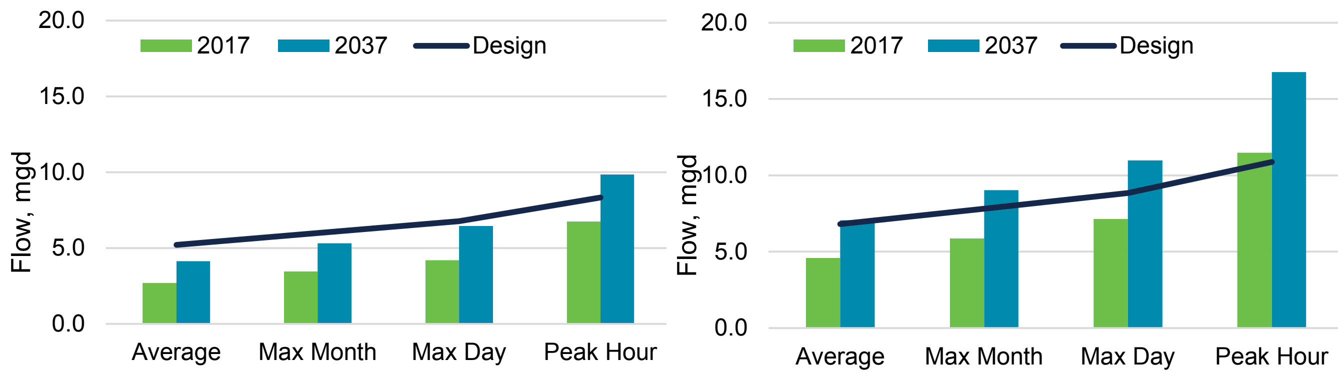
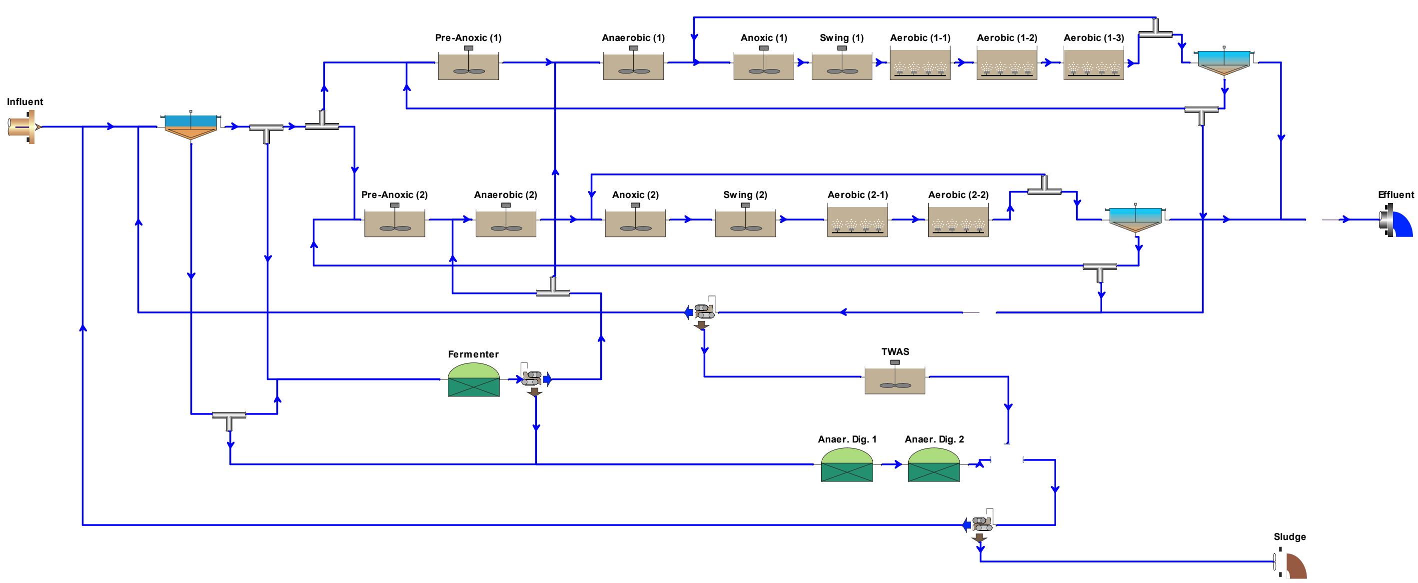


Figure 7-13: Bioreactor Treatment Capacity (Left: Side 1 Bioreactor; Right: Side 2 Bioreactor)

Using average influent and effluent data for 2015-2017 and plant process data for internal flow rates and concentrations for a number of parameters, a BioWin model was set up and calibrated to mimic current plant performance. This model was then used to evaluate the plant's capacity for treating the projected 2037 flows and loads. In order to allow the Side 1 bioreactor to continue to treat the same flows but vary the treatment capacity for the Side 2 bioreactor, the two sides were separated in the model. This also allowed for better definition of individual bioreactor cells as the two sides have a different number of cells in each train. Figure 7-14 shows the modeled plant. The brown bioreactor basins correspond to the cells listed in Table 7-11 above.

Figure 7-14: BioWin Model Configuration



Modeling results suggest that the plant will perform well with respect to $cBOD_5$ removal at all modeled flow conditions. However, model performance for nutrient removal was more limited and only showed good results for influent flows up to about 9.2 mgd and corresponding loads. As flows and loads increase beyond this point, model output suggests that the plant would not be able to meet its current nutrient limits unless some operational changes were to be put into place. Suggested operational changes are described in Chapter 8 as part of the treatment alternative analysis.

7.11.3. Conclusions

- The physical condition of the bioreactor basins and equipment is generally good; individual equipment is described in more detail in the following sections.
- Smaller equipment is serviced or replaced as part of regular plant maintenance.
- The ducking scum skimmers are discussed further in Chapter 8.
- The treatment process is stable and for years has been providing good effluent results without requiring significant adjustments to process parameters.
- Treatment capacity is present at all projected flows and loads for $cBOD_5$ removal but reaches its limit for nutrient removal at an average influent flow of about 9.2 mgd and corresponding load. At higher flows, the plant would likely begin to exceed existing MPDES nutrient limits.
- No capacity or age concerns exist for individual bioreactor equipment.

7.12. RECYCLE PUMPS AND MIXERS

Submersible axial-flow wall pumps, which are high-flow, low-head propeller pumps, are used in recycling nitrified mixed liquor back to the main anoxic zone. These MLR pumps are sized for operation at a flow rate of three times the annual average design flow rate (3Q) and are equipped with VFDs to permit turndown to about 30 percent of that (1Q). Magnetic flow meters are installed on each recycle line. A total of four wall mounted, internal recycle pumps are used, one for each process train. Standby units are provided for installation as required.

Submersible low speed mixers are used to keep process mixed liquor in suspension in the anoxic and anaerobic zones and in the final stages of the aerobic zones. The anoxic/aerobic “swing” zone in each process train also requires mixing when these zones are operated anoxically for increased nitrogen removal. The same mixers were used for both bioreactor sides. The following summarizes MLR pump and mixer details:

- Mixers and MLR pumps installed in 2004
- In good condition with regular maintenance and replacement of wear parts
- Service life of at least 20 years
- Sized for up to 3Q; operated at 1Q – 1.5Q
- Capacity of pumps and mixers: adequate for each bioreactor train; upsizing not required – capacity expansion would add additional process trains and associated equipment

Table 7-14 summarizes specific equipment information and capacities for the mixers and MLR pumps.

Table 7-14: Bioreactor Pumps and Mixers

Equipment Description	Old Bioreactors	New Bioreactors
Mixed Liquor Recycle Pumps		
Type	Submersible Propeller	Submersible Propeller
Number	2 (1 per bioreactor)	2 (1 per bioreactor)
Capacity	5,450 gpm	7,100 gpm
Power	20 hp	20 hp
Control	Variable Speed	Variable Speed
Mixers		
Type	Submersible Propeller	Submersible Propeller
Number	8 (4 per bioreactor)	8 (4 per bioreactor)
Capacity	5,000 gpm	5,000 gpm
Number	2 (1 per bioreactor)	2 (1 per bioreactor)
Capacity	6,100	6,100
Power	5.5 hp	5.5 hp

7.13. BLOWERS AND AERATION EQUIPMENT

7.13.1. Description

Three multi-stage air blowers with VFDs are used to supply both bioreactor sides. The blowers are equipped with variable speed drives to control airflow and capable of turndown to 60 percent of maximum flow. The units are located in the blower building adjacent to the Side 1 bioreactors. The aerobic and swing zones of the bioreactors are equipped with flexible membrane fine bubble diffusers. The following summarizes blower and aeration system information:

- Three multistage, centrifugal blowers serve both bioreactor sides
- Fine bubble diffuser system installed in aerobic and swing zones of both bioreactor sides
- Blowers and aeration equipment installed in 2004
- Blowers modified in 2008 to increase capacity; one blower used to supply all required air
- Blowers and aeration system regularly maintained; no need for replacement
- May look at more energy efficient blowers as the existing blowers age

Table 7-15 lists available blower and aeration system information.

Table 7-15: Blowers and Aeration Equipment

Bioreactor Equipment		
Blowers		
Number of Blowers	3 (1 duty, 2 standby)	
Type of Blower	Multi-Stage Centrifugal with VFD	
Motor Size	350 hp	
Capacity (each)	8,600 scfm	
Aeration Equipment	*Old Bioreactor*	*New Bioreactor*
Type	Fine Bubble Membrane Disk	Fine Bubble Membrane Disk
Max Month Air Demand	5,110 scfm	6,640 scfm
Average Air Demand	3,830 scfm	4,980 scfm

7.13.2. Condition, Performance, and Capacity

- Blowers and membrane discs in good condition
- Blower capacity: adequate for existing bioreactors
- Blower analysis done in 2012:
  - Expense of more energy efficient blowers exceeded benefit of energy savings
  - Renewed analysis warranted if energy efficient blowers drop in price
- Membrane discs replaced at manufacturer-recommended intervals
- Number of diffusers appropriate for needed air flows
- Air valve actuators are nearing design life; replacement scheduling in progress

7.13.3. Conclusions

- No changes to the blowers are planned or needed at this time for condition or capacity reasons.
- Blowers are one of the largest consumers of energy at wastewater treatment plants and consideration for replacing the blowers with more energy efficient units may be warranted – see the discussion on energy efficiency at the end of this Chapter.
- No change to the type of aeration system is recommended.
- Aeration system component replacement will occur as needed.

7.14. SECONDARY CLARIFIERS AND RELATED EQUIPMENT

7.14.1. Description

The Missoula plant is currently served by six secondary clarifiers. The Side 1 clarifiers include two units with a 79-foot diameter and 12-foot side wall depth (SWD) and one clarifier with a 90-foot diameter and 14-foot SWD. All three are equipped with central feed, inboard peripheral weirs, and center drive sludge rakes with hydraulic sludge collection to a central sludge collection cell.

The Side 2 final clarifiers are larger (100-foot diameter, 14-foot SWD) and are configured similarly to the old clarifiers with the exception of the sludge withdrawal mechanisms. Sludge in the Side 2 clarifiers is collected by a single ported suction header that provides full floor suction and is tapered to maintain adequate velocity throughout the header.

The clarifiers on each side are sized such that two units can accommodate the flow from their associated bioreactors at currently experienced flows with the third clarifier providing excess capacity. The following list summarizes clarifier information:

- Side 1 – serving original (old) bioreactor:
  - One 90-ft diameter, 14-ft side water depth, constructed in 1980s (clarifier 1)
  - Two 79-ft diameter, 12-ft side water depth, constructed in 1973 (clarifiers 2 and 3)
  - Only the two 79-ft diameter clarifiers are in service
- Side 2 – serving new bioreactor:
  - Three 100-ft diameter, 14-ft side water depth, constructed in 2004 (clarifiers 4-6)
  - Only two clarifiers are in service
  - Provisions exist for construction of a fourth clarifier

7.14.2. Condition, Performance, and Capacity

Both sets of clarifiers are in good operating condition and have been performing satisfactorily with excellent solids removal as evidenced by consistently low effluent TSS concentrations – see Table 7-16. All clarifiers have been put on a rotation to perform major maintenance on the clarifier mechanisms over the next two to five years.

Table 7-16: Secondary Clarifier Process Details

Period	Average Effluent TSS
2013	3.8
2014	4.5
2015	4.4
2016	5.1
2017	4.9
2015-2017	4.8

The clarifiers currently have excess capacity, so that one clarifier on each side is out of service at any time, leaving the unit as a redundant or standby unit to be used when service on another clarifier is performed. Since the Side 1 plant flows will not increase beyond its design capacity, the Side 1 clarifiers will continue to be adequately sized. As flows increase, the additional flow is routed to Side 2. Side 2 clarifier capacity was checked for 2037 flows and use of the third clarifier would be recommended to provide consistent solids separation. Table 7-17 lists secondary clarifier operating parameters and capacities for current and 2037 operating conditions.

Table 7-17: Secondary Clarifier Process Details

Side 1 Secondary Clarifiers				
Number	3			
2 of Diameter/Depth	79 ft / 12 ft			
1 of Diameter/Depth	90 ft / 14 ft			
The largest clarifier is typically out of service				
Parameter	**2017 Condition**		**2037 Condition**	
	Max Day	**Peak Hour**	**Max Day**	**Peak Hour**
Flow, mgd	6.8	8.3	6.8	8.3
MLSS, mg/L	2,050	2,050	2,050	2,050
RAS flow, mgd	3.7	3.7	3.7	3.7
Total Surface Area (all in service), sf Solids Loading Rate (SLR), lbs/day/sf Surface Overflow Rate (SOR) (gpd/sf)	16,156 12 --	16,156 -- 473	16,156 17 --	16,156 -- 513
Tot. Surface Area (lgst out of service), sf SLR, lbs/day/sf SOR, gpd/sf	9,803 20 --	9,803 -- 780	9,803 29 --	9,803 -- 847
Side 2 Secondary Clarifiers				
Number	3			
Diameter/Depth	100 ft / 14 ft			
One clarifier is typically out of service				
Parameter	**2017 Avg. Condition**		**2037 Condition**	
	Max Day	**Peak Hour**	**Max Day**	**Peak Hour**
Flow, mgd	6.6	10.6	10.6	18.3
MLSS, mg/L	2,500	2,500	2,500	2,500
RAS flow, mgd	6.3	6.4	6.4	6.4
Total Surface Area (all in service), sf SLR, lbs/day/sf SOR, gpd/sf	23,560 11 --	23,560 -- 448	23,560 18 --	23,560 -- 777
Tot. Surface Area (one out of service), sf SLR, lbs/day/sf SOR, gpd/sf	15,700 17 --	15,700 -- 672	15,700 27 --	15,700 -- 1,165
Circular DEQ 2 Requirements and Typical Design				
Circular DEQ 2 Requirements Max Day SLR (lbs/day/sf) Peak Hour SOR (gpd/sf)	 -- --	 35 --	 -- 1,000	
Typical Design SLR (lbs/day/sf) SOR (gpd/sf)	 24 - 36 400 - 700	 38 --	 -- 1,000 – 1,600	

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7.14.3. Conclusions

- Aside from planned major maintenance, no equipment replacement is needed.
- The old clarifiers have excess capacity relative to the capacity of the old bioreactor.
- The new clarifiers have redundant capacity relative to the capacity of the new bioreactor.
- Preliminary analysis shows that the three new clarifiers are sufficient to handle the 2037 peak hour flow; BioWin is capable of providing a state point analysis, which will be included in modeling for 2037 conditions to verify that the clarifier capacity will be adequate for the projected flows.

7.15. RETURN ACTIVATED SLUDGE PUMPS

7.15.1. Description

The recycle pumping equipment for the Side 1 bioreactors and clarifiers consists of two pumps installed in 2004 and one pump dating back to the 1980s. The new pumps are dedicated to the two 79-ft clarifiers and the old pump is on standby along with the 90-ft clarifier. RAS is returned to the pre-anoxic cells of both bioreactor trains and pump rates are automatically paced relative to plant influent flow rates by means of variable frequency drives (VFDs) and flow meters.

RAS pumping for the Side 2 bioreactors is provided by four pumps installed with the plant upgrade in 2004. Three pumps can handle the sludge flow from the three clarifiers with the forth pump on standby. RAS is returned to the pre-anoxic cell of both bioreactor trains and pump rates are also paced based on influent flow rates. The same pumps can also be used for clarifier dewatering.

For the Side 1 bioreactors, sludge wasting occurs only from one of the RAS lines by means of a metered and automatically regulated side stream. Even distribution of the remaining RAS is ensured by mixing in the splitter box prior to flowing into the bioreactor trains and one other cross-over/mixing point in the system. Sludge wasting from the Side 2 bioreactors is also carried out from a metered and automatically regulated side stream line off the RAS discharge header; however, there is only one discharge header for all clarifiers and bioreactor trains and wasting occurs from the full RAS flow prior to re-entry into the bioreactor trains. This arrangement does not require independent WAS pumping to the sludge handling operations.

The following list summarizes pump information:

- RAS pumps for Side 1 clarifiers: variable speed, non-clog centrifugal pumps
  - Clarifier 1 pump installed in the 1980s, rebuilt in 2015; on standby
  - Two pumps dedicated to clarifiers 2 and 3 replaced in 2004
- RAS pumps for Side 2 clarifiers: four variable speed, non-clog centrifugal pumps
  - Manual valves for drawing from more than one clarifier
  - Same pumps used for clarifier dewatering if needed
- Waste activated sludge diverted automatically via actuated valves and flow meters; no separate WAS pumps exist

Table 7-18 lists detailed information for the two sets of RAS pumps.

Table 7-18: RAS Pumps

Equipment Description	Older Pump	Newer Pumps
Side 1 Bioreactors		
Number of Pumps	1	2
Type	Non-clog, centrifugal	Non-clog, centrifugal
Capacity, each		1,300 gpm
Side 1 Firm Capacity		2,600 gpm (3.74 mgd)
Head		30 feet
Drive	Variable speed	Variable speed
Motor Size	50 hp	15 hp
Side 2 Bioreactors		
Number of Pumps	3 duty, 1 standby	
Type	Non-clog, centrifugal	
Capacity, each	2,230 gpm	
Side 2 Firm Capacity	6,720 gpm (9.63 mgd)	
Head	28 feet	
Drive	Variable speed	
Motor Size	25 hp	

7.15.2. Condition, Performance, and Capacity

- RAS pumped at 0.95 times the influent with good results; no operational change planned
- Two newer pumps for old clarifiers:
  - Good condition
  - Sized appropriately for RAS flows
- One older pump for old clarifiers:
  - Rebuilt since 1980 but may be reaching end of useful life
  - Oversized
  - Rarely used – for redundancy only
- Fixed capacity for RAS flows for the old clarifiers/bioreactor; no future capacity concerns
- Pumps for new clarifiers in good condition; no reported issues; expected to last through planning period
- New clarifier pumps sized to 1.25 times the design average flow of 7.8 mgd to the new bioreactors
  - Capacity exceeds projected flows for 2037

7.15.3. Conclusions

- The oldest RAS pump for the old clarifiers likely will require replacement in the planning period due to age and inappropriate size.

- The new RAS pumps are expected to last through the planning period with regular maintenance and service.
- There are no capacity concerns for the old or new RAS pumps.

7.16. SCUM PUMPING FROM BIOREACTORS AND FINAL CLARIFIERS

7.16.1. Description

- No scum collection from surface of Side 1 bioreactor
- Clarifier scum skimming on both sides by standard skimmers and scum beach
- Side 1 clarifier scum pump station:
  - Pneumatic diaphragm pump identical to the primary sludge and TWAS pumps
  - Installed in 1985
- Side 2 Bioreactor scum collection from the bioreactors by ducking skimmers:
  - Discharge to scum pit and gravity flow to scum pump station
  - Scum pump discharge via WAS line to WAS storage tank
  - Currently not operational
- Side 2 clarifier scum collected in scum pits, pumped to WAS storage tank by dedicated scum pump station and piping
  - Pumps installed in 2004

Table 7-19: Bioreactor and Final Clarifier Scum Pump Summary

Equipment Description	Bioreactor Scum	Clarifier Scum
Side 1 Bioreactors/Clarifiers		
Number of Pumps	1	1 (older)
Type	Pneumatic diaphragm	Pneumatic diaphragm
Capacity	100 gpm	100 gpm
Side 2 Bioreactors/Clarifiers		
Number of Pumps	2	2
Type	Pneumatic diaphragm	Pneumatic diaphragm
Capacity	100gpm	100 gpm

7.16.2. Condition, Performance, and Capacity

- Side 1 scum pump in good condition despite age; regular rebuild keeps pump updated
- Side 2 bioreactor scum skimmers never quite functioned as intended
  - Currently in need of repair to put back into service
  - Pumping through WAS line affects ability to flow WAS; air lock by scum and air trapped at high point in pipe, preventing WAS from establishing proper flow after scum pumping
- All scum pumps in good condition and expected to last through planning period
- Plant staff will monitor older pump to determine if replacement necessary

- No capacity changes expected to Side 1, therefore no pump capacity issues
- Capacity of Side 2 scum pumps sufficient to accommodate fourth clarifier and no capacity issues expected

7.16.3. Conclusions

- Scum pumps will likely not require replacement nor upsizing during the planning period.
- Plant staff will explore options for Side 2 scum skimming equipment repair or replacement.

7.17. UV DISINFECTION SYSTEM AND EFFLUENT FLOW MEASUREMENT

7.17.1. Description

During the 2004 WWTP upgrade, the existing chlorination system was replaced with UV disinfection. The existing chlorine contact basin was converted to accommodate the UV disinfection system. Slide gates control the inlet and two automatic level control gates control the system water level. Two banks of automatic self-cleaning low pressure high intensity UV are installed in the basin. A second channel with the same dimensions parallel to the operating UV system is available to house additional UV banks for future expansion. The level control gates serve both UV channels. The following summarizes the UV system:

- Installed in 2004; exterior installation
- One channel with two redundant banks; automatic chemical/mechanical cleaning
- Second channel available for future expansion
- Level control provided by two automatic level control (ALC) gates serving both channels
- Lamp type is older style – newer systems use more efficient and effective lamps; system could be retrofitted for increased capacity

Table 7-20: UV Disinfection System Configuration

Equipment Description	Size or Design Criteria
Automatic Level Control Gates	
Number	2 (serving both channels)
Hydraulic Design Flow (total)	13.8 mgd
UV Lamps	
Type	Low Pressure High Intensity
Number of Banks	2
Number of Modules	24
Number of Lamps	192
Max Power Demand	48 kW
Automatic Cleaning	Chemical/Mechanical
Treatment Design Flow	13.8 mgd

7.17.2. Condition, Performance, and Capacity

- System in good condition and operating as designed
- Algae growth in approach channel and UV system
- Electronic components replaced over the years as needed
- Lamps replaced approximately annually or as they fail; typical of UV lamps
- Per Trojan information, system sized to treat flows up to 13.8 mgd but capable of treating up to 15.5 mgd; peak hydraulic flow is 19.2 mgd
- Per plant staff, water level rises above lamps and above ballast boxes at flows over approximately 16 to 17 mgd even with ACL gates lifted out of flow; hydraulic limitations of channel geometry
- No permit violations throughout the high flow

7.17.3. Conclusions

- No changes to the UV system are currently needed but hydraulic upgrades may be needed if flows routinely exceed 16 mgd.
- If effluent sampling shows increasingly higher *E. coli* concentrations with increasing flows, planning for a system expansion should begin.
- If LED UV systems are developed within the planning period, system replacement should be evaluated at that time.
- Retrofitting system with newer Heraeus lamps could increase treatment capacity to 23.2 mgd

7.18. POPLAR PLANTATION EFFLUENT REUSE FACILITY

7.18.1. Description

The City of Missoula invested in a poplar plantation irrigated with treated WWTP effluent to further reduce nutrients discharged to the Clark Fork River. A pilot plot installed in 2007 proved to be promising and was followed by a full-scale plantation in 2013-14. The plantation covers 180 acres of which 152 acres are currently planted with about 72,000 hybrid poplar trees. Diversion of effluent to poplar irrigation began in August 2014. The flow rate was dictated by the water demand of the trees and the pump was sized accordingly. From 2014 through the 2018 season, 0.8 mgd were diverted from May through September. After the 2018 irrigation season, a larger pump was installed, increasing the irrigation flow to about 1.5 mgd as demand from the growing trees has increased. The pump is operated on a VFD and allows for reducing flows if less than full capacity is needed. The following summarizes the current poplar farm and irrigation system.

- Poplar farm management: Hybrid Energy Group, LLC
- Poplar farm area: 180 ac total; 152 ac currently planted; 72,000 trees
- Tree age for harvesting: 12-15 years
- Irrigation pump: single end suction centrifugal pump drawing from the effluent flow measurement structure
- Pump control by varying VFD (flow) to maintain constant discharge pressure

Table 7-21: Poplar Plantation and Irrigation Pump

Parameter	Value
Poplar Farm	
Total Area	180 ac
Planted Area	152 ac
Number of Trees	~72,000
Current Water Demand	1.3 – 1.5 mgd
Required Pressure at Plantation	45-60 psi
Expected Harvest Age	12-15 years
Irrigation Pump	
Number	1
Type	End suction centrifugal
Capacity	1,100 gpm
Total Dynamic Head	155 ft
Drive	Variable speed
Motor Size	60 hp

7.18.2. Condition, Performance, and Capacity

- Pump installed in late 2019 – no performance information available yet

- Capacity expected to be adequate for full growth and full planting on current plantation area
- Diversion of effluent from the river successful in reducing nutrient load to the Clark Fork River:

Table 7-22: Effluent Nutrient Reduction due to Poplar Irrigation

Parameter (Effluent Concentration)	0.8 mgd (2014-2017)	1.5 mgd (projected)
Total Phosphorous (0.7 mg/L)	4.7 lb/d	8.8 lb/d
Total Nitrogen (9.7 mg/L)	65 lb/d	121 lb/d

7.18.3. Conclusions

- The poplar plantation in its current configuration is operating as intended and no further upgrade plans are foreseeable.
- Potential expansion for additional effluent nutrient diversion is discussed in Chapter 8.

7.19. DISSOLVED AIR FLOTATION THICKENING UNIT

7.19.1. Description

The Missoula WWTP currently has two Envirex Model 45 Float-Treat dissolved air floatation (DAF) thickeners that were installed in 1974. The units consist of steel flotation basins, sludge feed pumps, a pressurized saturation tank and compressed air system and were originally equipped with a polymer chemical feed system that was not used and was removed during the 2004 plant upgrade. The system currently operates with EDUR DAF pumps that introduce micro bubbles to the water as they pump, eliminating the need for the pressurized saturation system, which is maintained for redundancy. The flotation basins are situated in the upper level of the sludge thickener room, while the saturation tank, pumps, air and chemical systems are located in the lower level next to the concrete sludge storage tank. The following summarizes the DAF system:

- Two DAF thickeners served by two multiphase, air-entraining pumps
- DAF thickeners installed in 1974
- Pumps may be directed to either DAF thickener or both pump to the same thickener
- Current operation has both pumps pumping to a single thickener with good results
- DAF thickeners are used without polymer addition
- Skimmer removes thickened sludge off the top for pumping to TWAS tank
- Subnatant returned by gravity flow to the head of the plant

Table 7-23: Dissolved Air Flotation Thickening Equipment

Parameter	Criteria or Capacity
Dissolved Air Flotation Thickener	
No. of Thickening Units	2
Maximum Solids Handling Capacity Per Unit	14,200 lb/day of dry solids
Nominal Recovery Rate	85 percent
Design Solids Loading Per Unit	11,700 lb/day of dry solids
Dimensions (each)	9.5 ft x 50 ft x 8.5 ft deep
Average Operating Depth	7.42 ft
Effective Surface Area of Each Flotation Basin	450 sf
Skimmer Drive Motor Size	0.5 hp

DAF Sludge Feed Pumps

- Two non-clog centrifugal pumps
- Intermittent pumping from WAS storage tank
- WAS blended with discharge from air-entraining pumps prior to discharge into the DAF thickener
- Pumps installed in 2004

Compressed Air System

- Replaced in 2004
- Two Atlas Copco Model GA22-100 air compressors and a 400-gal air receiver
- Each air compressor capacity: 131 cfm at 100 psig; equipped with 30 hp motor
- Not currently used; maintained for redundancy to the air entraining pumps

Air Entraining Pumps

- Three multiphase pumps installed in 2006, 2010, and 2019 to replace pressurized saturation tanks

TWAS Pumps

- Two pneumatic diaphragm pumps identical to the primary sludge and all scum pumps
- Installed in 1995 and serviced regularly

Table 7-24: DAF System Pumping Equipment

Parameter	Pump		
	DAF Sludge Feed Pumps	*Air Entrainment Pumps*	*TWAS Pumps*
Type	Horizontal Non-clog Centrifugal	Horizontal Centrifugal	Pneumatic Diaphragm
Number	2 (1 Duty, 1 Standby)	3 (2 duty, 1 standby)	2 (1 Duty, 1 Standby)
Rated Capacity (each)	250 gpm @ 25 ft	250 gpm @ 155 ft	75 gpm @ 20 ft
Drive	Electric variable speed	Constant speed	Pneumatic
Motor Size	7.5 hp	30 hp	N/A
Motor Speed	1,200 rpm	3500 rpm	N/A
Air Flow Requirement	N/A	N/A	40 cfm

7.19.2. Condition, Performance, and Capacity

- DAF thickener components including chains, scrapers, sprockets, and skimmer blades replaced as needed over the years but aging
- Steel tanks in fair condition; plant staff concerns over eventual corrosion and age-related problems that cannot be fixed by replacement of parts
- Capacity of the DAF thickeners in excess of current WAS production
- Air-entraining pumps at capacity with redundancy
- DAF sludge feed pumps in good condition; adequately sized for WAS flows expected at the design capacity of the plant

7.19.3. Conclusions

- There are no imminent capacity issues with the DAF thickening system.
- Based on the system age, planning should start immediately for replacing the system with a different thickening system such as thickeners offered as piggy-back units to the PW Tech dewatering volute press currently in operation at the plant. Alternatives are discussed in Chapter 8 of this Facility Plan.

7.20. TWAS STORAGE TANK

7.20.1. Description

- Dimensions: 25 ft wide x 25 ft long x 21 ft high with varying side water depth
- Aerated by dedicated blower installed in 2010
  - Dedicated blower with variable speed drive; capacity up to 350 scfm (full tank)
  - Aeration to prevent secondary phosphorous release and odors

- Served by a photoionization odor control unit installed in 2010

Table 7-25: TWAS Tank and Equipment

Equipment Description	Size or Design Criteria
TWAS Tank	
Basin Dimensions	25 ft long x 25 ft wide x 21 ft deep
Sludge Depth	0 - 18 ft
Blower	
Type	Rotary lobe
Motor Size	30 hp
Capacity	350 scfm
Odor Control System	
Type	Photoionization
Manufacturer	Neutralox
Peak Inlet Concentration	10 ppm H_2S
System Performance	99% H_2S removal
Rated Capacity	1,400 cfm

7.20.2. Condition, Performance, and Capacity

- Storage volume adequate for current plant design capacity
- Storage volume for 2037 conditions expected to be adequate; dewatering equipment flow rates to accommodate increasing TWAS volumes
- Effectiveness of prevention of secondary phosphorous release not quantified due to lack of data

7.20.3. Conclusions

- The TWAS tank and associated equipment are not in need of repair or replacement.
- The size of the TWAS tank is adequate for the design capacity of the plant and likely through the planning period.
- Future sizing and strategies for use with potential sludge thickening options will be evaluated.

7.21. FERMENTER

7.21.1. Description

In order to achieve high efficiencies of phosphorous removal in the activated sludge process, an adequate food source for the phosphorous accumulating organisms (PAOs) must be provided. Primary sludge fermentation is an efficient way of providing volatile fatty acids for increasing the rate of phosphorous release in the anaerobic zone, which then favors a greater phosphorous uptake in the aerobic zones.

The Missoula WWTP has one fermenter which was converted from a digester during the last plant upgrade in 2004. Capacity is limited by the available fermenter volume which dictates the solids retention time (SRT). Typically, an SRT of six days is used to produce the optimal amount of VFAs. A shorter SRT will not make use of the full potential of VFA production, while during a longer residence time methanogenic bacteria will begin to convert the VFAs to methane and other byproducts. The fermenter is currently fed with about 50 percent of the primary sludge generated in the plant and has additional capacity beyond the current sludge flows.

Elutriation water is diverted from the primary clarifier effluent to achieve the target HRT in the fermenter. The elutriation water dissolves VFAs as it passes through the sludge blanket and helps release VFAs otherwise trapped in the floc. The supernatant rich in VFAs and intermediate products flows by gravity to the anaerobic zones in the bioreactors. Fermented sludge is pumped to the primary and secondary digesters with a pneumatic diaphragm pump. The following summarizes the fermenter system:

- One fermenter, converted from digester in 2004 and internally coated with corrosion-resistant coating system; originally constructed in 1961; fitted with fixed covers in 1982, received corrosion resistant interior lining, exterior insulation and cladding in 1994
- Supernatant rich in VFAs used in biological phosphorous removal in the bioreactors
- Flow by gravity to bioreactors
- Fermented sludge pumped to digesters by pneumatic diaphragm pump identical to primary sludge, scum, and TWAS pumps

Table 7-26: Fermenter Design Summary

Parameter	Value
Fermenter	
Diameter / SWD	50 ft / 25 ft
Volume	367,000 gal
Average Sludge Blanket Depth	7.3 ft
Elutriation Flow (supernatant)	~150 gpm (max 500 gpm)
Design HRT	~12 hrs
Current Average HRT	~35 hrs
Design SRT	6 days
Current Average SRT	5 days
Sludge Transfer Pump	
Number	1
Type	Pneumatic diaphragm
Capacity	100 gpm
Head	45 feet

7.21.2. Condition, Performance, and Capacity

- Good structure and equipment condition – no concerns about equipment age or deterioration
- Fermenter capacity adequate for current flows and loads
- Fermenter use: 50% of primary sludge
    - Capacity exists for up to 48,000 gpd of primary sludge at 6-day SRT
    - Capacity adequate for projected 2037 flows

- Very good biological phosphorous removal confirms adequate VFA production
    - 2018 data: approximately 30 mg/L VFAs in bioreactor influent, assuming 20 mg/L VFAs in plant influent
- No condition or capacity issues for the sludge transfer pump for current or future flows

7.21.3. Conclusions

- Changes to fermenter operation and capacity are not likely to be needed during the planning period.
- Operation with a thicker sludge blanket accommodating a larger primary sludge mass may be needed at higher flows and loads to produce the required VFAs to treat the higher phosphorous loading to the process.

7.22. PRIMARY AND SECONDARY DIGESTERS

7.22.1. Description

The primary digester was originally constructed in 1982 and has been in service as a primary digester since. No information is available about upgrades or major maintenance since installation but it may have received cleaning and maintenance in 2004 as part of the overall plant upgrade. Portions of the gas handling system were upgraded during the 2004 project to eliminate gas leakage and improve efficiency. The secondary digester is the second of the pair originally constructed in 1961. The first basin was converted to the above-discussed fermenter, while the second is utilized as a secondary digester.

The primary digester is heated and mechanically mixed. The secondary digester is also mechanically mixed but not heated and essentially serves as a storage tank providing more flexibility for dewatering processes. The capability to heat the secondary digester is in place; however, plant staff noted that heating the digester in the past did not result in increased volatile solids destruction and was consequently discontinued. The following summarizes information about the digesters:

- Primary digester constructed in 1982; fixed covers; heated; mixed by air sparging system
- Secondary digester constructed in 1961; fitted with fixed covers in 1982, received corrosion resistant interior lining, exterior insulation and cladding in 1994; unheated; equipped with mechanical mixing system
- Digester gas handling system upgraded in 2004
- Digesters receive primary sludge and fermented primary sludge
- No facilities for sending secondary sludge to the digesters

Table 7-27: Digester Design Summary

Parameter	Value
Primary Digester	
Diameter / Depth	65 ft / 28 ft
Volume	695,000 ft^3
Temperature	34°C
Mixing System	Air Sparging
Secondary Digester	
Diameter / Depth	50 ft/ 25 ft
Volume	367,000 ft^3
Temperature	Not heated
Mixing System	Mechanical

- Digester gas use:
  - One digester gas compressor (boiler); installed in 1982; heating of primary digester and administrative building
  - One co-generation unit; installed in 2018; heating of primary digester and administration building; electricity generation
  - Early data from co-generation unit to be evaluated for performance and energy savings
- Gas scrubber installed in 2004
- Flare used for excess digester gas; replaced in 2004

7.22.2. Condition, Performance, and Capacity

- Ongoing monitoring of visible portions of digester interiors for deterioration; no concerns over deterioration at this time
- Cleaning, replacement of valves, roof coating and insulation, and aeration diffusers planned for 2020 for primary digester
- Ongoing monitoring of air sparge piping to detect clogs caused by settled grit or debris
  - Clogged supply pipe to trigger major digester cleaning event
  - Most recent clogged pipe and major cleaning ~20 years ago
- Circular DEQ-2 requirement: maximum volatile suspended solids (VSS) loading of 80 pounds per 1,000 cubic feet of digester volume per day
  - Digester influent and effluent data for 2015 through 2017: average loading of 130 lb/1,000 cf/d
  - Digester capacity not sufficient for true digestion if full solids stabilization is required
  - Digester capacity plus composting adequate for solids stabilization per Federal code 40 CFR 503
- The co-generation unit replaces digester gas boiler for heating of digester and administration building
- Digester boiler maintained for redundancy to co-generation unit
- Additional redundancy for heating the digester and administration building provided by natural gas boiler

7.22.3. Conclusions

- Plant staff are monitoring the physical condition of digesters and no immediate repair or replacement concerns exist. The digesters are expected to last through the planning period.
- The current capacity is adequate for plant solids production and the current sludge disposal method, which includes composting. Sludge would not meet 40 CFR 503 requirements without composting.
- Operational capacity for future flows will be more limited and less solids destruction will be provided by the digesters, relying more heavily on composting as a means to meet solids stabilization requirements. Composting has become a permanent process in WWTP solids management.

7.23. DEWATERING SYSTEMS

7.23.1. Description

Digested Sludge and TWAS Tank Transfer Pumps

- Two sets of three positive displacement rotary lobe pumps installed in 2004
- One pump of each set replaced with larger pump in 2006 to accommodate higher flow rates of the centrifuge
- Currently use smaller 2004 pumps to pump to volute press

Table 7-28: Digested Sludge and TWAS Tank Transfer Pumps

Parameter	Value
Digested Sludge	
Number	2 / 1
Type	Rotary lobe
Capacity	50 / 300 gpm
Head	35 / 152 ft
Drive	Variable speed
Motor Size	3 / 20 hp
TWAS Tank	
Number	2 / 1
Type	Rotary lobe
Capacity	50 / 300 gpm
Head	35 / 116 ft
Drive	Variable speed
Motor Size	3 / 25 hp

Centrifuge

- Centrifuge installed in 2006 to replace three belt filter presses (BFPs)

- High capacity
- Fewer labor hours required compared to BFPs
- More concentrated return flows to process

Table 7-29: Centrifuge Performance Criteria

Parameter	Centrifuge
Centrifuge	
Number	1
Flow Rate, gpm	250
Solids Loading, dry lbs/hour	2,200
Minimum Dewater Sludge Solids, %	16
Minimum Solids Capture, %	95

Volute Press

- Volute press installed in 2016; currently only two of three possible volutes installed
- Lower capacity than centrifuge; longer runtimes
- Less concentrated return flows to process
- Fewer labor hours required compared to centrifuge

Table 7-30: Volute Press Performance Criteria

Parameter	Centrifuge
Volute Press	
Number	1
Flow Rate, gpm	70
Solids Loading, dry lbs/hour	700
Minimum Cake Solids, %	15
Minimum Solids Capture, %	≥95

Polymer Delivery System

- New skid-mounted polymer feed system installed with new volute press in 2016
  - Uses neat polymer supplied in totes
  - Capacity matches that of volute press at full buildout with three volutes installed
- Older polymer system maintained as backup for feeding centrifuge
  - Uses dry polymer
  - Consists of wetting and feeding unit, mixing tanks, aging tanks, feed pumps
  - Has higher capacity than neat polymer skid to match higher throughput of centrifuge

7.23.2. Condition, Performance, and Capacity

- Sludge transfer pumps adequately sized for plant design sludge production with redundancy
- Capacity dictated by dewatering process equipment flow rates; pump capacity in excess of sludge production rates
- Volute press has lower flows than centrifuge, so no capacity concerns
- Pumps expected to last through the planning period
- The volute press is the primary dewatering equipment
    - Sized for plant design solids production with expansion capability to increase capacity by 50%
    - No operational issues; low labor requirements free up staff for other tasks
- Centrifuge provides redundancy to the volute press; used only when the press is being serviced
- Centrifuge nears end of serviceable life in 2026
- Polymer system operating well; no reported issues
    - System sized to match full buildout capacity of the volute press; no capacity concerns

7.23.3. Conclusions

- The overall dewatering capacity meets or exceeds current plant design sludge production.
- Modeling output for sludge production at 2037 conditions will be compared to equipment capacities to determine if additional capacity will be needed.
- Centrifuge replacement will be evaluated in concert with options for replacement of the DAF thickeners for compatibility and capacity and should be included in a future thickening and dewatering evaluation.

7.24. WWTP SCADA SYSTEM

The WWTP uses a central system control and data acquisition (SCADA) system to coordinate all automated and monitored plant functions. In recent years, the WWTP SCADA system was migrated from an outdated Windows XP-based system called “Lookout” to “Factory Talk” by Rockwell Automation. After some initial trouble during system migration, Factory Talk has reportedly been working well and no major upgrades are planned at this time. The system is supported by a local integrator who is available when the system needs modifications.

The typical life for these electronics-based systems is ten to 15 years and replacement of upgrading within the planning period will likely be necessary. At that time, it may be prudent to consider upgrading the water and wastewater systems to the same software to allow for efficiency in staff training, as well as overall uniformity. The water and wastewater systems do not currently utilize the same SCADA platform.

7.25. WWTP ENERGY EFFICIENCY

An in-depth energy analysis of the WWTP was not part of this Facility Plan; however, as the city of Missoula is energy conscious, a thorough analysis may be beneficial for identifying equipment that could be replaced with newer, more energy efficient units and evaluate the costs and benefits of replacement. The bioreactor blowers are some of the largest energy consumers, if not the largest, and interest in energy savings through replacement with more energy efficient blowers has been expressed in the past. Preliminary blower analyses were completed in 2010 and 2012, both showing that the cost of investment in new blowers outpaced the savings through energy efficiency. At this time, seven years later, it may be worth taking another look to see if an in-depth energy study would be warranted.

7.26. COMPOSTING

7.26.1. Description

- Formerly EKO Compost acquired by city of Missoula in 2016
- Composting site adjacent to WWTP; 32.45 acre parcel
- Employs bunker aerated static pile method
    - Transitional facility uses rectangular layout; ultimately to be transitioned to a radial layout
    - Natural soil bottom (not impervious surface)
- 2017 equipment purchase to replace inadequately sized, old, or otherwise needed equipment
- Replacement trammel screen planned for future CIP list
- Ongoing improvements to fully implement system as proposed in 2016 Biosolids & Green Waste Management Study (Anderson-Montgomery Consulting Engineers, 2016)

Table 7-31: Composting Equipment Summary

Equipment	Number / Size
Composting Facility	
Front End Loaders	4 / 4 cy
Trammel Screens	2
Tub Grinders	1 (1 backup)
Shaker Screen	1
Rotary Mixer	1
Blowers	2 / 40 hp

7.26.2. Condition, Performance, and Capacity

- Facility historically not optimally managed as EKO Compost
- City working to reduce backlog of material, replace aging or undersized equipment
- Odors still a nuisance but reportedly reduced due to City’s efforts
- Acceptance of WWTP sludge guaranteed
- Capacity to increase with ongoing facility improvements

7.26.3. Conclusions

- The purchase of the composting facility assured that the WWTP sludge will be processed in accordance with 40 CFR 503 regulations without the need for additional digester volume.
- Ongoing investment into improving operations at the composting facility will further reduce odors.
- Regulations will be monitored regarding requirement for impervious surfaces in the future and upgrades will be made as needed. A compost CIP, Phase 2 Radial Bunker System with an impervious concrete pad is part of the City's budget planning.
- No further analysis of capacity and performance of the composting operations will be provided as part of this Wastewater Facility Plan and 2016 Biosolids & Green Waste Management Study will be incorporated by reference.

7.27. SUMMARY OF UNIT PROCESS CAPACITIES

The following table lists unit processes and associated equipment and their capacities. Capacities of older equipment may be estimated. Capacities are given with the largest unit out of service.

Table 7-32: Summary of Unit Process Capacities

Unit Process	Current Unit Process Capacities	Comments
Mechanical Bar Screens	Hydraulic capacity (each): 12.5 mgd	
Grit Chamber	Hydraulic capacity (each): 13 mgd	
Influent Lift Pumps Firm Capacity	Design: 20 mgd Observed firm capacity: ~14 mgd; Observed total capacity: ~20 mgd	Capacity issues currently being addressed
Primary Clarifiers	ADF: 11.0 mgd Loading limited; not critical to overall treatment performance	No capacity concerns for 2037
Primary Effluent Lift Station – Side 1 Pumps	Firm capacity: 16 mgd	No capacity concerns for 2037
Primary Effluent Lift Station – Side 2 Pumps	Firm capacity: 9.3 mgd 13.4 mgd with 2 large pumps; ~15 mgd with all 3 pumps	Replace small pump with larger for increased capacity; firm capacity still below 2037 peak hour flows
Aeration Basins – Side 1 Bioreactors (Listed flows refer to treatment capacity of the bioreactor. Capacity depends on desired effluent nutrient concentrations)	ADF: 5.2 mgd MMF: 6.0 mgd MDF: 6.8 mgd PHF: 8.3 mgd	No capacity concerns for 2037 flows; however process configuration not adequate for higher nutrient removal. See Chapter 8.
Aeration Basins – Side 2 Bioreactors (Listed flows refer to treatment capacity of the bioreactor. Capacity depends on desired effluent nutrient concentrations)	ADF: 6.8 mgd MMF: 7.8 mgd MDF: 8.8 mgd PHF: 10.9 mgd	See Chapter 8 for process alternatives addressing lower effluent limits and associated capacity limitations
Secondary Clarifiers – Side 1 Bioreactors	Same as bioreactors above	No capacity concerns for 2037
Secondary Clarifiers – Side 2 Bioreactors	Same as bioreactors above	Need additional clarifier for 2037
Disinfection System (Listed capacities refer to treatment and hydraulic capacity)	ADF: 12.0 mgd MMF: 13.8 mgd MDF: 15.6 mgd	Capacity increase needed to hydraulically accommodate 2037 peak flows.
WAS Thickening (DAF Thickener)	Max solids per day: 14,200 lb	Replacement discussed in Chapter 8
Fermenter	48,000 gpd of primary sludge at 3% - 4% solids VFA output sufficient for current and 2037 bio-P removal	No capacity concerns for 2037
Anaerobic Digester[1] (65 ft tank)	Capacity not critical due to composting	No capacity concerns for 2037
Anaerobic Digester[1] (50 ft tank)	Capacity not critical due to composting	No capacity concerns for 2037
Dewatering[2] *- Centrifuge and volute press*	Solids throughput: 700 lb/hr	No capacity concerns for 2037

1. Used for primary and fermented sludge only.
2. Running blend of digested sludge and TWAS.
Abbreviations: ADF, average day flow; MMF, maximum month flow; MDF, maximum day flow; PHF, peak hour flow.

7.27.1. Conclusions

- Condition:
  - The vast majority of equipment and structures is in good condition and expected to last through the planning period. Exceptions are a couple of older pumps, currently used only as standby equipment, and the WAS thickening equipment. Replacement alternatives for WAS thickening equipment are discussed in Chapter 8.
  - Plant staff keep abreast of aging equipment through an active service, rebuild, and replacement program.
- Performance:
  - Overall, the plant has been operating reliably, producing effluent that consistently meets permit requirements.
  - Among individual pieces of equipment, only the influent pumps have been showing performance issues, which are currently being addressed.
- Hydraulic Capacity:
  - The plant in its current configuration will be adequate to handle 2037 average and maximum month flows and loads, given the same effluent requirements.
  - Additional hydraulic capacity may need to be developed toward the end of the planning period for maximum day and peak hour flow events; continued monitoring of extreme flow events should be used to determine the need for additional capacity.
  - Detailed hydraulic analysis should be used to determine upgrades necessitated by hydraulic restrictions.
- Treatment Capacity:
  - At current effluent nutrient limits, the plant in its present configuration should be able to produce effluent that meets the permit limits up to an annual average influent flow of about 9.2 mgd projected to occur by 2027.
  - At flows above 9.2 mgd, operational strategies may need to change to keep the plant in compliance with nutrient limits, and planning for process upgrades should be under way.
  - At lower nutrient limits, process upgrades will be necessary to achieve adequate nutrient removal. Recommendations for changing effluent limits are included in Chapter 8 of the Facility Plan.

7.28. REFERENCES

Anderson-Montgomery Consulting Engineers. (2016). *Missoula Biosolids & Green Waste Management Study.*

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APPENDIX 7-1

WASTEWATER TREATMENT PLANT HYDRAULIC PROFILE CALCULATIONS

HYDRAULIC PROFILE CALCULATOR

By: Rika Lashley Date: 4/3/2018
Chkd: WWW, ECS Date: 1/15/2019
Project No.: 1657.039

Project: 2018 Missoula Wastewater Facility Plan
Subject: Hydraulic Profile - UV System to Outfall

Scenarios

Scenario	Description	Flow
1	2017 Average Flow	7.27 mgd
2	2017 Maximum Month Flow	9.32 mgd
3	2017 Maximum Day Flow	11.32 mgd
4	2017 Peak Hour Flow	18.2 mgd
5	2037 Average flow	11.2 mgd
6	2037 Maximum Month Flow	14.3 mgd
7	2037 Maximum Day Flow	17.4 mgd
8	2037 Peak Hour Flow	26.6 mgd
9	Plant Design Average Flow	12.0 mgd
10	Plant Design Peak Hour Flow	19.2 mgd

Assumptions:
Starting water level is 100-yr flood from 2002 WWTP Record Drawings.
No changes in plant configuration for 2037.

Initial Values	Scenario	1	2	3	4	5	6	7	8	9	10
	Plant Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
	RAS Flow, mgd										
	Starting Water Level, ft	3134.10	3134.10	3134.10	3134.10	3134.10	3134.10	3134.10	3134.10	3134.10	3134.10

Pressure Line		**Scenario**	1	2	3	4	5	6	7	8	9	10
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
Number:	1	Downstream Water Level, ft	3134.10	3134.10	3134.10	3134.10	3134.10	3134.10	3134.10	3134.10	3134.10	3134.10
Description:	36" Pipe from outfall structure to outfall headwall											
# of Total:	1 of 1	Velocity, ft/s	1.59	2.04	2.48	3.98	2.45	3.13	3.81	5.82	2.63	4.20
		Velocity Head, ft	0.04	0.06	0.10	0.25	0.09	0.15	0.23	0.53	0.11	0.27
Dimensions		Friction Loss, ft/1000ft	0.19	0.29	0.42	1.02	0.41	0.65	0.94	2.05	0.47	1.12
Diameter, in	36	Friction Loss, ft	0.01	0.01	0.01	0.03	0.01	0.02	0.03	0.06	0.01	0.03
Length, ft	30	Minor Loss, ft	0.06	0.10	0.14	0.37	0.14	0.23	0.34	0.79	0.16	0.41
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Friction Coefficient, C	150	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Minor Loss Coefficient, K	1.50											
Downstream Invert Elev, ft	3124.00	Total Loss, ft	0.06	0.11	0.16	0.40	0.15	0.25	0.37	0.85	0.17	0.44
Upstream Invert Elev, ft	3124.00											
		Upstream Water Level, ft	3134.16	3134.21	3134.26	3134.50	3134.25	3134.35	3134.47	3134.95	3134.27	3134.54

Open Channel		**Scenario**	1	2	3	4	5	6	7	8	9	10
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
Number:	2	Downstream Water Level, ft	3134.16	3134.21	3134.26	3134.50	3134.25	3134.35	3134.47	3134.95	3134.27	3134.54
Description:	Oufall structure											
# of Total:	1 of 1	Channel Slope, %	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		Critical Depth, ft	0.48	0.56	0.64	0.88	0.64	0.75	0.86	1.13	0.67	0.91
Dimensions		Normal Depth, ft	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Width, ft	6	Downstream Depth, ft	9.16	9.21	9.26	9.50	9.25	9.35	9.47	9.95	9.27	9.54
Length, ft	16	Downstream Velociy, ft/s	0.20	0.26	0.32	0.49	0.31	0.39	0.47	0.69	0.33	0.52
Friction Coefficient, n	0.013	Upstream Depth, ft	9.16	9.21	9.26	9.50	9.25	9.35	9.47	9.95	9.27	9.55
Downstream Invert Elev, ft	3125.00	Upstream Velocity, ft/s	0.20	0.26	0.32	0.49	0.31	0.39	0.47	0.69	0.33	0.52
Upstream Invert Elev, ft	3125.00											
		Total Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		Upstream Water Level, ft	3134.16	3134.21	3134.26	3134.50	3134.25	3134.35	3134.47	3134.95	3134.27	3134.55

Pressure Line		**Scenario**	1	2	3	4	5	6	7	8	9	10
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
Number:	3	Downstream Water Level, ft	3134.16	3134.21	3134.26	3134.50	3134.25	3134.35	3134.47	3134.95	3134.27	3134.55
Description:	48" PLE - effluent pipe											
# of Total:	1 of 1	Velocity, ft/s	0.89	1.15	1.39	2.24	1.38	1.76	2.14	3.27	1.48	2.36
		Velocity Head, ft	0.01	0.02	0.03	0.08	0.03	0.05	0.07	0.17	0.03	0.09
Dimensions		Friction Loss, ft/1000ft	0.05	0.08	0.12	0.28	0.12	0.18	0.26	0.57	0.13	0.31
Diameter, in	48	Friction Loss, ft	0.01	0.02	0.03	0.06	0.03	0.04	0.06	0.13	0.03	0.07
Length, ft	225	Minor Loss, ft	0.02	0.04	0.06	0.14	0.05	0.09	0.13	0.31	0.06	0.16
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Friction Coefficient, C	140	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Minor Loss Coefficient, K	1.84											
Downstream Invert Elev, ft	3125.00	Total Loss, ft	0.03	0.06	0.08	0.21	0.08	0.13	0.19	0.44	0.09	0.23
Upstream Invert Elev, ft	3125.00											
		Upstream Water Level, ft	3134.20	3134.26	3134.34	3134.71	3134.33	3134.48	3134.66	3135.39	3134.37	3134.78

HYDRAULIC PROFILE CALCULATOR

By: Rika Lashley Date: 4/3/2018
Chkd: WWW, ECS Date: 1/15/2019
Project No.: 1657.039

Project: 2018 Missoula Wastewater Facility Plan
Subject: Hydraulic Profile - UV System to Outfall

Rectangular Weir		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
Number:	4	Downstream Water Level, ft	3134.20	3134.26	3134.34	3134.71	3134.33	3134.48	3134.66	3135.39	3134.37	3134.78
Description:	Final Effluent Measurement Structure (FEMS) with Rectangular Weir											
# of Total:	1 of 1	Flow Condition	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Sbmrgd	Normal	Normal
		Free Fall, ft	0.70	0.64	0.56	0.19	0.57	0.42	0.24	0.00	0.53	0.12
Dimensions		Head Over Weir, ft	0.50	0.59	0.67	0.92	0.67	0.79	0.90	1.27	0.70	0.96
Weir Length, ft	10											
Weir Type	Contracted	Total Loss, ft	1.20	1.23	1.23	1.12	1.23	1.21	1.14	0.79	1.23	1.08
Weir Crest Elevation ft	3134.90											
		Upstream Water Level, ft	3135.40	3135.49	3135.57	3135.82	3135.57	3135.69	3135.80	3136.17	3135.60	3135.86

Pressure Line		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
Number:	5	Downstream Water Level, ft	3135.40	3135.49	3135.57	3135.82	3135.57	3135.69	3135.80	3136.17	3135.60	3135.86
Description:	48" Pipe from UV Channel to FEMS											
# of Total:	1 of 1	Velocity, ft/s	0.89	1.15	1.39	2.24	1.38	1.76	2.14	3.27	1.48	2.36
		Velocity Head, ft	0.01	0.02	0.03	0.08	0.03	0.05	0.07	0.17	0.03	0.09
Dimensions		Friction Loss, ft/1000ft	0.05	0.08	0.12	0.28	0.12	0.18	0.26	0.57	0.13	0.31
Diameter, in	48	Friction Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Length, ft	7	Minor Loss, ft	0.02	0.03	0.05	0.12	0.04	0.07	0.11	0.25	0.05	0.13
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Friction Coefficient, C	140	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Minor Loss Coefficient, K	1.50											
Downstream Invert Elev, ft	3125.00	Total Loss, ft	0.02	0.03	0.05	0.12	0.05	0.07	0.11	0.25	0.05	0.13
Upstream Invert Elev, ft	3125.00											
		Upstream Water Level, ft	3135.42	3135.52	3135.62	3135.94	3135.61	3135.76	3135.90	3136.43	3135.65	3135.99

Circular Orifice (Submerged)		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
Number:	6	Downstream Water Level, ft	3135.42	3135.52	3135.62	3135.94	3135.61	3135.76	3135.90	3136.43	3135.65	3135.99
Description:	48" x 48" Opening in UV Effluent Box											
# of Total:	1 of 1	Velocity, fps	0.71	0.91	1.10	1.77	1.09	1.39	1.69	2.59	1.17	1.87
		Velocity Head, ft	0.01	0.01	0.02	0.05	0.02	0.03	0.04	0.10	0.02	0.05
Dimensions												
Diameter, ft	4.5	Total Loss, ft	0.01	0.02	0.03	0.08	0.03	0.05	0.07	0.16	0.03	0.08
Coefficient of Dischage	0.8											
Modeled as circular orifice w/ equiv. area		**Upstream Water Level, ft**	3135.43	3135.54	3135.65	3136.02	3135.64	3135.81	3135.97	3136.59	3135.68	3136.07

Rectangular Weir		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
Number:	7	Downstream Water Level, ft	3135.43	3135.54	3135.65	3136.02	3135.64	3135.81	3135.97	3136.59	3135.68	3136.07
Description:	Weir Wall in UV Effluent Box											
# of Total:	1 of 1	Flow Condition	Sbmrgd	Sbmrgd	Sbmrgd	Sbmrgd	Sbmrgd	Sbmrgd	Sbmrgd	Sbmrgd	Sbmrgd	Sbmrgd
		Free Fall, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Dimensions		Head Over Weir, ft	3.43	3.54	3.65	4.03	3.64	3.81	3.98	4.61	3.68	4.08
Weir Length, ft	13.5											
Weir Type	Contracted	Total Loss, ft	0.00	0.00	0.00	0.01	0.00	0.00	0.01	0.02	0.00	0.01
Weir Crest Elevation ft	3132.00											
		Upstream Water Level, ft	3135.43	3135.54	3135.65	3136.03	3135.64	3135.81	3135.98	3136.61	3135.68	3136.08

Rectangular Weir		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
Number:	8	Downstream Water Level, ft	3135.43	3135.54	3135.65	3136.03	3135.64	3135.81	3135.98	3136.61	3135.68	3136.08
Description:	UV System Weir - uses variable orifice flap gates											
# of Total:	1 of 1	Flow Condition	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Sbmrgd	Normal	Sbmrgd
		Free Fall, ft	0.65	0.54	0.43	0.05	0.44	0.27	0.10	0.00	0.40	0.00
Dimensions		Head Over Weir, ft	0.52	0.62	0.70	0.97	0.70	0.82	0.94	1.34	0.73	1.00
Weir Length, ft	9.33	Two gates combined										
Weir Type	Contracted	Total Loss, ft	1.17	1.16	1.14	1.02	1.14	1.10	1.04	0.81	1.13	1.00
Weir Crest Elevation ft	3136.08	This is the TOW elevation under the flap gates										
Assume gates wide open		**Upstream Water Level, ft**	3136.60	3136.70	3136.78	3137.05	3136.78	3136.90	3137.02	3137.42	3136.81	3137.08

HYDRAULIC PROFILE CALCULATOR

By: Rika Lashley Date: 4/3/2018
Project: 2018 Missoula Wastewater Facility Plan Chkd: WWW, ECS Date: 1/15/2019
Subject: Hydraulic Profile - UV System to Outfall Project No.: 1657.039

Additional Headloss		Scenario	1	2	3	4	5	6	7	8	9	10
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
Number:	9	Downstream Water Level, ft	3136.60	3136.70	3136.78	3137.05	3136.78	3136.90	3137.02	3137.42	3136.81	3137.08
Description:	UV System Headloss - estimated											
# of Total:	1 of 1	Est. HL through 2 UV banks	0.04	0.14	0.25	0.60	0.24	0.40	0.56	1.00	0.28	0.65
Dimensions												
		Total Loss, ft	0.04	0.14	0.25	0.60	0.24	0.40	0.56	1.00	0.28	0.65
Ballast elev - 3137.33												
Channel bottom - 3134.41		**Upstream Water Level, ft**	3136.64	3136.84	3137.03	3137.65	3137.02	3137.30	3137.58	3138.42	3137.09	3137.73

Rectangular Orifice (Submerged)		Scenario	1	2	3	4	5	6	7	8	9	10
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
Number:	10	Downstream Water Level, ft	3136.64	3136.84	3137.03	3137.65	3137.02	3137.30	3137.58	3138.42	3137.09	3137.73
Description:	UV Channel Inlet Gate											
# of Total:	1 of 1	Velocity, ft/s	0.75	0.96	1.17	1.88	1.16	1.47	1.79	2.74	1.24	1.98
		Velocity Head, ft	0.01	0.01	0.02	0.05	0.02	0.03	0.05	0.12	0.02	0.06
Dimensions												
Width, ft	5											
Length, ft (height)	3	Total Loss, ft	0.01	0.02	0.03	0.09	0.03	0.05	0.08	0.18	0.04	0.10
Coefficient of Dischage	0.8											
		Upstream Water Level, ft	3136.66	3136.86	3137.07	3137.73	3137.05	3137.36	3137.66	3138.60	3137.13	3137.83

Open Channel		Scenario	1	2	3	4	5	6	7	8	9	10
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
Number:	11	Downstream Water Level, ft	3136.66	3136.86	3137.07	3137.73	3137.05	3137.36	3137.66	3138.60	3137.13	3137.83
Description:	Old Chlorine Contact Basin											
# of Total:	1 of 1	Channel Slope, %	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		Critical Depth, ft	0.31	0.36	0.41	0.56	0.41	0.48	0.55	0.72	0.43	0.58
Dimensions		Normal Depth, ft	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Width, ft	11.75	Downstream Depth, ft	11.66	11.86	12.07	12.73	12.05	12.36	12.66	13.60	12.13	12.83
Length, ft	35	Downstream Velociy, ft/s	0.08	0.10	0.12	0.19	0.12	0.15	0.18	0.26	0.13	0.20
Friction Coefficient, n	0.013	Upstream Depth, ft	11.66	11.86	12.07	12.73	12.05	12.36	12.66	13.60	12.13	12.83
Downstream Invert Elev, ft	3125.00	Upstream Velocity, ft/s	0.08	0.10	0.12	0.19	0.12	0.15	0.18	0.26	0.13	0.20
Upstream Invert Elev, ft	3125.00											
		Total Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		Upstream Water Level, ft	3136.66	3136.86	3137.07	3137.73	3137.05	3137.36	3137.66	3138.60	3137.13	3137.83

HYDRAULIC PROFILE CALCULATOR

Project: 2018 Missoula Wastewater Facility Plan
Subject: WWTP Hydraulic Profile - Side 1 Primary Lift Pumps to UV System

By: Rika Lashley Date: 4/3/2018
Chkd: WWW, ECS Date: 1/15/2019
Project No.: 1657.039

Scenarios

Scenario	Description	Flow
1	2017 Average Flow	7.27 mgd
2	2017 Maximum Month Flow	9.32 mgd
3	2017 Maximum Day Flow	11.32 mgd
4	2017 Peak Hour Flow	18.2 mgd
5	2037 Average flow	11.2 mgd
6	2037 Maximum Month Flow	14.3 mgd
7	2037 Maximum Day Flow	17.4 mgd
8	2037 Peak Hour Flow	26.6 mgd
9		
10		

Flow Split:

Side 1 bioreactors/clarifiers	37%	
Side 2 bioreactors/clarifiers	63%	
Side 1 maximum month flow	6.0	mgd
Side 1 maximum day flow	6.8	mgd
Side 1 peak hour flow	8.3	mgd

Assumptions:
- No changes in plant configuration for 2037.
- Only two seondary clarifiers in use for all scenarios.
- Elevations largely taken from 2002 WWTP Record Drawings.
- RAS capped at pump station firm capacity (3.7 mgd).

Initial Values	Scenario	1	2	3	4	5	6	7	8	9	10
	Plant Flow, mgd	2.69	3.45	4.19	6.73	4.14	6.00	6.80	8.30		
	RAS Flow, mgd	2.42	3.10	3.70	3.70	3.70	3.70	3.70	3.70		
	Starting Water Level, ft	3136.66	3136.86	3137.07	3137.73	3137.05	3137.36	3137.66	3138.60	3137.13	3137.83

Pressure Line		Scenario	1	2	3	4	5	6	7	8	9	10
		Flow Factor	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50		
Segment	RAS = 0	Flow, mgd	1.34	1.72	2.09	3.37	2.07	3.00	3.40	4.15		
Number:	1	Downstream Water Level, ft	3136.66	3136.86	3137.07	3137.73	3137.05	3137.36	3137.66	3138.60	3137.13	3137.83
Description:	42" SE from Clarifier Effluent Junction Box to UV basin											
# of Total:	1 of 1	Velocity, ft/s	0.22	0.28	0.34	0.54	0.33	0.48	0.55	0.67		
		Velocity Head, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01		
Dimensions		Friction Loss, ft/1000ft	0.01	0.01	0.01	0.03	0.01	0.03	0.03	0.05		
Diameter, in	42	Friction Loss, ft	0.00	0.00	0.00	0.01	0.00	0.01	0.01	0.01		
Length, ft	240	Minor Loss, ft	0.00	0.00	0.00	0.01	0.00	0.01	0.01	0.01		
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		
Friction Coefficient, C	120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
Minor Loss Coefficient, K	1.55											
Downstream Invert Elev, ft	3125.00	Total Loss, ft	0.00	0.00	0.01	0.01	0.01	0.01	0.02	0.02		
Upstream Invert Elev, ft	3125.00											
		Upstream Water Level, ft	3136.66	3136.86	3137.07	3137.75	3137.06	3137.37	3137.67	3138.62		

Pressure Line		Scenario	1	2	3	4	5	6	7	8	9	10
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		
Segment	RAS = 0	Flow, mgd	2.69	3.45	4.19	6.73	4.14	6.00	6.80	8.30		
Number:	2	Downstream Water Level, ft	3136.66	3136.86	3137.07	3137.75	3137.06	3137.37	3137.67	3138.62		
Description:	30" SE from Reducer to Junction Box											
# of Total:	1 of 1	Velocity, ft/s	0.85	1.09	1.32	2.12	1.31	1.89	2.14	2.62		
		Velocity Head, ft	0.01	0.02	0.03	0.07	0.03	0.06	0.07	0.11		
Dimensions	1	Friction Loss, ft/1000ft	0.11	0.17	0.25	0.59	0.24	0.48	0.60	0.87		
Diameter, in	30	Friction Loss, ft	0.00	0.01	0.01	0.02	0.01	0.01	0.02	0.03		
Length, ft	30	Minor Loss, ft	0.01	0.02	0.03	0.07	0.03	0.06	0.07	0.11		
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		
Friction Coefficient, C	120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
Minor Loss Coefficient, K	1.00											
Downstream Invert Elev, ft	3132.50	Total Loss, ft	0.01	0.02	0.03	0.09	0.03	0.07	0.09	0.13		
Upstream Invert Elev, ft	3132.50											
		Upstream Water Level, ft	3136.67	3136.89	3137.11	3137.84	3137.09	3137.44	3137.76	3138.76		

Pressure Line		Scenario	1	2	3	4	5	6	7	8	9	10
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		
Segment	RAS = 0	Flow, mgd	2.69	3.45	4.19	6.73	4.14	6.00	6.80	8.30		
Number:	3	Downstream Water Level, ft	3136.67	3136.89	3137.11	3137.84	3137.09	3137.44	3137.76	3138.76		
Description:	42" SE from Clarifier No. 1 to Reducer											
# of Total:	1 of 1	Velocity, ft/s	0.43	0.55	0.67	1.08	0.67	0.96	1.09	1.33		
		Velocity Head, ft	0.00	0.00	0.01	0.02	0.01	0.01	0.02	0.03		
Dimensions		Friction Loss, ft/1000ft	0.02	0.03	0.05	0.12	0.05	0.09	0.12	0.17		
Diameter, in	42	Friction Loss, ft	0.00	0.00	0.00	0.01	0.00	0.01	0.01	0.02		
Length, ft	100	Minor Loss, ft	0.00	0.00	0.01	0.02	0.01	0.01	0.02	0.03		
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		
Friction Coefficient, C	120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
Minor Loss Coefficient, K	0.91											
Downstream Invert Elev, ft	3132.50	Total Loss, ft	0.00	0.01	0.01	0.03	0.01	0.02	0.03	0.04		
Upstream Invert Elev, ft	3132.50											
		Upstream Water Level, ft	3136.68	3136.90	3137.12	3137.86	3137.10	3137.46	3137.79	3138.80		

HYDRAULIC PROFILE CALCULATOR

Project: 2018 Missoula Wastewater Facility Plan
Subject: WWTP Hydraulic Profile - Side 1 Primary Lift Pumps to UV System

By: Rika Lashley Date: 4/3/2018
Chkd: WWW, ECS Date: 1/15/2019
Project No.: 1657.039

Open Channel		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25		
Segment	RAS = 0	Flow, mgd	0.67	0.86	1.05	1.68	1.04	1.50	1.70	2.08		
Number:	4	Downstream Water Level, ft	3136.68	3136.90	3137.12	3137.86	3137.10	3137.46	3137.79	3138.80		
Description:	Clarifier No. 1 Launder (half)											
# of Total:	1 of 1	Channel Slope, %	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
		Critical Depth, ft	0.20	0.24	0.27	0.37	0.27	0.35	0.38	0.43		
Dimensions		Normal Depth, ft	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a		
Width, ft	2	Downstream Depth, ft	4.18	4.40	4.62	5.36	4.60	4.96	5.29	6.30		
Length, ft	141	Downstream Velociy, ft/s	0.12	0.15	0.18	0.24	0.17	0.23	0.25	0.25		
Friction Coefficient, n	0.013	Upstream Depth, ft	4.18	4.40	4.62	5.36	4.60	4.96	5.29	6.30		
Downstream Invert Elev, ft	3132.50	Upstream Velocity, ft/s	0.12	0.15	0.18	0.24	0.17	0.23	0.25	0.25		
Upstream Invert Elev, ft	3132.50											
		Total Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
		Upstream Water Level, ft	3136.68	3136.90	3137.12	3137.86	3137.10	3137.46	3137.79	3138.80		

V-Notch Weir		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25		
Segment	RAS = 0	Flow, mgd	0.67	0.86	1.05	1.68	1.04	1.50	1.70	2.08		
Number:	5	Downstream Water Level, ft	3136.68	3136.90	3137.12	3137.86	3137.10	3137.46	3137.79	3138.80		
Description:	Clarifier No. 1 Weir (half)											
# of Total:	1 of 1	Flow Condition	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Normal		
		Free Fall, ft	2.19	1.97	1.75	1.01	1.77	1.41	1.08	0.07		
Dimensions		Head Over Weir, ft	0.07	0.08	0.09	0.10	0.09	0.10	0.10	0.11		
Number of Weirs	283											
V-Notch Angle, deg	90	Total Loss, ft	2.26	2.05	1.84	1.11	1.85	1.51	1.18	0.18		
Weir Crest Elevation ft	3138.87											
		Upstream Water Level, ft	3138.94	3138.95	3138.96	3138.97	3138.96	3138.97	3138.97	3138.98		

Pressure Line		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50		
Segment	RAS = 1	Flow, mgd	2.56	3.28	3.94	5.22	3.92	4.85	5.25	6.00		
Number:	6	Downstream Water Level, ft	3138.94	3138.95	3138.96	3138.97	3138.96	3138.97	3138.97	3138.98		
Description:	42" ML from Secondary Clarifier Splitterbox to Clarifier No. 1 Centerfeed											
# of Total:	1 of 1	Velocity, ft/s	0.41	0.53	0.63	0.84	0.63	0.78	0.84	0.96		
		Velocity Head, ft	0.00	0.00	0.01	0.01	0.01	0.01	0.01	0.01		
Dimensions		Friction Loss, ft/1000ft	0.02	0.03	0.04	0.07	0.04	0.06	0.07	0.09		
Diameter, in	42	Friction Loss, ft	0.00	0.00	0.01	0.01	0.01	0.01	0.01	0.01		
Length, ft	130	Minor Loss, ft	0.01	0.01	0.01	0.03	0.01	0.02	0.03	0.03		
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		
Friction Coefficient, C	120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
Minor Loss Coefficient, K	2.30											
Downstream Invert Elev, ft	3122.00	Total Loss, ft	0.01	0.01	0.02	0.03	0.02	0.03	0.03	0.05		
Upstream Invert Elev, ft	3132.50											
		Upstream Water Level, ft	3138.95	3138.96	3138.98	3139.01	3138.97	3139.00	3139.01	3139.03		

Rectangular Weir		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		
Segment	RAS = 1	Flow, mgd	5.11	6.55	7.89	10.43	7.84	9.70	10.50	12.00		
Number:	7	Downstream Water Level, ft	3138.95	3138.96	3138.98	3139.01	3138.97	3139.00	3139.01	3139.03		
Description:	Clarifier Splitterbox Weir											
# of Total:	1 of 1	Flow Condition	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Normal		
Difficult to model		Free Fall, ft	1.55	1.54	1.52	1.49	1.53	1.50	1.49	1.47		
Dimensions		Head Over Weir, ft	0.29	0.34	0.39	0.46	0.38	0.44	0.47	0.51		
Weir Length, ft	16											
Weir Type	Contracted	Total Loss, ft	1.84	1.88	1.91	1.96	1.91	1.94	1.96	1.98		
Weir Crest Elevation ft	3140.50											
		Upstream Water Level, ft	3140.79	3140.84	3140.89	3140.96	3140.88	3140.94	3140.97	3141.01		

Additional Headloss		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		
Segment	RAS = 1	Flow, mgd	5.11	6.55	7.89	10.43	7.84	9.70	10.50	12.00		
Number:	8	Downstream Water Level, ft	3140.79	3140.84	3140.89	3140.96	3140.88	3140.94	3140.97	3141.01		
Description:	Additional Splitterbox Headloss (based on 2002 Plans)											
# of Total:	1 of 1	Headloss	0.01	0.01	0.02	0.03	0.01	0.01	0.02	0.03		
Dimensions												
Underflow or orifice into splitter box												
no drawing available		Total Loss, ft	0.01	0.01	0.02	0.03	0.01	0.01	0.02	0.03		
		Upstream Water Level, ft	3140.80	3140.85	3140.91	3140.99	3140.89	3140.95	3140.99	3141.04		

HYDRAULIC PROFILE CALCULATOR

Project: 2018 Missoula Wastewater Facility Plan	By: Rika Lashley	Date: 4/3/2018
Subject: WWTP Hydraulic Profile - Side 1 Primary Lift Pumps to UV System	Chkd: WWW, ECS	Date: 1/15/2019
	Project No.: 1657.039	

Additional Headloss			Scenario	1	2	3	4	5	6	7	8	9	10
			Flow Factor	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50		
Segment	RAS = 1		Flow, mgd	2.56	3.28	3.94	5.22	3.92	4.85	5.25	6.00		
Number:	9		Downstream Water Level, ft	3140.80	3140.85	3140.91	3140.99	3140.89	3140.95	3140.99	3141.04		
Description:	Bioreactor (based on 2002 Plans)												
# of Total:	1 of 1		Headloss, ft	0.20	0.20	0.20	0.32	0.20	0.22	0.30	0.36		
Dimensions													
			Total Loss, ft	0.20	0.20	0.20	0.32	0.20	0.22	0.30	0.36		
			Upstream Water Level, ft	3141.00	3141.05	3141.11	3141.31	3141.09	3141.17	3141.29	3141.40		

Rectangular Weir			Scenario	1	2	3	4	5	6	7	8	9	10
			Flow Factor	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50		
Segment	RAS = 0		Flow, mgd	1.34	1.72	2.09	3.37	2.07	3.00	3.40	4.15		
Number:	10		Downstream Water Level, ft	3141.00	3141.05	3141.11	3141.31	3141.09	3141.17	3141.29	3141.40		
Description:	Bioreactor Splitter Box												
# of Total:	1 of 1		Flow Condition	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Sbmrgd		
			Free Fall, ft	0.33	0.28	0.22	0.02	0.24	0.16	0.04	0.00		
Dimensions			Head Over Weir, ft	0.26	0.30	0.34	0.47	0.34	0.44	0.48	0.55		
Weir Length, ft		5											
Weir Type		Contracted	Total Loss, ft	0.59	0.58	0.57	0.49	0.58	0.60	0.52	0.48		
Weir Crest Elevation ft		3141.33											
Non-submerged weir elev., ft		3141.41	**Upstream Water Level, ft**	3141.59	3141.63	3141.67	3141.80	3141.67	3141.77	3141.81	3141.88		

Pressure Line			Scenario	1	2	3	4	5	6	7	8	9	10
			Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		
Segment	RAS = 0		Flow, mgd	2.69	3.45	4.19	6.73	4.14	6.00	6.80	8.30		
Number:	11		Downstream Water Level, ft	3141.59	3141.63	3141.67	3141.80	3141.67	3141.77	3141.81	3141.88		
Description:	36" PE from Reducer to Bioreactor												
# of Total:	1 of 1		Velocity, ft/s	0.59	0.75	0.92	1.47	0.91	1.31	1.49	1.82		
			Velocity Head, ft	0.01	0.01	0.01	0.03	0.01	0.03	0.03	0.05		
Dimensions			Friction Loss, ft/1000ft	0.04	0.07	0.10	0.24	0.10	0.20	0.25	0.36		
Diameter, in		36	Friction Loss, ft	0.00	0.01	0.01	0.02	0.01	0.01	0.02	0.03		
Length, ft		75	Minor Loss, ft	0.01	0.01	0.02	0.06	0.02	0.04	0.06	0.09		
Solids Concentration, %		0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		
Friction Coefficient, C		120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
Minor Loss Coefficient, K		1.68											
Downstream Invert Elev, ft		3130.50	Total Loss, ft	0.01	0.02	0.03	0.07	0.03	0.06	0.08	0.11		
Upstream Invert Elev, ft		3130.50											
			Upstream Water Level, ft	3141.60	3141.65	3141.70	3141.88	3141.70	3141.83	3141.88	3142.00		

Pressure Line			Scenario	1	2	3	4	5	6	7	8	9	10
			Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		
Segment	RAS = 0		Flow, mgd	2.69	3.45	4.19	6.73	4.14	6.00	6.80	8.30		
Number:	12		Downstream Water Level, ft	3141.60	3141.65	3141.70	3141.88	3141.70	3141.83	3141.88	3142.00		
Description:	24" PE Effluent Lift Pump Manifold to Reducer												
# of Total:	1 of 1		Velocity, ft/s	1.32	1.70	2.06	3.32	2.04	2.95	3.35	4.09		
			Velocity Head, ft	0.03	0.04	0.07	0.17	0.06	0.14	0.17	0.26		
Dimensions			Friction Loss, ft/1000ft	0.32	0.51	0.73	1.76	0.72	1.42	1.79	2.59		
Diameter, in		24	Friction Loss, ft	0.03	0.05	0.07	0.17	0.07	0.13	0.17	0.25		
Length, ft		95	Minor Loss, ft	0.02	0.03	0.04	0.10	0.04	0.08	0.11	0.16		
Solids Concentration, %		0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		
Friction Coefficient, C		120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
Minor Loss Coefficient, K		0.61											
Downstream Invert Elev, ft		3130.50	Total Loss, ft	0.05	0.08	0.11	0.27	0.11	0.22	0.28	0.40		
Upstream Invert Elev, ft		3130.50											
			Upstream Water Level, ft	3141.65	3141.73	3141.81	3142.15	3141.81	3142.05	3142.16	3142.40		

Pressure Line			Scenario	1	2	3	4	5	6	7	8	9	10
			Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		
Segment	RAS = 0		Flow, mgd	2.69	3.45	4.19	6.73	4.14	6.00	6.80	8.30		
Number:	13		Downstream Water Level, ft	3141.65	3141.73	3141.81	3142.15	3141.81	3142.05	3142.16	3142.40		
Description:	18" Primary Effluent Pump Discharge												
# of Total:	1 of 1		Velocity, ft/s	2.35	3.02	3.67	5.90	3.63	5.25	5.95	7.27		
			Velocity Head, ft	0.09	0.14	0.21	0.54	0.20	0.43	0.55	0.82		
Dimensions			Friction Loss, ft/1000ft	1.31	2.07	2.97	7.14	2.91	5.77	7.27	10.51		
Diameter, in		18	Friction Loss, ft	0.01	0.02	0.03	0.07	0.03	0.06	0.07	0.11		
Length, ft		10	Minor Loss, ft	0.12	0.20	0.30	0.77	0.29	0.61	0.78	1.16		
Solids Concentration, %		0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		
Friction Coefficient, C		120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
Minor Loss Coefficient, K		1.42											
Downstream Invert Elev, ft		3139.00	Total Loss, ft	0.14	0.22	0.33	0.84	0.32	0.67	0.85	1.27		
Upstream Invert Elev, ft		3139.00											
			Upstream Water Level, ft	3141.78	3141.95	3142.14	3142.99	3142.13	3142.71	3143.01	3143.67		

HYDRAULIC PROFILE CALCULATOR

Project: 2018 Missoula Wastewater Facility Plan
Subject: WWTP Hydraulic Profile - Side 2 Primary Lift Pumps to UV System

By: Rika Lashley Date: 4/3/2019
Chkd: WWW, ECS Date: 1/15/2019
Project No.: 1657.039

Scenarios

Scenario	Description		
1	2017 Average Flow	7.27 mgd	Flow Split:
2	2017 Maximum Month Flow	9.32 mgd	Side 1 bioreactors/clarifiers 37%
3	2017 Maximum Day Flow	11.32 mgd	Side 2 bioreactors/clarifiers 63%
4	2017 Peak Hour Flow	18.2 mgd	
5	2037 Average flow	11.2 mgd	Assumptions:
6	2037 Maximum Month Flow	14.3 mgd	No changes in plant configuration for 2037.
7	2037 Maximum Day Flow	17.4 mgd	Two clarifiers in use for 2017; three clarifiers in use for 2037.
8	2037 Peak Hour Flow	26.6 mgd	Elevations largely taken from 2002 WWTP Record Drawings.
9	Plant Design Average Flow	12.0 mgd	RAS: 0.95Q; capped at pump station firm capacity (9.6 mgd).
10	Plant Design Peak Hour Flow	19.2 mgd	

Initial Values	Scenario	1	2	3	4	5	6	7	8	9	10
	Plant Flow, mgd	4.58	5.87	7.13	9.90	7.06	8.30	10.60	18.30	6.50	10.90
	RAS Flow, mgd	4.35	5.58	6.78	9.41	6.70	7.89	9.60	9.60	6.18	9.60
	Starting Water Level, ft	3136.66	3136.86	3137.07	3137.73	3137.05	3137.36	3137.66	3138.60	3137.13	3137.83

Pressure Line		Scenario	1	2	3	4	5	6	7	8	9	10
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	4.58	5.87	7.13	9.90	7.06	8.30	10.60	18.30	6.50	10.90
Number:	1	Downstream Water Level, ft	3136.66	3136.86	3137.07	3137.73	3137.05	3137.36	3137.66	3138.60	3137.13	3137.83
Description:	42" SE from Reducing Wye to UV Basin											
# of Total:	1 of 1	Velocity, ft/s	0.74	0.94	1.15	1.59	1.13	1.33	1.70	2.94	1.05	1.75
		Velocity Head, ft	0.01	0.01	0.02	0.04	0.02	0.03	0.05	0.13	0.02	0.05
Dimensions		Friction Loss, ft/1000ft	0.06	0.09	0.13	0.24	0.13	0.17	0.27	0.73	0.11	0.28
Diameter, in	42	Friction Loss, ft	0.01	0.01	0.01	0.02	0.01	0.02	0.03	0.08	0.01	0.03
Length, ft	106	Minor Loss, ft	0.02	0.03	0.04	0.08	0.04	0.06	0.10	0.29	0.04	0.10
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Friction Coefficient, C	120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Minor Loss Coefficient, K	2.15											
Downstream Invert Elev, ft	3126.92	Total Loss, ft	0.02	0.04	0.06	0.11	0.06	0.08	0.13	0.37	0.05	0.13
Upstream Invert Elev, ft	3126.92											
		Upstream Water Level, ft	3136.68	3136.90	3137.12	3137.84	3137.11	3137.43	3137.78	3138.97	3137.18	3137.96

Pressure Line		Scenario	1	2	3	4	5	6	7	8	9	10
		Flow Factor	0.50	0.50	0.50	0.50	0.33	0.33	0.33	0.33	0.50	0.50
Segment	RAS = 0	Flow, mgd	2.29	2.94	3.57	4.95	2.33	2.74	3.50	6.04	3.25	5.45
Number:	2	Downstream Water Level, ft	3136.68	3136.90	3137.12	3137.84	3137.11	3137.43	3137.78	3138.97	3137.18	3137.96
Description:	30" SE from Reducing Wye to Reducing Wye											
# of Total:	1 of 1	Velocity, ft/s	0.72	0.93	1.12	1.56	0.73	0.86	1.10	1.90	1.02	1.72
		Velocity Head, ft	0.01	0.01	0.02	0.04	0.01	0.01	0.02	0.06	0.02	0.05
Dimensions		Friction Loss, ft/1000ft	0.08	0.13	0.18	0.34	0.08	0.11	0.18	0.48	0.15	0.40
Diameter, in	30	Friction Loss, ft	0.01	0.02	0.03	0.05	0.01	0.02	0.03	0.07	0.02	0.06
Length, ft	142	Minor Loss, ft	0.01	0.02	0.02	0.05	0.01	0.01	0.02	0.07	0.02	0.06
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Friction Coefficient, C	120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Minor Loss Coefficient, K	1.22											
Downstream Invert Elev, ft	3126.92	Total Loss, ft	0.02	0.03	0.05	0.09	0.02	0.03	0.05	0.14	0.04	0.11
Upstream Invert Elev, ft	3126.92											
		Upstream Water Level, ft	3136.70	3136.93	3137.17	3137.94	3137.13	3137.46	3137.83	3139.11	3137.22	3138.07

Pressure Line		Scenario	1	2	3	4	5	6	7	8	9	10
		Flow Factor	0.50	0.50	0.50	0.50	0.33	0.33	0.33	0.33	0.50	0.50
Segment	RAS = 0	Flow, mgd	2.29	2.94	3.57	4.95	2.33	2.74	3.50	6.04	3.25	5.45
Number:	3	Downstream Water Level, ft	3136.70	3136.93	3137.17	3137.94	3137.13	3137.46	3137.83	3139.11	3137.22	3138.07
Description:	20" SE from Clarifier 6 to Reducing Wye											
# of Total:	1 of 1	Velocity, ft/s	1.62	2.08	2.53	3.51	1.65	1.94	2.48	4.28	2.30	3.86
		Velocity Head, ft	0.04	0.07	0.10	0.19	0.04	0.06	0.10	0.28	0.08	0.23
Dimensions		Friction Loss, ft/1000ft	0.58	0.92	1.32	2.42	0.60	0.81	1.27	3.49	1.11	2.89
Diameter, in	20	Friction Loss, ft	0.00	0.00	0.01	0.01	0.00	0.00	0.01	0.02	0.01	0.01
Length, ft	5	Minor Loss, ft	0.03	0.05	0.07	0.13	0.03	0.04	0.07	0.20	0.06	0.16
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Friction Coefficient, C	120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Minor Loss Coefficient, K	0.70											
Downstream Invert Elev, ft	3126.92	Total Loss, ft	0.03	0.05	0.08	0.15	0.03	0.05	0.07	0.22	0.06	0.18
Upstream Invert Elev, ft	3126.92											
		Upstream Water Level, ft	3136.73	3136.99	3137.25	3138.08	3137.16	3137.51	3137.90	3139.32	3137.28	3138.25

HYDRAULIC PROFILE CALCULATOR

By: Rika Lashley Date: 4/3/2019
Chkd: WWW, ECS Date: 1/15/2019
Project No.: 1657.039

Project: 2018 Missoula Wastewater Facility Plan
Subject: WWTP Hydraulic Profile - Side 2 Primary Lift Pumps to UV System

Open Channel		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	0.25	0.25	0.25	0.25	0.17	0.17	0.17	0.17	0.25	0.25
Segment	RAS = 0	Flow, mgd	1.15	1.47	1.78	2.48	1.16	1.37	1.75	3.02	1.63	2.73
Number:	4	Downstream Water Level, ft	3136.73	3136.99	3137.25	3138.08	3137.16	3137.51	3137.90	3139.32	3137.28	3138.25
Description:	Clarifier No. 6 Launder (half)											
# of Total:	1 of 1	Channel Slope, %	0.28	0.28	0.28	0.28	0.28	0.28	0.28	0.28	0.28	0.28
		Critical Depth, ft	0.29	0.34	0.39	0.48	0.29	0.33	0.38	0.55	0.37	0.52
Dimensions		Normal Depth, ft	0.36	0.42	0.48	0.60	0.36	0.40	0.47	0.70	0.45	0.65
Width, ft	2	Downstream Depth, ft	0.29	0.43	0.69	1.52	0.60	0.95	1.34	2.76	0.72	1.69
Length, ft	157	Downstream Velociy, ft/s	3.06	2.67	2.00	1.26	1.49	1.12	1.01	0.85	1.74	1.25
Friction Coefficient, n	0.013	Upstream Depth, ft	0.36	0.42	0.49	1.12	0.36	0.56	0.93	2.34	0.47	1.28
Downstream Invert Elev, ft	3136.56	Upstream Velocity, ft/s	2.49	2.70	2.82	1.71	2.47	1.90	1.45	1.00	2.69	1.64
Upstream Invert Elev, ft	3137.00											
		Total Loss, ft	0.62	0.44	0.24	0.04	0.20	0.05	0.03	0.01	0.18	0.03
		Upstream Water Level, ft	3137.36	3137.42	3137.49	3138.12	3137.36	3137.56	3137.93	3139.34	3137.47	3138.28

V-Notch Weir		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	0.25	0.25	0.25	0.25	0.17	0.17	0.17	0.17	0.25	0.25
Segment	RAS = 0	Flow, mgd	1.15	1.47	1.78	2.48	1.16	1.37	1.75	3.02	1.63	2.73
Number:	5	Downstream Water Level, ft	3137.36	3137.42	3137.49	3138.12	3137.36	3137.56	3137.93	3139.34	3137.47	3138.28
Description:	Clarifier No. 6 Weir (half)											
# of Total:	1 of 1	Flow Condition	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Normal
		Free Fall, ft	3.18	3.12	3.05	2.42	3.18	2.98	2.61	1.20	3.07	2.26
Dimensions		Head Over Weir, ft	0.08	0.09	0.10	0.12	0.09	0.09	0.10	0.13	0.10	0.12
Number of Weirs	314											
V-Notch Angle, deg	90	Total Loss, ft	3.27	3.21	3.15	2.54	3.26	3.07	2.71	1.33	3.17	2.38
Weir Crest Elevation ft	3140.54											
		Upstream Water Level, ft	3140.62	3140.63	3140.64	3140.66	3140.63	3140.63	3140.64	3140.67	3140.64	3140.66

Pressure Line		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	0.50	0.50	0.50	0.50	0.33	0.33	0.33	0.33	0.50	0.50
Segment	RAS = 1	Flow, mgd	4.47	5.72	6.95	9.65	4.54	5.34	6.67	9.21	6.34	10.25
Number:	6	Downstream Water Level, ft	3140.62	3140.63	3140.64	3140.66	3140.63	3140.63	3140.64	3140.67	3140.64	3140.66
Description:	24" ML from Secondary Clarifier Splitterbox to Clarifier No. 6 Centerfeed											
# of Total:	1 of 1	Velocity, ft/s	2.20	2.82	3.42	4.75	2.24	2.63	3.28	4.53	3.12	5.05
		Velocity Head, ft	0.08	0.12	0.18	0.35	0.08	0.11	0.17	0.32	0.15	0.40
Dimensions		Friction Loss, ft/1000ft	0.82	1.30	1.87	3.42	0.85	1.15	1.73	3.14	1.57	3.83
Diameter, in	24	Friction Loss, ft	0.07	0.12	0.17	0.31	0.08	0.10	0.16	0.28	0.14	0.34
Length, ft	90	Minor Loss, ft	0.17	0.27	0.40	0.77	0.17	0.24	0.37	0.70	0.33	0.87
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Friction Coefficient, C	120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Minor Loss Coefficient, K	2.20											
Downstream Invert Elev, ft	3119.00	Total Loss, ft	0.24	0.39	0.57	1.08	0.25	0.34	0.52	0.98	0.47	1.21
Upstream Invert Elev, ft	3119.00											
		Upstream Water Level, ft	3140.86	3141.02	3141.21	3141.74	3140.87	3140.97	3141.16	3141.65	3141.11	3141.88

Rectangular Weir		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	0.50	0.50	0.50	0.50	0.33	0.33	0.33	0.33	0.50	0.50
Segment	RAS = 1	Flow, mgd	4.47	5.72	6.95	9.65	4.54	5.34	6.67	9.21	6.34	10.25
Number:	7	Downstream Water Level, ft	3140.86	3141.02	3141.21	3141.74	3140.87	3140.97	3141.16	3141.65	3141.11	3141.88
Description:	Clarifier Splitterbox Weir											
# of Total:	1 of 1	Flow Condition	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Normal
		Free Fall, ft	1.21	1.05	0.86	0.33	1.20	1.10	0.91	0.42	0.96	0.19
Dimensions		Head Over Weir, ft	0.42	0.49	0.56	0.70	0.42	0.47	0.55	0.68	0.53	0.73
Weir Length, ft	8											
Weir Type	Contracted	Total Loss, ft	1.62	1.54	1.42	1.04	1.62	1.57	1.45	1.10	1.49	0.92
Weir Crest Elevation ft	3142.07											
		Upstream Water Level, ft	3142.49	3142.56	3142.63	3142.77	3142.49	3142.54	3142.62	3142.75	3142.60	3142.80

HYDRAULIC PROFILE CALCULATOR

By: Rika Lashley Date: 4/3/2019
Chkd: WWW, ECS Date: 1/15/2019

Project: 2018 Missoula Wastewater Facility Plan
Subject: WWTP Hydraulic Profile - Side 2 Primary Lift Pumps to UV System
Project No.: 1657.039

Pressure Line		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 1	Flow, mgd	8.93	11.45	13.91	19.31	13.76	16.19	20.20	27.90	12.68	20.50
Number:	8	Downstream Water Level, ft	3142.49	3142.56	3142.63	3142.77	3142.49	3142.54	3142.62	3142.75	3142.60	3142.80
Description:	42" ML from Bioreactor Effluent Channel to Clarifier Splitter Box											
# of Total:	1 of 1	Velocity, ft/s	1.44	1.84	2.24	3.10	2.21	2.60	3.25	4.49	2.04	3.30
		Velocity Head, ft	0.03	0.05	0.08	0.15	0.08	0.11	0.16	0.31	0.06	0.17
Dimensions		Friction Loss, ft/1000ft	0.19	0.31	0.44	0.81	0.43	0.58	0.88	1.60	0.37	0.90
Diameter, in	42	Friction Loss, ft	0.04	0.06	0.09	0.16	0.08	0.11	0.17	0.31	0.07	0.18
Length, ft	196	Minor Loss, ft	0.07	0.11	0.17	0.32	0.16	0.23	0.36	0.68	0.14	0.37
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Friction Coefficient, C	120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Minor Loss Coefficient, K	2.17											
Downstream Invert Elev, ft	3125.13	Total Loss, ft	0.11	0.17	0.25	0.48	0.25	0.34	0.53	0.99	0.21	0.54
Upstream Invert Elev, ft	3125.13											
		Upstream Water Level, ft	3142.60	3142.74	3142.89	3143.25	3142.74	3142.88	3143.15	3143.74	3142.81	3143.34

Additional Headloss		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Segment	RAS = 1	Flow, mgd	4.47	5.72	6.95	9.65	6.88	8.09	10.10	13.95	6.34	10.25
Number:	9	Downstream Water Level, ft	3142.60	3142.74	3142.89	3143.25	3142.74	3142.88	3143.15	3143.74	3142.81	3143.34
Description:	Bioreactor (calculated separately)											
# of Total:	1 of 1	Headloss, ft	0.78	0.70	0.63	0.47	0.70	0.57	0.45	0.35	0.67	0.47
Dimensions												
		Total Loss, ft	0.78	0.70	0.63	0.47	0.70	0.57	0.45	0.35	0.67	0.47
		Upstream Water Level, ft	3143.37	3143.44	3143.52	3143.72	3143.45	3143.46	3143.60	3144.09	3143.48	3143.81

Rectangular Weir		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Segment	RAS = 0	Flow, mgd	2.29	2.94	3.57	4.95	3.53	4.15	5.30	9.15	3.25	5.45
Number:	10	Downstream Water Level, ft	3143.37	3143.44	3143.52	3143.72	3143.45	3143.46	3143.60	3144.09	3143.48	3143.81
Description:	Bioreactor Splitter Box Weir											
# of Total:	1 of 1	Flow Condition	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Sbmrgd	Normal	Normal
		Free Fall, ft	0.63	0.56	0.48	0.28	0.55	0.54	0.40	0.00	0.52	0.19
Dimensions		Head Over Weir, ft	0.32	0.38	0.44	0.54	0.43	0.48	0.57	0.83	0.41	0.58
Weir Length, ft	6											
Weir Type	Contracted	Total Loss, ft	0.95	0.94	0.92	0.82	0.99	1.03	0.97	0.74	0.93	0.77
Weir Crest Elevation ft	3144.00											
		Upstream Water Level, ft	3144.32	3144.38	3144.44	3144.54	3144.43	3144.48	3144.57	3144.83	3144.41	3144.58

Pressure Line		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	4.58	5.87	7.13	9.90	7.06	8.30	10.60	18.30	6.50	10.90
Number:	11	Downstream Water Level, ft	3144.32	3144.38	3144.44	3144.54	3144.43	3144.48	3144.57	3144.83	3144.41	3144.58
Description:	36" PE from Reducer to Bioreactor Splitter Box											
# of Total:	1 of 1	Velocity, ft/s	1.00	1.29	1.56	2.17	1.54	1.82	2.32	4.01	1.42	2.39
		Velocity Head, ft	0.02	0.03	0.04	0.07	0.04	0.05	0.08	0.25	0.03	0.09
Dimensions		Friction Loss, ft/1000ft	0.12	0.19	0.27	0.50	0.27	0.36	0.57	1.55	0.23	0.59
Diameter, in	36	Friction Loss, ft	0.05	0.08	0.12	0.22	0.12	0.16	0.25	0.68	0.10	0.26
Length, ft	439	Minor Loss, ft	0.07	0.11	0.16	0.31	0.16	0.22	0.36	1.07	0.14	0.38
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Friction Coefficient, C	120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Minor Loss Coefficient, K	4.31											
Downstream Invert Elev, ft	3121.00	Total Loss, ft	0.12	0.19	0.28	0.53	0.28	0.38	0.61	1.75	0.24	0.64
Upstream Invert Elev, ft	3130.50											
		Upstream Water Level, ft	3144.44	3144.58	3144.72	3145.08	3144.71	3144.86	3145.18	3146.58	3144.65	3145.22

HYDRAULIC PROFILE CALCULATOR

By: Rika Lashley Date: 4/3/2019
Chkd: WWW, ECS Date: 1/15/2019
Project: 2018 Missoula Wastewater Facility Plan
Subject: WWTP Hydraulic Profile - Side 2 Primary Lift Pumps to UV System
Project No.: 1657.039

Pressure Line		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	4.58	5.87	7.13	9.90	7.06	8.30	10.60	18.30	6.50	10.90
Number:	12	Downstream Water Level, ft	3144.44	3144.58	3144.72	3145.08	3144.71	3144.86	3145.18	3146.58	3144.65	3145.22
Description:	30" PE from Reducer to Reducer											
# of Total:	1 of 1	Velocity, ft/s	1.44	1.85	2.25	3.12	2.22	2.62	3.34	5.77	2.05	3.44
		Velocity Head, ft	0.03	0.05	0.08	0.15	0.08	0.11	0.17	0.52	0.07	0.18
Dimensions		Friction Loss, ft/1000ft	0.29	0.46	0.66	1.21	0.65	0.87	1.37	3.77	0.56	1.45
Diameter, in	30	Friction Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.00	0.01
Length, ft	4	Minor Loss, ft	0.02	0.04	0.05	0.10	0.05	0.07	0.12	0.35	0.04	0.12
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Friction Coefficient, C	120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Minor Loss Coefficient, K	0.67											
Downstream Invert Elev, ft	3130.50	Total Loss, ft	0.02	0.04	0.06	0.11	0.05	0.07	0.12	0.36	0.05	0.13
Upstream Invert Elev, ft	3130.50											
		Upstream Water Level, ft	3144.47	3144.61	3144.77	3145.18	3144.76	3144.94	3145.30	3146.95	3144.69	3145.35

Pressure Line		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	4.58	5.87	7.13	9.90	7.06	8.30	10.60	18.30	6.50	10.90
Number:	13	Downstream Water Level, ft	3144.47	3144.61	3144.77	3145.18	3144.76	3144.94	3145.30	3146.95	3144.69	3145.35
Description:	24" PE Lift Pump Manifold to Reducer											
# of Total:	1 of 1	Velocity, ft/s	2.26	2.89	3.51	4.88	3.47	4.09	5.22	9.01	3.20	5.37
		Velocity Head, ft	0.08	0.13	0.19	0.37	0.19	0.26	0.42	1.26	0.16	0.45
Dimensions		Friction Loss, ft/1000ft	0.86	1.36	1.96	3.59	1.92	2.59	4.07	11.18	1.65	4.29
Diameter, in	24	Friction Loss, ft	0.01	0.01	0.02	0.04	0.02	0.03	0.04	0.11	0.02	0.04
Length, ft	10	Minor Loss, ft	0.04	0.06	0.09	0.17	0.09	0.12	0.19	0.58	0.07	0.21
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Friction Coefficient, C	120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Minor Loss Coefficient, K	0.46											
Downstream Invert Elev, ft	3139.00	Total Loss, ft	0.04	0.07	0.11	0.21	0.11	0.15	0.24	0.69	0.09	0.25
Upstream Invert Elev, ft	3139.00											
		Upstream Water Level, ft	3144.51	3144.69	3144.88	3145.39	3144.87	3145.08	3145.53	3147.64	3144.78	3145.60

Pressure Line		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	4.58	5.87	7.13	9.90	7.06	8.30	10.60	18.30	6.50	10.90
Number:	14	Downstream Water Level, ft	3144.51	3144.69	3144.88	3145.39	3144.87	3145.08	3145.53	3147.64	3144.78	3145.60
Description:	18" Effluent Lift Pump Discharge											
# of Total:	1 of 1	Velocity, ft/s	4.01	5.14	6.24	8.67	6.18	7.27	9.28	16.02	5.69	9.54
		Velocity Head, ft	0.25	0.41	0.61	1.17	0.59	0.82	1.34	3.99	0.50	1.41
Dimensions		Friction Loss, ft/1000ft	3.50	5.54	7.94	14.56	7.78	10.51	16.52	45.37	6.69	17.40
Diameter, in	18	Friction Loss, ft	0.03	0.04	0.06	0.12	0.06	0.08	0.13	0.36	0.05	0.14
Length, ft	8	Minor Loss, ft	0.20	0.33	0.49	0.94	0.48	0.66	1.08	3.23	0.41	1.15
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Friction Coefficient, C	120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Minor Loss Coefficient, K	0.81											
Downstream Invert Elev, ft	0.00	Total Loss, ft	0.23	0.38	0.55	1.06	0.54	0.75	1.22	3.59	0.46	1.28
Upstream Invert Elev, ft	0.00											
		Upstream Water Level, ft	3144.74	3145.06	3145.44	3146.45	3145.41	3145.83	3146.75	3151.23	3145.24	3146.88

HYDRAULIC PROFILE CALCULATOR

By: Rika Lashley Date: 10/18/2018
Chkd: WWW, ECS Date: 1/15/2019
Project No.: 1657.039

Project: 2018 Missoula Wastewater Facility Plan
Subject: WWTP Hydraulic Profile - Headworks to Primary Lift Pump Wet Well

Scenarios

Scenario	Description		
1	2017 Average Flow	7.27	mgd
2	2017 Maximum Month Flow	9.32	mgd
3	2017 Maximum Day Flow	11.32	mgd
4	2017 Peak Hour Flow	18.2	mgd
5	2037 Average flow	11.2	mgd
6	2037 Maximum Month Flow	14.3	mgd
7	2037 Maximum Day Flow	17.4	mgd
8	2037 Peak Hour Flow	26.6	mgd
9	Plant Design Average Flow	12.0	mgd
10	Plant Design Peak Hour Flow	19.2	mgd

Assumptions:
- No changes in plant configuration for 2037.
- Three primary clarifiers in service for all scenarios.
- Elevations largely taken from 2002 WWTP Record Drawings and 2012 Headworks Record Drawings.
- Starting water level is max primary lift pump wet well level.
- No more than two influent lift pumps in service for any scenario.
- Two influent screens in service for all scenarious

Initial Values	Scenario	1	2	3	4	5	6	7	8	9	10
	Plant Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
	RAS Flow, mgd										
	Starting Water Level, ft	3134.00	3134.00	3134.00	3134.00	3134.00	3134.00	3134.00	3134.00	3134.00	3134.00
	Maximum wet well water level.										

Pressure Line		Scenario	1	2	3	4	5	6	7	8	9	10
		Flow Factor	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33
Segment	RAS = 0	Flow, mgd	2.40	3.08	3.74	6.01	3.70	4.72	5.74	8.78	3.96	6.34
Number:	1	Downstream Water Level, ft	3134.00	3134.00	3134.00	3134.00	3134.00	3134.00	3134.00	3134.00	3134.00	3134.00
Description:	24" PE from Clarifier 3 to Primary Effluent Lift Station Wet Well											
# of Total:	1 of 1	Velocity, ft/s	1.18	1.51	1.84	2.96	1.82	2.32	2.83	4.32	1.95	3.12
		Velocity Head, ft	0.02	0.04	0.05	0.14	0.05	0.08	0.12	0.29	0.06	0.15
Dimensions		Friction Loss, ft/1000ft	0.26	0.41	0.59	1.42	0.58	0.91	1.31	2.87	0.66	1.57
Diameter, in	24	Friction Loss, ft	0.01	0.02	0.03	0.06	0.03	0.04	0.06	0.13	0.03	0.07
Length, ft	44	Minor Loss, ft	0.05	0.08	0.12	0.31	0.12	0.19	0.29	0.67	0.14	0.35
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Friction Coefficient, C	120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Minor Loss Coefficient, K	2.30											
Downstream Invert Elev, ft	3126.92	Total Loss, ft	0.06	0.10	0.15	0.37	0.14	0.23	0.34	0.79	0.16	0.42
Upstream Invert Elev, ft	3126.92											
		Upstream Water Level, ft	3134.06	3134.10	3134.15	3134.37	3134.14	3134.23	3134.34	3134.79	3134.16	3134.42

Open Channel		Scenario	1	2	3	4	5	6	7	8	9	10
		Flow Factor	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17
Segment	RAS = 0	Flow, mgd	1.20	1.54	1.87	3.00	1.85	2.36	2.87	4.39	1.98	3.17
Number:	2	Downstream Water Level, ft	3134.06	3134.10	3134.15	3134.37	3134.14	3134.23	3134.34	3134.79	3134.16	3134.42
Description:	Clarifier 3 Launder (half)											
# of Total:	1 of 1	Channel Slope, %	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04
		Critical Depth, ft	0.36	0.43	0.49	0.67	0.48	0.57	0.65	0.86	0.51	0.69
Dimensions		Normal Depth, ft	1.01	1.23	1.43	2.13	1.42	1.74	2.05	2.96	1.50	2.23
Width, ft	1.5	Downstream Depth, ft	0.36	0.43	0.49	0.67	0.48	0.57	0.65	0.86	0.51	0.69
Length, ft	141	Downstream Velociy, ft/s	3.42	3.71	3.96	4.64	3.94	4.28	4.57	5.26	4.04	4.72
Friction Coefficient, n	0.013	Upstream Depth, ft	0.67	0.78	0.88	1.17	0.87	1.01	1.14	1.47	0.91	1.21
Downstream Invert Elev, ft	3136.44	Upstream Velocity, ft/s	1.84	2.04	2.20	2.65	2.19	2.41	2.61	3.07	2.25	2.71
Upstream Invert Elev, ft	3136.49											
Estimated channel floor elevation based on observed free fall during site visit		Total Loss, ft	3.10	3.17	3.22	3.28	3.22	3.27	3.28	3.17	3.23	3.28
		Upstream Water Level, ft	3137.16	3137.27	3137.37	3137.66	3137.36	3137.50	3137.63	3137.96	3137.40	3137.70

V-Notch Weir		Scenario	1	2	3	4	5	6	7	8	9	10
		Flow Factor	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17
Segment	RAS = 0	Flow, mgd	1.20	1.54	1.87	3.00	1.85	2.36	2.87	4.39	1.98	3.17
Number:	3	Downstream Water Level, ft	3137.16	3137.27	3137.37	3137.66	3137.36	3137.50	3137.63	3137.96	3137.40	3137.70
Description:	Clarifier 3 Weir (half)											
# of Total:	1 of 1	Flow Condition	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Normal
		Free Fall, ft	1.48	1.37	1.27	0.98	1.28	1.14	1.01	0.68	1.24	0.94
Dimensions		Head Over Weir, ft	0.09	0.10	0.11	0.13	0.11	0.12	0.13	0.15	0.11	0.13
Number of Weirs	283											
V-Notch Angle, deg	90	Total Loss, ft	1.57	1.47	1.38	1.11	1.39	1.26	1.14	0.83	1.35	1.08
Weir Crest Elevation ft	3138.64											
		Upstream Water Level, ft	3138.73	3138.74	3138.75	3138.77	3138.75	3138.76	3138.77	3138.79	3138.75	3138.77

HYDRAULIC PROFILE CALCULATOR

Project: 2018 Missoula Wastewater Facility Plan
Subject: WWTP Hydraulic Profile - Headworks to Primary Lift Pump Wet Well

By: Rika Lashley Date: 10/18/2018
Chkd: WWW, ECS Date: 1/15/2019
Project No.: 1657.039

Pressure Line		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33
Segment	RAS = 0	Flow, mgd	2.40	3.08	3.74	6.01	3.70	4.72	5.74	8.78	3.96	6.34
Number:	4	Downstream Water Level, ft	3138.73	3138.74	3138.75	3138.77	3138.75	3138.76	3138.77	3138.79	3138.75	3138.77
Description:	24" Pl from Primary Clarifier Splitter Box to Clarifier 3											
# of Total:	1 of 1	Velocity, ft/s	1.18	1.51	1.84	2.96	1.82	2.32	2.83	4.32	1.95	3.12
		Velocity Head, ft	0.02	0.04	0.05	0.14	0.05	0.08	0.12	0.29	0.06	0.15
Dimensions		Friction Loss, ft/1000ft	0.26	0.41	0.59	1.42	0.58	0.91	1.31	2.87	0.66	1.57
Diameter, in	24	Friction Loss, ft	0.03	0.05	0.07	0.17	0.07	0.11	0.15	0.34	0.08	0.18
Length, ft	117	Minor Loss, ft	0.07	0.11	0.16	0.42	0.16	0.26	0.38	0.90	0.18	0.47
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Friction Coefficient, C	120	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Minor Loss Coefficient, K	3.10											
Downstream Invert Elev, ft	3130.17	Total Loss, ft	0.10	0.16	0.23	0.59	0.23	0.37	0.54	1.24	0.26	0.65
Upstream Invert Elev, ft	3130.17											
		Upstream Water Level, ft	3138.83	3138.90	3138.98	3139.36	3138.98	3139.13	3139.31	3140.03	3139.01	3139.43

Rectangular Weir		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33	0.33
Segment	RAS = 0	Flow, mgd	2.40	3.08	3.74	6.01	3.70	4.72	5.74	8.78	3.96	6.34
Number:	5	Downstream Water Level, ft	3138.83	3138.90	3138.98	3139.36	3138.98	3139.13	3139.31	3140.03	3139.01	3139.43
Description:	Primary Clarifier Splitter Box Weir											
# of Total:	1 of 1	Flow Condition	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Sbmrgd	Normal	Normal
		Free Fall, ft	0.62	0.55	0.47	0.09	0.47	0.32	0.14	0.00	0.44	0.02
Dimensions		Head Over Weir, ft	0.53	0.63	0.72	0.98	0.71	0.84	0.95	1.37	0.74	1.02
Weir Length, ft	3											
Weir Type	Contracted	Total Loss, ft	1.15	1.18	1.19	1.07	1.19	1.16	1.10	0.79	1.18	1.04
Weir Crest Elevation ft	3139.45											
Future weir level, ft	3140.00	**Upstream Water Level, ft**	3139.98	3140.08	3140.17	3140.43	3140.16	3140.29	3140.40	3140.82	3140.19	3140.47

Pressure Line		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
Number:	6	Downstream Water Level, ft	3139.98	3140.08	3140.17	3140.43	3140.16	3140.29	3140.40	3140.82	3140.19	3140.47
Description:	42" Pl from Grit System to Primary Clarifier Splitter Box											
# of Total:	1 of 1	Velocity, ft/s	1.17	1.50	1.82	2.93	1.80	2.30	2.80	4.28	1.93	3.09
		Velocity Head, ft	0.02	0.03	0.05	0.13	0.05	0.08	0.12	0.28	0.06	0.15
Dimensions		Friction Loss, ft/1000ft	0.11	0.18	0.26	0.63	0.25	0.40	0.58	1.26	0.29	0.69
Diameter, in	42	Friction Loss, ft	0.01	0.02	0.03	0.07	0.03	0.05	0.07	0.15	0.03	0.08
Length, ft	117	Minor Loss, ft	0.04	0.07	0.11	0.28	0.10	0.17	0.25	0.59	0.12	0.31
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Friction Coefficient, C	130	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Minor Loss Coefficient, K	2.08											
Downstream Invert Elev, ft	3130.25	Total Loss, ft	0.06	0.09	0.14	0.35	0.13	0.22	0.32	0.74	0.15	0.39
Upstream Invert Elev, ft	3130.53											
		Upstream Water Level, ft	3140.04	3140.17	3140.30	3140.78	3140.30	3140.50	3140.72	3141.56	3140.35	3140.86

Open Channel		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Segment	RAS = 0	Flow, mgd	3.64	4.66	5.66	9.10	5.60	7.15	8.70	13.30	6.00	9.60
Number:	7	Downstream Water Level, ft	3140.04	3140.17	3140.30	3140.78	3140.30	3140.50	3140.72	3141.56	3140.35	3140.86
Description:	Grit System Effluent Channel											
# of Total:	1 of 1	Channel Slope, %	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		Critical Depth, ft	0.48	0.56	0.64	0.88	0.64	0.75	0.86	1.13	0.67	0.91
Dimensions		Normal Depth, ft	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Width, ft	3	Downstream Depth, ft	0.48	0.56	0.64	0.88	0.64	0.75	0.86	1.13	0.67	0.91
Length, ft	12	Downstream Velociy, ft/s	3.92	4.26	4.55	5.33	4.53	4.91	5.25	6.04	4.64	5.42
Friction Coefficient, n	0.013	Upstream Depth, ft	0.60	0.70	0.79	1.05	0.78	0.91	1.02	1.34	0.81	1.09
Downstream Invert Elev, ft	3140.83	Upstream Velocity, ft/s	3.13	3.45	3.72	4.45	3.70	4.06	4.38	5.13	3.80	4.54
Upstream Invert Elev, ft	3140.83											
		Total Loss, ft	1.39	1.35	1.31	1.10	1.31	1.23	1.13	0.61	1.30	1.06
		Upstream Water Level, ft	3141.43	3141.53	3141.62	3141.88	3141.61	3141.74	3141.85	3142.17	3141.64	3141.92

Additional Headloss		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Segment	RAS = 0	Flow, mgd	3.64	4.66	5.66	9.10	5.60	7.15	8.70	13.30	6.00	9.60
Number:	8	Downstream Water Level, ft	3141.43	3141.53	3141.62	3141.88	3141.61	3141.74	3141.85	3142.17	3141.64	3141.92
Description:	Grit System Headloss per Manufacturer											
# of Total:	1 of 1											
Dimensions												
Given as 0.25" to 1"												
		Total Loss, ft	0.02	0.04	0.06	0.08	0.06	0.07	0.08	0.10	0.06	0.08
		Upstream Water Level, ft	3141.45	3141.57	3141.68	3141.97	3141.67	3141.81	3141.93	3142.27	3141.70	3142.00

HYDRAULIC PROFILE CALCULATOR

By: Rika Lashley Date: 10/18/2018

Project: 2018 Missoula Wastewater Facility Plan

Chkd: WWW, ECS Date: 1/15/2019

Subject: WWTP Hydraulic Profile - Headworks to Primary Lift Pump Wet Well

Project No.: 1657.039

Open Channel		Scenario	1	2	3	4	5	6	7	8	9	10
		Flow Factor	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Segment	RAS = 0	Flow, mgd	3.64	4.66	5.66	9.10	5.60	7.15	8.70	13.30	6.00	9.60
Number:	9	Downstream Water Level, ft	3141.45	3141.57	3141.68	3141.97	3141.67	3141.81	3141.93	3142.27	3141.70	3142.00
Description:	Grit System Influent Channel											
# of Total:	1 of 1	Channel Slope, %	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		Critical Depth, ft	0.48	0.56	0.64	0.88	0.64	0.75	0.86	1.13	0.67	0.91
Dimensions		Normal Depth, ft	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Width, ft	3	Downstream Depth, ft	0.62	0.74	0.85	1.14	0.84	0.98	1.10	1.44	0.87	1.17
Length, ft	12	Downstream Velociy, ft/s	3.02	3.26	3.45	4.13	3.44	3.77	4.06	4.77	3.54	4.23
Friction Coefficient, n	0.013	Upstream Depth, ft	0.66	0.78	0.88	1.18	0.88	1.02	1.15	1.49	0.91	1.22
Downstream Invert Elev, ft	3140.83	Upstream Velocity, ft/s	2.85	3.10	3.30	3.96	3.29	3.62	3.90	4.59	3.38	4.06
Upstream Invert Elev, ft	3140.83											
Downstream is actually 3138.33 but the spreadsheet can't compute steep slopes.		Total Loss, ft	0.04	0.04	0.04	0.05	0.04	0.04	0.05	0.06	0.04	0.05
		Upstream Water Level, ft	3141.49	3141.61	3141.71	3142.01	3141.71	3141.85	3141.98	3142.32	3141.74	3142.05

Pressure Line		Scenario	1	2	3	4	5	6	7	8	9	10
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
Number:	10	Downstream Water Level, ft	3141.49	3141.61	3141.71	3142.01	3141.71	3141.85	3141.98	3142.32	3141.74	3142.05
Description:	36" PLI from Influent Lift Pump Manifold to Grit System											
# of Total:	1 of 1	Velocity, ft/s	1.59	2.04	2.48	3.98	2.45	3.13	3.81	5.82	2.63	4.20
		Velocity Head, ft	0.04	0.06	0.10	0.25	0.09	0.15	0.23	0.53	0.11	0.27
Dimensions		Friction Loss, ft/1000ft	0.24	0.38	0.55	1.32	0.54	0.85	1.22	2.67	0.61	1.46
Diameter, in	36	Friction Loss, ft	0.03	0.05	0.07	0.18	0.07	0.12	0.17	0.36	0.08	0.20
Length, ft	136	Minor Loss, ft	0.09	0.15	0.22	0.57	0.22	0.35	0.52	1.22	0.25	0.64
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Friction Coefficient, C	130	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Minor Loss Coefficient, K	2.32											
Downstream Invert Elev, ft	3121.05	Total Loss, ft	0.12	0.20	0.30	0.75	0.29	0.47	0.69	1.58	0.33	0.83
Upstream Invert Elev, ft	3121.05											
		Upstream Water Level, ft	3141.61	3141.81	3142.01	3142.77	3142.00	3142.32	3142.67	3143.91	3142.08	3142.88

Pressure Line		Scenario	1	2	3	4	5	6	7	8	9	10
		Flow Factor	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Segment	RAS = 0	Flow, mgd	3.64	4.66	5.66	9.10	5.60	7.15	8.70	13.30	6.00	9.60
Number:	11	Downstream Water Level, ft	3141.61	3141.81	3142.01	3142.77	3142.00	3142.32	3142.67	3143.91	3142.08	3142.88
Description:	16" Influent Pump 3 Discharge											
# of Total:	1 of 1	Velocity, ft/s	4.03	5.16	6.27	10.08	6.20	7.92	9.64	14.74	6.65	10.64
		Velocity Head, ft	0.25	0.41	0.61	1.58	0.60	0.97	1.44	3.37	0.69	1.76
Dimensions		Friction Loss, ft/1000ft	3.49	5.53	7.92	19.07	7.77	12.20	17.55	38.48	8.82	21.05
Diameter, in	16	Friction Loss, ft	0.20	0.32	0.46	1.11	0.45	0.71	1.02	2.23	0.51	1.22
Length, ft	58	Minor Loss, ft	0.37	0.60	0.89	2.29	0.87	1.41	2.09	4.89	1.00	2.55
Solids Concentration, %	0	Solids Multiplier	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Friction Coefficient, C	130	Solids Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Minor Loss Coefficient, K	1.45											
Downstream Invert Elev, ft	3107.00	Total Loss, ft	0.57	0.92	1.34	3.39	1.32	2.12	3.11	7.12	1.51	3.77
Upstream Invert Elev, ft	3124.00											
		Upstream Water Level, ft	3142.18	3142.73	3143.35	3146.16	3143.32	3144.44	3145.78	3151.03	3143.58	3146.65

Additional Headloss		Scenario	1	2	3	4	5	6	7	8	9	10
		Flow Factor	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Segment	RAS = 0	Flow, mgd	3.64	4.66	5.66	9.10	5.60	7.15	8.70	13.30	6.00	9.60
Number:	12	Downstream Water Level, ft	3142.18	3142.73	3143.35	3146.16	3143.32	3144.44	3145.78	3151.03	3143.58	3146.65
Description:	Influent Lift Pumps											
# of Total:	1 of 1	Pump Lift for Max Level	-27.18	-27.73	-28.35	-31.16	-28.32	-29.44	-30.78	-36.03	-28.58	-31.65
		Pump Lift for Min Level	-33.18	-33.73	-34.35	-37.16	-34.32	-35.44	-36.78	-42.03	-34.58	-37.65
Dimensions												
Max Wet Well Level	3115.00											
Min Wet Well Level	3109.00	Total Loss, ft	-27.18	-27.73	-28.35	-31.16	-28.32	-29.44	-30.78	-36.03	-28.58	-31.65
		Upstream Water Level, ft	3115.00	3115.00	3115.00	3115.00	3115.00	3115.00	3115.00	3115.00	3115.00	3115.00

Rectangular Orifice (Submerged)		Scenario	1	2	3	4	5	6	7	8	9	10
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
Number:	13	Downstream Water Level, ft	3115.00	3115.00	3115.00	3115.00	3115.00	3115.00	3115.00	3115.00	3115.00	3115.00
Description:	Baffle Wall in Pump Forebay - Underflow											
# of Total:	1 of 1	Velocity, ft/s	0.22	0.29	0.35	0.56	0.34	0.44	0.53	0.81	0.37	0.59
		Velocity Head, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.01
Dimensions												
Width, ft	2.33											
Length, ft	21.67	Total Loss, ft	0.00	0.00	0.00	0.01	0.00	0.00	0.01	0.02	0.00	0.01
Coefficient of Dischage	0.8											
Top of Baffle Wall	3116.50	**Upstream Water Level, ft**	3115.00	3115.00	3115.00	3115.01	3115.00	3115.00	3115.01	3115.02	3115.00	3115.01

HYDRAULIC PROFILE CALCULATOR

By: Rika Lashley Date: 10/18/2018
Chkd: WWW, ECS Date: 1/15/2019
Project No.: 1657.039

Project: 2018 Missoula Wastewater Facility Plan
Subject: WWTP Hydraulic Profile - Headworks to Primary Lift Pump Wet Well

Open Channel		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Segment	RAS = 0	Flow, mgd	3.64	4.66	5.66	9.10	5.60	7.15	8.70	13.30	6.00	9.60
Number:	14	Downstream Water Level, ft	3115.00	3115.00	3115.00	3115.01	3115.00	3115.00	3115.01	3115.02	3115.00	3115.01
Description:	Channel Downstream of Screen 1											
# of Total:	1 of 1	Channel Slope, %	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		Critical Depth, ft	0.39	0.47	0.53	0.73	0.53	0.62	0.71	0.94	0.55	0.75
Dimensions		Normal Depth, ft	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Width, ft	4	Downstream Depth, ft	0.39	0.47	0.53	0.73	0.53	0.62	0.71	0.94	0.55	0.75
Length, ft	20	Downstream Velociy, ft/s	3.56	3.87	4.13	4.84	4.12	4.47	4.77	5.49	4.21	4.93
Friction Coefficient, n	0.013	Upstream Depth, ft	0.53	0.61	0.68	0.91	0.68	0.79	0.89	1.15	0.71	0.94
Downstream Invert Elev, ft	3115.00	Upstream Velocity, ft/s	2.67	2.95	3.20	3.86	3.18	3.51	3.79	4.47	3.27	3.94
Upstream Invert Elev, ft	3115.00											
		Total Loss, ft	0.53	0.61	0.68	0.90	0.68	0.78	0.88	1.13	0.71	0.93
		Upstream Water Level, ft	3115.53	3115.61	3115.68	3115.91	3115.68	3115.79	3115.89	3116.15	3115.71	3115.94

Additional Headloss		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Segment	RAS = 0	Flow, mgd	3.64	4.66	5.66	9.10	5.60	7.15	8.70	13.30	6.00	9.60
Number:	15	Downstream Water Level, ft	3115.53	3115.61	3115.68	3115.91	3115.68	3115.79	3115.89	3116.15	3115.71	3115.94
Description:	Screens - estimated headloss based on typical bar screens											
# of Total:	1 of 1	Estimated Headloss, ft	0.50	0.54	0.61	0.74	0.58	0.68	0.71	0.87	0.65	0.77
Dimensions												
Max Spec'd Headloss, ft	0.83											
at 12.4 mgd per screen		Total Loss, ft	0.50	0.54	0.61	0.74	0.58	0.68	0.71	0.87	0.65	0.77
		Upstream Water Level, ft	3116.03	3116.15	3116.29	3116.65	3116.26	3116.47	3116.60	3117.02	3116.36	3116.71

Open Channel		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Segment	RAS = 0	Flow, mgd	3.64	4.66	5.66	9.10	5.60	7.15	8.70	13.30	6.00	9.60
Number:	16	Downstream Water Level, ft	3116.03	3116.15	3116.29	3116.65	3116.26	3116.47	3116.60	3117.02	3116.36	3116.71
Description:	Channel Upstream of Screen 1											
# of Total:	1 of 1	Channel Slope, %	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		Critical Depth, ft	0.39	0.47	0.53	0.73	0.53	0.62	0.71	0.94	0.55	0.75
Dimensions		Normal Depth, ft	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Width, ft	4	Downstream Depth, ft	1.03	1.15	1.29	1.65	1.26	1.47	1.60	2.02	1.36	1.71
Length, ft	11	Downstream Velociy, ft/s	1.37	1.57	1.69	2.13	1.72	1.88	2.11	2.55	1.71	2.17
Friction Coefficient, n	0.013	Upstream Depth, ft	1.03	1.15	1.30	1.66	1.26	1.47	1.60	2.03	1.36	1.72
Downstream Invert Elev, ft	3115.00	Upstream Velocity, ft/s	1.37	1.56	1.69	2.12	1.71	1.88	2.10	2.54	1.70	2.16
Upstream Invert Elev, ft	3115.00											
		Total Loss, ft	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00
		Upstream Water Level, ft	3116.03	3116.15	3116.30	3116.66	3116.26	3116.47	3116.60	3117.03	3116.36	3116.72

Parshall Flume		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
Number:	17	Downstream Water Level, ft	3116.03	3116.15	3116.30	3116.66	3116.26	3116.47	3116.60	3117.03	3116.36	3116.72
Description:	Parshall Flume											
# of Total:	1 of 1	Flow Condition	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Normal	Normal
		DS Level Above Crest, ft	-1.17	-1.05	-0.90	-0.54	-0.94	-0.73	-0.60	-0.17	-0.84	-0.48
Dimensions		Head Over Crest, ft	0.80	0.94	1.06	1.43	1.05	1.23	1.39	1.82	1.10	1.48
Flume Width, in	48	DS Level / US Level	-1.46	-1.12	-0.85	-0.38	-0.89	-0.59	-0.43	-0.10	-0.76	-0.33
Crest Elevation, ft	3117.20											
		Total Loss, ft	1.97	1.98	1.96	1.97	1.99	1.96	1.99	1.99	1.94	1.96
		Upstream Water Level, ft	3118.00	3118.14	3118.26	3118.63	3118.25	3118.43	3118.59	3119.02	3118.30	3118.68

Open Channel		**Scenario**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
		Flow Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Segment	RAS = 0	Flow, mgd	7.27	9.32	11.32	18.20	11.20	14.30	17.40	26.60	12.00	19.20
Number:	18	Downstream Water Level, ft	3118.00	3118.14	3118.26	3118.63	3118.25	3118.43	3118.59	3119.02	3118.30	3118.68
Description:	Plant Influent Channel											
# of Total:	1 of 1	Channel Slope, %	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		Critical Depth, ft	0.46	0.54	0.62	0.85	0.61	0.72	0.82	1.09	0.64	0.88
Dimensions		Normal Depth, ft	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
Width, ft	6.35	Downstream Depth, ft	0.75	0.89	1.01	1.38	1.00	1.18	1.34	1.77	1.05	1.43
Length, ft	27	Downstream Velociy, ft/s	2.36	2.56	2.73	3.21	2.72	2.96	3.16	3.66	2.79	3.27
Friction Coefficient, n	0.013	Upstream Depth, ft	0.78	0.91	1.04	1.41	1.03	1.21	1.37	1.80	1.08	1.46
Downstream Invert Elev, ft	3117.25	Upstream Velocity, ft/s	2.28	2.49	2.66	3.15	2.65	2.89	3.10	3.60	2.72	3.21
Upstream Invert Elev, ft	3117.25											
		Total Loss, ft	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03
		Upstream Water Level, ft	3118.03	3118.16	3118.29	3118.66	3118.28	3118.46	3118.62	3119.05	3118.33	3118.71

WASTEWATER FACILITY PLAN

CHAPTER 8 - FUTURE TREATMENT PLANT ANALYSIS AND ALTERNATIVES

Morrison Maierle

engineers ▪ surveyors ▪ planners ▪ scientists

CHAPTER 8 FUTURE TREATMENT PLANT ANALYSIS AND ALTERNATIVES

8.1. INTRODUCTION

This chapter looks to the future of the Missoula wastewater treatment plant (WWTP) in light of increasing flows and loads and changing permit limits for nutrients. An evaluation matrix is used to develop alternatives that would satisfy a variety of different flow and effluent limit scenarios. The alternatives are described and evaluated for their ability to produce the needed effluent quality, cost, and feasibility of construction.

8.2. EVALUATION SCENARIOS AND ALTERNATIVE DEVELOPMENT

While some improvement alternatives are independent of treatment performance, for most alternatives, two planning aspects must be considered when evaluating options for future Missoula WWTP modification and/or expansion:

1. Influent flows and loads
2. Required effluent limits

Influent flow is considered when evaluating the hydraulic capacity of all WWTP components. For planning purposes, average, maximum month, maximum day, and peak hour flows are sufficient to cover the range of possibilities the equipment must be able to handle and for how long. For the influent and primary effluent lift pumps, which need to be able to match incoming flows, low flows are also an important consideration for pump sizing and operational scheme. Low influent flows typically occur in the early morning hours when activity in the city is low. Effluent data for daily minimum flow reveals that the 10^{th} percentile of low flows is 2.2 mgd or 1,500 gpm. Influent data for two weeks during November and December 2018 was also available and yielded low flows near 3.74 mgd or 2,600 gpm. For planning purposes, 2,000 gpm (2.88 mgd) are carried forward.

Influent load and desired effluent quality are considered when evaluating treatment process components and their ability to treat the incoming waste load to the desired effluent quality. As desired effluent quality is dictated largely by MPDES permit requirements (see Chapter 2), these requirements may change significantly after two to three permit cycles. Therefore, a matrix was developed to show the different planning aspects for different flows/loads and current and potential future permitting situations. 2037 flows and loads were used as presented in Chapter 2. Future permit limits were based on the assumption that Missoula would be operating under a General Nutrient Variance with the same limits as currently included in Circular DEQ-12B (Mt. Dept. of Env. Quality, May 2018). These limits are given as a concentration but permitting generally calculates a load based limit using the plant design capacity. As anti-backsliding rules prevent MDEQ from increasing nutrient load limits as compared to current limits, it was assumed that future load limits would be calculated using the currently listed plant design flow of 12 mgd. In addition, alternatives addressing aging equipment for WAS thickening is also presented. Table 8-1 shows the different conditions and four scenarios for which alternatives were developed:

Table 8-1: Evaluation Scenarios and Alternatives

Evaluation Scenarios		Flow and Load Conditions		Age / Performance
		2037 Maximum Month Flow and Load: Flow 14.3 mgd $cBOD_5$ 28,950 lb/d TSS 34,600 lb/d NH_3 3,610 lb/d TN 4,940 lb/d TP 650 lb/d	**Current Minimum Flow:** 2.2 mgd **2037 Peak Hour Flow:** 26.6 mgd	--
Effluent Quality Conditions	**2015 Avg. Monthly Permit Limits:** TN 910 lb/d TP 101 lb/d	***1. Alternatives for Compliance with Current Permit Limits at 2037 Flows & Loads***	--	--
	2017 Variance Limits at 2037 Avg. Flow TN 6.0 mg/L (600 lb/d) TP 0.3 mg/L (30 lb/d)	***2. Alternatives for Compliance with Lower Nutrient Limits at 2037 Flows and Loads***	--	--
	Other	***3. Alternatives for Compliance with E. Coli Limits at 2037 Flows***	***4. Alternatives for Meeting Plant Hydraulic Capacity Requirements***	***5. Equipment Replacement Due to Age or Performance***

Based on this matrix, the following options and alternatives were developed. Alternatives designated with a "P" are process options, alternatives designated with an "E" are equipment options, and alternatives designated with a "D" are effluent disposal options.

1. **Alternatives for Compliance with Current Permit Limits at 2037 Flows and Loads:**
   - Alternative P-1 – Operational Changes
   - Alternative P-2 – Side Stream Treatment for Nutrients
   - Alternative D-1 – Poplar Farm Expansion

2. **Alternatives for Compliance with Lower Nutrient Limits at 2037 Flows and Loads:**
   - Alternative P-3 – Change in Process Configuration and Addition of a Process Train and Clarifier
     - Alternative P-3A – Chemical Addition for Alternative P-3
     - Alternative P-3B – Tertiary Filtration for Alternative P-3
   - Alternative P-4 – Membrane Bioreactor in Existing Basins with Change in Process Configuration
   - Alternative P-5 – Switching to an Alternative Treatment Process
   - Alternative P-6 – Tertiary Treatment for Nutrient Polishing

- Alternative D-2 – New Outfall in Different Receiving Water

3. **Alternatives for Compliance with E. coli Limits at 2037 Flows:**
   - Alternative E-1A – Replacement of UV Lamps for Increased System Capacity
   - Alternative E-1B – Installation of Redundant System in Spare Channel

4. **Alternatives for Meeting Plant Hydraulic Capacity Requirements:**
   - Alternative E-2 – Influent Lift Pumping Improvements
   - Alternative E-3 – Overall Plant Hydraulic Capacity Improvements
   - Alternative E-4 – Replacement of UV System Level Control Gates
   - Alternative E-5 – Side 2 Primary Effluent Lift Pump Replacement

5. **Equipment Replacement Due to Age or Performance:**
   - Alternative E-6A – Replacement of DAF Thickener with New DAF Thickener
   - Alternative E-6B – Replacement of DAF Thickener with a Volute Thickener

8.3. GENERAL ALTERNATIVE EVALUATION CONSIDERATIONS

The alternatives were evaluated for process performance and capacity, capital and operational costs, feasibility, including ease of incorporation into the existing WWTP, and other factors as they applied to individual alternatives. The following sections discuss some of these considerations.

8.3.1. Process Performance – Capabilities and Limitations of Process Modeling

Process modeling with the BioWin software was used extensively to help predict process performance at future conditions. The model was set up initially to replicate current plant performance for effluent quality, side stream concentrations, and solids concentrations at various stages in the process. Model elements represent plant components and input was based on actual basin volumes, configurations, aeration, side and recycle stream flow rates, and operational parameters including water temperature. The influent was specified according to available influent sampling data but model defaults were used for unknown parameters. These model default values are based on empirical information from many wastewater treatment plants and represent a robust basis for any plant model treating typical municipal wastewater. Model defaults were also largely used for kinetic parameters that influence bacterial activity calculated by the model. Some of these parameters were adjusted to match the actual performance of the plant. Once the current plant performance was matched, influent flows and loads, as well as process configurations were changed for various evaluated alternatives as described below. In addition, the removal efficiency of the primary clarifiers was reduced to match the predictions for the 2037 flows shown in Chapter 7, Figures 7-7 and 7-8.

While the BioWin model provides a powerful tool for testing what-if scenarios and future situations, it is only a model and works with the input it is given. It is difficult to accurately predict the exact composition of future wastewater, including temperatures, or how this might affect the biology in the plant. Therefore, modeling output used to size and cost the various alternatives discussed below is considered a reasonable assumption of future plant performance based on today's knowledge. It provides a foundation for high level planning based on predicted plant performance. The model output generated for this report does not replace ongoing process monitoring, data collection, and re-evaluation of options in the future when a plant upgrade has become more imminent.

8.3.2. Capital and Operational Cost Estimating

Two types of capital cost estimates are presented in this report: Class 3 estimates and Class 4/5 estimates. Classifications correspond to the AACE International definitions (AACE International, 2016). The AACE established definitions commonly used in cost estimating and collected and published the limits of confidence associated with different AACE-defined levels of cost estimates. Table 8-2 lists the AACE International characteristics of the cost estimate classes used in this report.

Table 8-2: AACE International Cost Estimate Classes 3 to 5

AACE Estimate Class	Class 5	Class 4	Class 3
Estimate Methodology	Parametric or Capacity Factored	Equipment Factored or Parametric	Semi-Detailed Unit Costs with Assembly Level Line Items
Expected Level of Accuracy	-50% to +100%	-30% to +50%	-20% to +30%
Maturity Level of Project Definition Deliverables (% of complete definition)	0% - 2%	1% - 15%	10% - 40%
As Applied in this Report	Initiation	Facility Planning	Preliminary Design

Overall, the cost estimates presented in this report are based on high level budgetary cost estimates from equipment manufacturers and known scaled costs from past construction projects. Ancillary facilities and equipment, including structural work, electrical, controls, HVAC, plumbing, and other more detailed project components were not detailed but rather included in the contingency. Using Class 4/5 cost estimating, a contingency of 65% was applied to all treatment alternative cost estimates to cover ancillary equipment, limited changes in project scope, unknown future bidding climates, and other uncertainties inherent to this level of planning. Class 3 cost estimating with a 25% contingency was employed for several equipment alternatives that have a more defined scope, fewer or no ancillary components, available manufacturer quotes for the equipment itself, and an implementation schedule within the next five years. Total costs are presented in the text of this report and more detailed estimates are included in Appendix 8-1.

Operational costs were estimated for significant energy or materials use or conservation thereof only. Operational costs for any processes that are currently in place were not accounted for, unless the evaluated process or equipment offered a significant reduction compared to current operations. Energy use was based on equipment operation at nameplate power and estimated operating times. Chemical use was based on an estimated average demand with input from system manufacturers and experience from other treatment plants. The presented operational costs allow for rough conclusions regarding financial feasibility of alternatives but do not offer sufficient detail for use in meaningful present worth analyses or operational planning.

8.3.3. Feasibility

Aside from cost, the primary aspect of feasibility used in this report was ease of implementation at the existing site. The 2002 WWTP Record Drawings and 2012 Headworks and Odor Control Record Drawings were consulted to estimate how easily the structures required for the alternatives could be integrated into the existing plant, given known exposed and buried structures, piping, and utilities.

Energy and resource use were considered for alternatives that would significantly increase or decrease energy consumption or chemical usage. The operational cost estimates prepared for resource use or conservation allow for contrasting high or low resource consumption of the evaluated alternatives; however, at this level of planning, accuracy of actual costs is low because many assumptions and estimates for pump head, operational hours, air demand, chemical use and other operational variables were made. If a more in-depth energy and resource use analysis is needed to optimize existing processes as well as future investments, it is recommended that a separate study be conducted that would include all blowers, pumps, chemical feed systems, digester gas and natural gas use to determine areas where investment in newer equipment or technology would have the most impact on reducing the plant's overall energy and resource use.

Public perception was a consideration connected to the disposal options. While public education has its place in working to change public perception, in some instances it may not be warranted, given other comparative options that carry less public "stigma."

8.4. ALTERNATIVES FOR COMPLIANCE WITH CURRENT PERMIT LIMITS AT 2037 FLOWS AND LOADS

Under this scenario, the flows and loads have increased to the point of requiring capacity improvements and slight process modifications; however, the MPDES effluent limits have not yet changed significantly, especially with respect to nutrients. Alternatives under this scenario include nutrient treatment capacity expansion options that would still be beneficial after effluent limits for nutrients become stricter.

8.4.1. Alternatives Summary

- Alternative P-1 – Operational Changes
    - This alternative explores modifying the solids retention time (SRT) and aeration schemes to increase the capacity of the existing bioreactors and optimize nutrient removal.

- Alternative P-2 – Side Stream Treatment for Nutrients
    - This alternative explores the effectiveness of removing nutrients from the volute press return stream to reduce nutrient loading to the treatment process for improved nutrient removal.
- Alternative D-1 – Poplar Farm Expansion
    - This alternative explores the effects and implications of expanding the poplar farm to further reduce nutrient load to the river during the growing season.

8.4.2. Alternative P-1 – Operational Changes

The biological process on both bioreactor sides has been operated at mixed liquor suspended solids (MLSS) concentrations between 2,100 and 2,300 mg/L. As noted in Chapter 7, treatment of up to about 9.2 mgd with the current operational configuration will be able to produce effluent meeting the current permit limits. A BioWin model set up to emulate current operations was used to simulate the effect of the increased 2037 flow and loading on the existing process. Results predictably included total nitrogen (TN) and total phosphorous (TP) effluent mass loads in excess of the current permit limits.

Operation at Higher Solids Retention Time. Increasing the solids retention time (SRT), increases the MLSS concentration, which means more bacteria are available for wastewater processes in the same overall treatment volume. Model output suggests that operation at a higher SRT/MLSS would allow for successful overall treatment and improved nutrient removal at 2037 annual average and maximum month flows and loads. MLSS concentrations between 3,500 and 4,300 mg/L appear to produce effluent that would be close to meeting current permit requirements, while not overloading the secondary clarifiers. When subtracting the nutrient load diverted to poplar irrigation, effluent could potentially meet current permit requirements.

The increase in MLSS concentration could be implemented gradually as influent loads increase to keep up permit compliance; however, it is not a permanent solution and eventually the existing plant would simply reach its treatment capacity. In addition, a higher SRT may have side effects such as changes in settling behavior or changes in the microbe composition. Especially during cold weather, plant staff would need to monitor mixed liquor quality to ensure that the longer SRT does not favor higher level organisms that may lead to foaming events.

Operation with a Post-Anoxic Zone. A second strategy would be to create a post-anoxic zone by turning on air in the swing zones and turning off air to the last aerated cell. At summer wastewater temperatures, the model predicts that the remaining aerated volume is sufficient for near-complete nitrification, while a larger volume becomes available for denitrification, yielding significantly lower effluent total nitrogen concentrations. A steady state model that assumes that the last cell is mixed and operating at the higher SRT described above predicts effluent concentrations for TN and TP that would allow the plant to meet current permit limits during 2037 maximum month flows and loads.

To operate the system as modeled, mixers would need to be installed in the last aeration basin of each process train. Alternately, operators may experiment with air on-off methods for the last cell that would find a balance between providing an anoxic environment and periodically re-suspending the biomass to allow it to pass through the process. Whether mixers are installed or air is turned on and off, it would be recommended to install online ammonia probes to ensure that sufficient air for nitrification is available at all times. During winter at colder wastewater temperatures, the aeration of the last cell would likely be needed to ensure complete nitrification. This would imply seasonal switching of the bioreactor configuration.

Dissolved Oxygen Considerations. One consequence of this mode of operation of the bioreactors would be discharge of lower dissolved oxygen concentrations because the last process zone would not be aerated. Downstream facilities would re-aerate the bioreactor effluent through surface aeration in the clarifiers and open channels and through air entrainment at the clarifier weirs, UV system weir, and effluent weir. This re-aeration cannot be quantified at this time and would need to be determined experimentally. The Missoula effluent would be very low in carbonaceous and nitrogenous oxygen demand and would not exert an appreciable oxygen demand on the river; therefore, the river DO concentration would be lowered at the point of mixing with the effluent plume by simple mixing as discussed in Chapter 2. Assuming a re-aerated effluent DO concentration of 3.0 mg/L, the DO concentration after mixing (~250 feet downstream of the outfall) would be 7.3 mg/L, which is about 1.2 mg/L below the DO_{sat} in the river at summer temperatures. This concentration is also above all of the DO standards listed in Circular DEQ-7 (Montana Dept. of Env. Quality, May 2017) except for the water column values recommended to achieve required inter-gravel DO concentrations of 5.0 to 6.5 mg/L. Preliminary calculations performed to determine a DO sag curve or DO deficit after initial mixing of the effluent with the river water show that no further depression of DO is to be expected. Instead a gradual increase in DO concentration would occur through reaeration and continued mixing with the ambient flow. Nevertheless, consultation with MDEQ is recommended prior to implementing a treatment regime that places an unaerated zone at the end of the treatment system.

Impact on Effluent Nutrient Concentrations. The existing bioreactor systems when operated at a higher SRT and with a post-anoxic zone during the summer months should be able to achieve the following average effluent nutrient concentrations and mass loads for the maximum month 2037 flow of 14.3 mgd:

- TN: 6.54 mg/L; 780 lb/d
- TP: 0.52 mg/L; 62 lb/d

These concentrations and calculated loads for maximum month conditions would be in compliance with current permit limits.

Equipment

While the above described options may be implemented for little cost, for the purposes of providing a capital cost estimate, it was assumed that new mixers would be installed in the last bioreactor cells. There currently are mixers in these cells but they are only sized to mix approximately half of each basin. For complete mixing, one additional mixer would need to be added to each of the basins. The following equipment was included in the capital cost estimate for this alternative:

- Mixers (4)
- Ammonia probes (4)

Integration into the existing SCADA system was assumed to be limited to monitoring of status and output of the mixers and ammonia probes.

Capital & Operational Costs

The capital cost estimate for this alternative includes the above listed equipment and is estimated to be $256,000. Operational costs are not expected to be different from current costs; however, a small energy savings may be realized when the last bioreactor cell in each train is not aerated. Most of this air would be diverted to the preceding aerated cells for $cBOD_5$ removal and nitrification. The additional denitrification occurring after $cBOD_5$ and ammonia are mostly removed is not expected to lower the overall oxygen demand of the process significantly.

Feasibility

This alternative comprises limited installation of new in-basin equipment without the need for structural or piping work. Therefore, the capital cost is comparatively low. The alternative also does not add new energy use but may help conserve some aeration energy when air to the last aerated cell is turned off during the nutrient removal season. Alternative P-1 is a no-cost to low-cost alternative that requires minimal additional equipment and no chemical addition or major capital investment. It may be implemented any time it appears warranted based on influent flows and loads and effluent quality and as plant staff deem appropriate.

Overall, this alternative was ranked with high feasibility for implementation due to low cost, ease of installation within the existing system, and high chance for keeping the WWTP in compliance with existing permit limits for nutrients at flows and loads up to 2037 maximum month conditions.

8.4.3. Alternative P-2 – Side Stream Treatment for Nutrients

Rationale

Digested primary sludge and thickened waste activated sludge (TWAS) are combined prior to dewatering in the volute press. The return stream (pressate) from the volute press contains high concentrations of nutrients. Typically, the phosphorous in the pressate is almost entirely comprised of soluble reactive phosphorous (SRP or ortho-phosphate) and a large part of the nitrogen is in the form of ammonia, although other forms of nitrogen are also present. Removing these nutrients from the pressate before the stream is reintroduced to the main process, reduces nutrient load on the main process. This reduced load means that more capacity exists for treating influent nutrients. While this alternative would provide some immediate benefit for nutrient removal, it would also benefit any subsequent process expansion.

Recent data for the pressate is not available; however, sampling for SRP was performed on the return streams from the centrifuge (centrate) and TWAS from 2006 through 2009. Results show SRP concentrations in TWAS averaging about 550 mg/L with upper range concentrations between 800 and

1,100 mg/L. Only a few sample results are available for the digested primary sludge and suggest that it is comparatively low in SRP, less than 100 mg/L. The data for the combined centrate shows an average of 260 mg/L with upper range concentrations between 450 and 660 mg/L. The volute press is operated seven days per week with return flows varying from 60,000 to 90,000 gallons per day. An average return of 75,000 gpd with an average SRP concentration of 260 mg/L would re-introduce approximately 160 pounds per day SRP to the treatment process. With a current plant influent total phosphorous load of 300 to 380 pounds per day, this return stream load would represent 50% of the influent TP load.

Since the 2006-2009 data was collected, improvements to the aeration of the TWAS tank were made, which may have reduced the amount of phosphorous released in the storage tank. Process modeling results suggest that the pressate returns only between 50 and 100 pounds of SRP per day to the process. However, in the absence of more recent plant data, the side stream treatment system and associated costs were based on the 2006-2009 data with some projections to account for higher plant flows in the future.

Table 8-3: Side Stream Treatment Design Criteria

Treatment Parameter	Influent	Effluent
Equalized Design Flow for 24-hr Continuous Feed [1]	100,000 gpd	--
Design Temperature [2]	30°C	--
BOD [3]	300 mg/L	--
Soluble COD [3]	500 mg/L	--
TSS [3]	500 mg/L	--
NH_3-N [4]	600 mg/L	<60
Alkalinity [3]	>3,000 mg/L	--
Ortho-P [4]	260 mg/L	<52

1. Projected for 2037.
2. For optimal process sizing. At lower temperatures, facilities may be larger.
3. Based on manufacturer’s experience at other plants.
4. Based on manufacturer’s experience and 2006-2009 data.

Phosphorous Removal. The side stream phosphorous removal process evaluated for this report converts soluble reactive phosphorous and ammonia to struvite, a natural slow-release fertilizer. Struvite formation is accomplished by a chemical precipitation process that requires the addition of magnesium and presence of ammonia to react with the SRP to form an insoluble precipitate. By dosing magnesium as magnesium oxide (MgO) into the side stream reactor, the magnesium will combine with the SRP and ammonium and precipitate as struvite. This process happens spontaneously in many treatment plants where secondary sludge from nutrient removal processes is anaerobically digested. Struvite precipitation has the potential for clogging pipes and equipment. This treatment process makes use of this precipitation reaction and allows for forming struvite where it can be controlled and removed.

The process evaluated for this report is not fully set up to produce clean struvite but rather a sludge that is very high in struvite and may be disposed of at the composting facility along with the regular plant sludge. The evaluated process may be augmented to include struvite dewatering, cleaning/washing and pelletizing to produce a product that can be sold as fertilizer. Another method is offered by the proprietary Ostara process, which produces clean struvite beads that Ostara buys back from the facility for commercial re-sale. These options were not explored in detail as the feasibility of this additional investment can only be studied when the full composition of the recycle streams with respect to phosphorous, magnesium, COD, ammonia, and other nitrogen species is known.

Nitrogen Removal. The nitrogen removal side stream treatment process would follow the phosphorous process and remove any ammonia and other nitrogen species that have not been utilized during struvite formation. The process evaluated for this report utilizes the same type of granular biomass as discussed in Alternative P-5 for potential future mainstream treatment. The granules are composed of a number of different bacteria that perform different functions depending on the amount of oxygen they receive as it diffuses into the granule. The same granule is able to simultaneously perform nitrification and denitrification without a separate process step. The process makes use of ammonia oxidizing bacteria (AOBs), which convert ammonia to nitrite and anammox bacteria, which convert nitrite and ammonia to nitrogen gas. The conversion to nitrate is largely skipped, leading to lower oxygen and zero carbon (food source) requirements. Therefore, side stream nitrogen removal can occur in a single process step located in a single tank.

Implementing side stream treatment and establishing a healthy source of granular biomass in the side stream reactor may facilitate an easier transition to implementation of a granular activated sludge process in the mainstream at some point in the future as discussed in Alternative P-5. Furthermore, as also discussed in Alternative P-5, experimentation is currently under way to also use the granular biomass for simultaneous phosphorus removal. This would require a food source for the phosphorous accumulating organisms or PAOs during side stream treatment.

Impact on Effluent Nutrient Concentrations. Modeling for annual average and maximum month 2037 flows and loads shows that the addition of side stream treatment to the existing treatment facility would likely produce effluent in compliance with current effluent limits. For annual average conditions, the process would produce total nitrogen concentrations of less than 9.5 mg/L, resulting in a load of 890 lb/d or less discharged to the Clark Fork River without diversion to the poplar farm. Modeling results also confirm that the existing process configuration has limitations with respect to denitrification and total nitrogen removal; however, installation of side stream treatment would postpone the necessity for an overall process upgrade. The following maximum month effluent nutrients are predicted for this alternative:

- TN: 9.9 mg/L; 1,181 lb/d
- TP: 0.42 mg/L; 50 lb/d

At maximum month flows, the predicted effluent nitrogen concentration of 9.9 mg/L would cause the effluent load to exceed the permitted mass by 270 lb/d – more than what can be diverted to the poplar farm. Without operational changes as described for Alternative P-1, side stream treatment would not consistently be able to produce effluent in compliance with current total nitrogen limits. However, the performance for phosphorous is better and produces effluent in compliance with current limits.

Equipment and Structures

Phosphorous Removal. Pressate would be pumped from the existing pressate drain line to the side stream reactor. The treatment process would occur in a single aerated reactor. The struvite would be kept in suspension by air mixing to avoid scale buildup on reactor components. The aeration would also ensure that any residual organics are converted to new biomass and carbon dioxide. A settler would be located within the reactor, where the struvite and sludge mixture would be allowed to settle out. Sludge/struvite would be withdrawn from the bottom of the reactor. The reactor may be housed in a concrete or steel tank. A steel tank reactor with interior and exterior coating was used for this evaluation. A building would be needed to house chemical feed equipment, blowers, and pumps. Major equipment and structures would be as follows:

- Circular steel tank; volume = 21,000 gal; diameter = 6.25 ft; SWD = 23 ft
- Building; 40 ft x 32 ft
- Blowers (2) and aeration system
- Sludge/struvite pumps (2)
- Chemical feed pumps (2)
- Pressate lift pumps (2)

It was assumed that the volute press can be operated to generate a continuous pressate stream. If this is not possible, an equalization tank would need to be added to provide a continuous and even influent flow to the side stream treatment system. Cost or feasibility of construction of an equalization basin was not included in this alternative analysis.

Nitrogen Removal. This process takes place in a continuous flow aerated tank designed to retain biomass and select for the granules that contain the AOBs and anammox bacteria. Aeration is also controlled to encourage growth of the granules rather than floc biomass. Occasional wasting of biomass will require pumping, especially if the wasted biomass is to be used in the bioreactor. In order to achieve a manageable reactor size, the side stream entering the reactor must be heated to about 30°C. Influent heating for these systems is often partially accomplished by harvesting heat from digester effluent via heat exchangers; however, it is uncertain if effluent from digester 1 can feasibly be diverted before entering digester 2 for this purpose. Therefore, it was assumed that all side stream influent heating would be accomplished with a natural gas heater. The heater, sludge waste pumps, and blowers would be housed in the same building as the equipment for the phosphorous removal system. Major equipment and structures will be as follows:

- Concrete tank; 15 ft wide x 15 ft long; volume = 33,500 gal; SWD = 20 ft
- Settler
- Blowers (2) and aeration system
- Sludge waste pumps

- Natural gas heat exchanger
- Yard piping

SCADA integration of both treatment components was assumed to be limited to monitoring of equipment status, and instrument readouts.

Capital and Operational Costs

The capital cost for this alternative is estimated to be $7,470,000 and includes the above listed structures and equipment, as well as electrical and controls. For an average flow of 11.2 mgd, operational costs would add approximately $124,000 to the annual WWTP budget. This amount would cover magnesium oxide, aeration, pumping to the side stream treatment facility, and waste sludge pumping.

Feasibility

This treatment system could be constructed in the space of the abandoned drying beds, away from the congested area between the bioreactors and Headworks Building. It involves limited yard piping work, allowing for connecting to one existing pipe within short distance of the proposed site for this alternative.

While pumping of the entire stream to be processed is required, this stream has a small volume when compared to total plant flow. The additional pumping costs would be insignificant when compared to alternatives that require pumping of the entire plant flow.

This side stream treatment would have limited impact on effluent nutrient concentrations. It would allow the WWTP to meet current permit limits for total phosphorous up to 2037 maximum month flows/loads and permit limits for total nitrogen for 2037 average annual flows/loads, allowing the plant to stay in compliance with current nutrient limits for most of the planning period. At any time prior to reaching 2037 influent flow and load levels, side stream treatment would help nutrient removal by freeing up capacity in the bioreactor for influent nutrients and achieving lower than required effluent nutrient concentrations.

The additional operational costs are relatively high; however, if a future mainstream upgrade should use granular activated sludge (Alternative P-5), energy savings in this mainstream process may offset some of the additional operational costs for the side stream treatment.

This particular side stream nitrogen removal process would prepare the WWTP for a potential future mainstream granular activated sludge process discussed in Alternative P-5. Feasibility of this alternative may be linked to feasibility and eventual decisions regarding mainstream upgrades. However, this alternative would require a significant investment now plus additional investment in the future for meeting future effluent limits at 2037 flows and loads.

8.4.4. Alternative D-1 – Poplar Farm Expansion

Rationale

The poplar farm in its current configuration has been fully operational since 2014 with the first full season of irrigation occurring in 2015. During all four years of operation, irrigation has averaged 0.8 mgd throughout the irrigation season. At average summertime effluent concentrations of 9.7 and 0.70 mg/L for TN and TP, respectively, diversion of this irrigation water has kept approximately 65 lbs of TN and 4.7 lbs of TP daily from being discharged to the Clark Fork River. These loads equate to about eleven percent of the current effluent nutrient load for both TN and TP. The maturing trees currently have a higher water and nutrient demand than the existing pumping capacity can supply and would be able to accept up to 1.5 mgd by 2019. Procurement and installation of a larger pump was recently completed. This will decrease the load of nutrients discharged to the Clark Fork River by an additional eight percent for a total nutrient discharge reduction of 19 percent (113 lbs of TN and 8.2 lbs of TP per day), given the same effluent concentrations. However, after harvest and replanting of the mature trees, which typically occurs on a 20-year cycle, irrigation flows will be lower for the newly planted trees and fewer nutrients would be diverted to the poplars for a few years. Expansion of the poplar farm would allow for diverting overall higher flows to irrigation, which would further reduce the nutrient load discharged to the river.

Feasibility

The area of the existing poplar farm is bordered by a gravel pit on the west, the city of Missoula composting facility on the southeast, several private businesses on the northeast, the Clark Fork River on the south, and Mullan Road on the north. Expansion would only be possible on privately owned land to the southwest of the existing farm. If that property is not available, any future expansion of the poplar farm would have to occur on land located at a considerable distance from the WWTP. Pumping for long distances and site administration at a remote site would be costly and would require additional analysis to determine how the increased operating costs compare with other methods of reducing nutrients discharged to the river. Seasonal pumping costs to convey 0.5 mgd of irrigation water to a site located about 1.5 miles from the WWTP is estimated to be about $3,000 to $4,000 per season. Location, elevation, irrigation days, and required irrigation system pressures would dictate exact pumping costs.

If it is assumed that the land could be purchased or leased by the city of Missoula and that it is suitable for supporting the hybrid poplar trees and irrigation system, this area would increase the poplar farm by about 30%. At current effluent nutrient concentrations, plant flows, and 2019 irrigation rates, this increase would keep an additional 30 lb/d of TN and 2.5 lb/d of TP (five to six additional percent of effluent nutrients) from being discharged to the Clark Fork River. This reduction would be the equivalent of improving the treatment so that the effluent concentration of the flow discharged to the Clark Fork was consistently lower by 0.6 mg/L TN and 0.1 mg/L TP at current average flows. It should be noted that if a future expansion of the poplar farm paralleled implementation of treatment process improvements that lowered effluent nutrient concentrations, the load diverted to the poplar farm would be smaller and less significant with respect to total load reduction to the Clark Fork River. Table 8-4 shows nutrient load reductions for different scenarios.

Expansion of the poplar farm appears to have low feasibility with respect to land availability contiguous with the existing poplar plantation. A remote expansion location would require an in-depth cost/benefit analysis focused on operational costs once potential sites are known to determine if this option offers a feasible cost benefit ratio. Because potential location information was not available, this option was not analyzed further.

Table 8-4: Poplar Farm Impact on Effluent Nutrient Concentrations

Scenario	Flow	Effluent Concentrations	Irrigation Flow	Effluent Nutrient Load Reduction	
Current Effluent Limits	7.27 mgd	9.87 mg/L TN 0.7 mg/L TP	0.8 mgd	65 lb/d 4.7 lb/d	11%
Current Effluent Limits, Larger Irrigation Pump	7.27 mgd	9.87 mg/L TN 0.7 mg/L TP	1.5 mgd	113 lb/d 8.2 lb/d	19%
Current Effluent Limits, Expanded Poplar Farm	7.27 mgd	9.87 mg/L TN 0.7 mg/L TP	2.0 mgd	165 lb/d 11.7 lb/d	28%
Current Effluent Limits	11.2 mgd	9.87 mg/L TN 0.7 mg/L TP	1.5 mgd	123 lb/d 8.8 lb/d	13%
Lower Effluent Limits, Expanded Poplar Farm	7.27 mgd	6.0 mg/L TN 0.3 mg/L TP	2.0 mgd	100 lb/d 5.0 lb/d	28%
Lower Effluent Limits, Expanded Poplar Farm	11.2 mgd	6.0 mg/L TN 0.3 mg/L TP	2.0 mgd	100 lb/d 5.0 lb/d	18%
Lower Effluent Limits, Upgraded Treatment, Expanded Poplar Farm	11.2 mgd	4.0 mg/L TN 0.2 mg/L TP	2.0 mgd	67 lb/d 1.7 lb/d	18%

8.5. ALTERNATIVES FOR COMPLIANCE WITH LOWER PERMIT LIMITS AT 2037 FLOWS AND LOADS

The alternatives examined for this scenario are possible options for successfully operating the WWTP in about 20 years, serving a significantly higher service population and having received significantly lower effluent limits for nutrients consistent with most of the state of Montana. As presented above, the assumed nutrient limits are 6.0 mg/L for total nitrogen and 0.3 mg/L for total phosphorous. The WWTP in its current configuration will not be able to produce this effluent quality for the projected 2037 flows and loads. Four treatment alternatives that have the potential of bringing the WWTP effluent into compliance are discussed. In addition, an alternate effluent disposal option is examined to reduce nutrient loading to the river.

8.5.1. Alternatives Summary

- Alternative P-3 – Change in Process Configuration and Addition of a Process Train and Clarifier
    - This alternative would change the process configuration to optimize nutrient removal in addition to adding treatment and clarification volume.
    - Alternative P-3A – Chemical Addition for Alternative P-3

        - This alternative would provide facilities for addition of chemicals for enhanced nitrogen and phosphorous removal.
    - Alternative P-3B – Tertiary Filtration for Alternative P-3
        - This alternative would provide tertiary filtration as a means to achieve very low effluent total phosphorous concentrations.
- Alternative P-4 – Membrane Bioreactor in Existing Basins with Change in Process Configuration
    - This alternative examines use of membrane filtration in place of clarification and its capacity to produce high quality effluent in the existing bioreactor volume. A change in process configuration is necessary for this alternative to accommodate the highly oxygenated RAS stream from the membrane basins.
- Alternative P-5 – Switching to an Alternative Treatment Process
    - This alternative may be a viable option in ten to twenty years. A qualitative discussion is offered introducing alternatives to the traditional activated sludge process for nutrient removal. These processes are still largely in their infancy and are expected to have matured by the time planning for a plant upgrade will become necessary.
- Alternative P-6 – Tertiary Treatment for Nutrient Polishing
    - This alternative involves use of tertiary treatment following the existing treatment process to polish the effluent for nutrients.
- Alternative D-2 – New Outfall in Different Receiving Water
    - This alternative offers considerations for discharge of a portion or all of the effluent to an irrigation ditch during the nutrient removal season.

8.5.2. Alternative P-3 – Change in Process Configuration and Addition of a Process Train and Clarifier

Rationale

BioWin modeling was used to examine whether the biological process in its current configuration would be capable of treating the increased loads to lower limits with the addition of one bioreactor train and clarifier. Modeling results showed that the existing process configuration would not be able to meet these conditions. While effluent total phosphorous concentrations were predicted to be below 0.5 mg/L, effluent total nitrogen results were greater than 10 mg/L. Post-denitrification, located in an anoxic zone following the aerobic zone, is needed to achieve low effluent total nitrogen concentrations without tertiary treatment. This is not currently provided, limiting the extent of effective total nitrogen removal. Therefore, addition of a process train and fourth clarifier without change in process configuration or addition of tertiary treatment was not pursued further.

Instead, BioWin modeling was used to determine a process configuration better suited to low-level nitrogen and phosphorous removal than the existing configuration. Similar to Alternative P-1, the goal was to find a configuration that could be implemented within the existing concrete basins but would require changing internal wall locations, recycle stream piping, and aeration and mixing equipment. This alternative would also include the addition of a third and fourth process train on Side 2 and potentially one additional clarifier.

Process Configuration. The existing layout of the Missoula bioreactors was considered for their adaptability when selecting potential process configurations. Two processes were identified for retrofitting the Missoula WWTP: the modified (5-stage) Bardenpho process and the modified UCT (University of Cape Town) process. Both have good documented performance for nitrogen and phosphorous removal and both could feasibly be implemented in Missoula. Figure 8-1 shows schematics of the two processes as depicted in Metcalf & Eddy, Table 8-25 (Metcalf & Eddie, 2003).

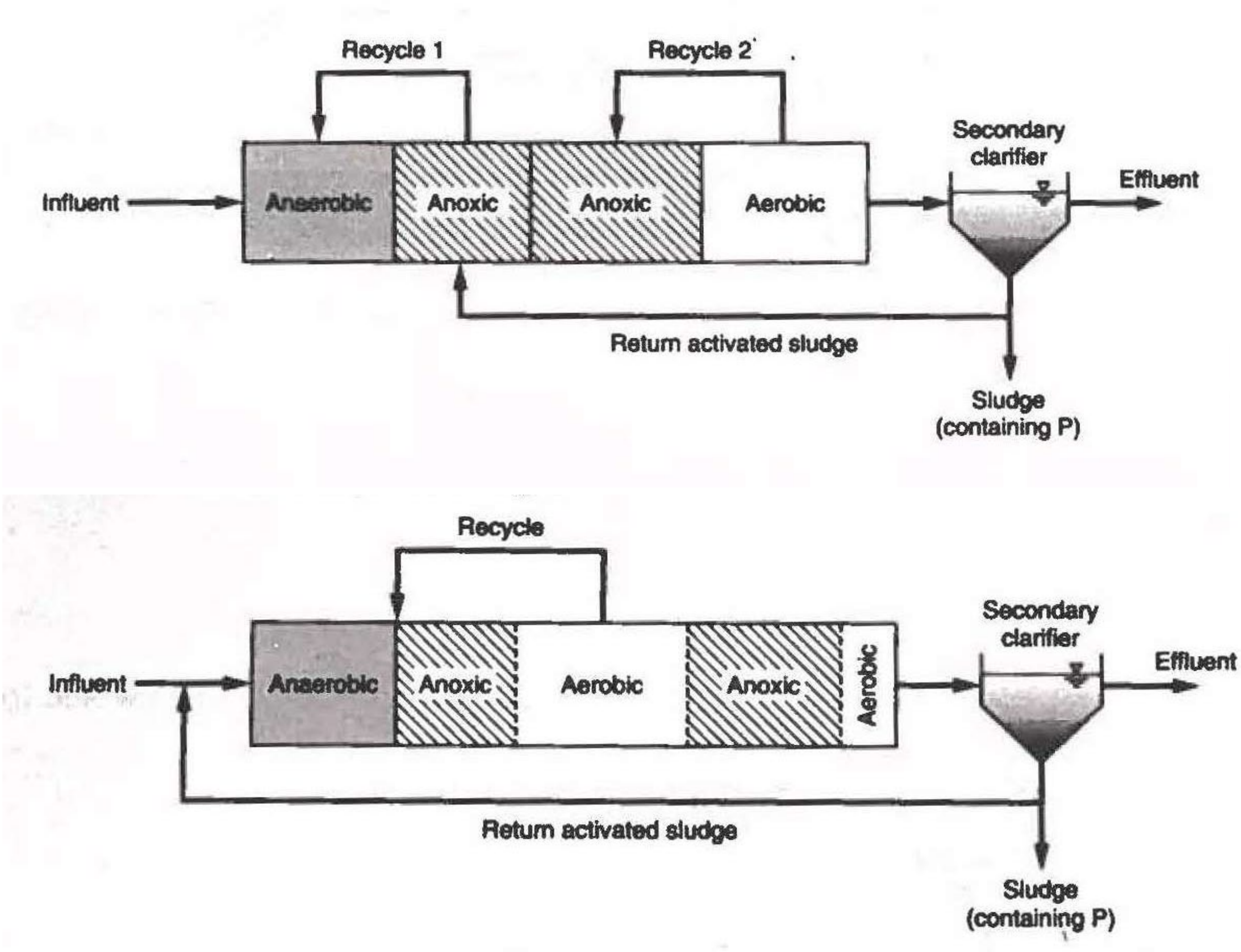

Figure 8-1. Schematics for Modified Bardenpho process (top) and Modified UCT process (bottom). Metcalf & Eddy Table 8-25, c and d (Metcalf & Eddie, 2003).

Modeled Process Performance. Both processes were modeled in BioWin using existing basin dimensions and volumes and assuming modified recycle streams and aeration/mixing regimes. Results predicted that the modified UCT process would provide excellent phosphorous removal but would not provide improved nitrogen removal compared to the existing process. Experience with other treatment plants in Montana has shown that a post-anoxic zone is necessary to achieve low level effluent nitrogen concentrations and the modified UCT process lacks this.

The 5-stage Bardenpho process includes a post-anoxic zone (anoxic zone following the aerobic zone) but was modeled without the final aerobic zone since incorporation of this zone into the existing basin layout would reduce volume needed for nutrient removal. Instead, a new re-aeration basin would be added prior to the outfall for both bioreactor sides combined. Re-aeration may be necessary following a process that ends in a non-aerated zone to ensure that MPDES effluent dissolved oxygen (DO) requirements and in-stream DO standards can be met. For this report, it was assumed that a re-aeration basin would be needed and was included in the cost estimate and feasibility considerations.

The modified 5-stage Bardenpho process was initially modeled with existing bioreactor walls unchanged and only aeration and recycle stream locations modified. This configuration did not yield satisfactory results. Subsequently, the bioreactor zones were changed in ways that would require removal and addition of walls to create optimal zone sizes. Modeling of this modified configuration at a 10-day SRT yielded promising results, for both annual average and maximum day conditions. During maximum day conditions the additional nutrient diversion to the poplar plantation would help ensure that the effluent load would meet the current Circular DEQ-12B limits.

Impact on Effluent Nutrient Concentrations. For annual average conditions, the modeled 5-stage Bardenpho configuration would be able to reduce total effluent nitrogen to between 4.5 and 5.0 mg/L and total effluent phosphorous to below 0.25 mg/L. These results would meet the planning permit limits of 6.0 and 0.3 mg/L for total nitrogen and total phosphorous, respectively.

When modeling 2037 maximum month conditions without changing MLSS concentrations, this plant configuration produced effluent concentrations below but very close to the current Circular DEQ-12B limits, indicating that if a maximum month were to occur during the nutrient season, the plant might not reliably be able to meet effluent requirements. Diversion of 1.5 mgd of effluent to the poplar farm would reduce the load discharged to the river by about 65 lb of nitrogen and 3.4 lb of phosphorous, which would keep the facility in compliance with the assumed load limit based on effluent concentrations of 6.0 and 0.3 mg/L for TN and TP, respectively, and a flow of 14.3 mgd. The following maximum month effluent limits are predicted for the 5-stage Bardenpho configuration without considering diversion to the poplar farm:

- TN: 5.23 mg/L; 624 lb/d
- TP: 0.27 mg/L; 32 lb/d

Clarification. The process model was run assuming operation of three clarifiers on Side 2. According to model output, three clarifiers would have excess capacity at 2037 maximum month flows and still operate within recommended parameters during maximum day flows. BioWin is capable of creating a state point analysis graph as shown in Figure 8-2. For a properly sized clarifier, the state point graph will show the intersection of the feed, overflow, and underflow lines below the flux curve. The closer the intersection is to the flux curve, the closer the clarifier is to reaching its capacity. Figure 8-2 shows that during maximum day flows the intersection of the flow lines nears the flux curve but is still below it. This suggests that the three existing clarifiers would be close to their maximum capacity but would still be expected to produce well settled effluent.

In order to provide redundancy, a fourth clarifier should be constructed with the new process train with routine operation occurring in only three clarifiers. Since process flow to Side 1 would not change in the future, the existing clarifiers would be adequate to settle the flow from the Side 1 bioreactor.

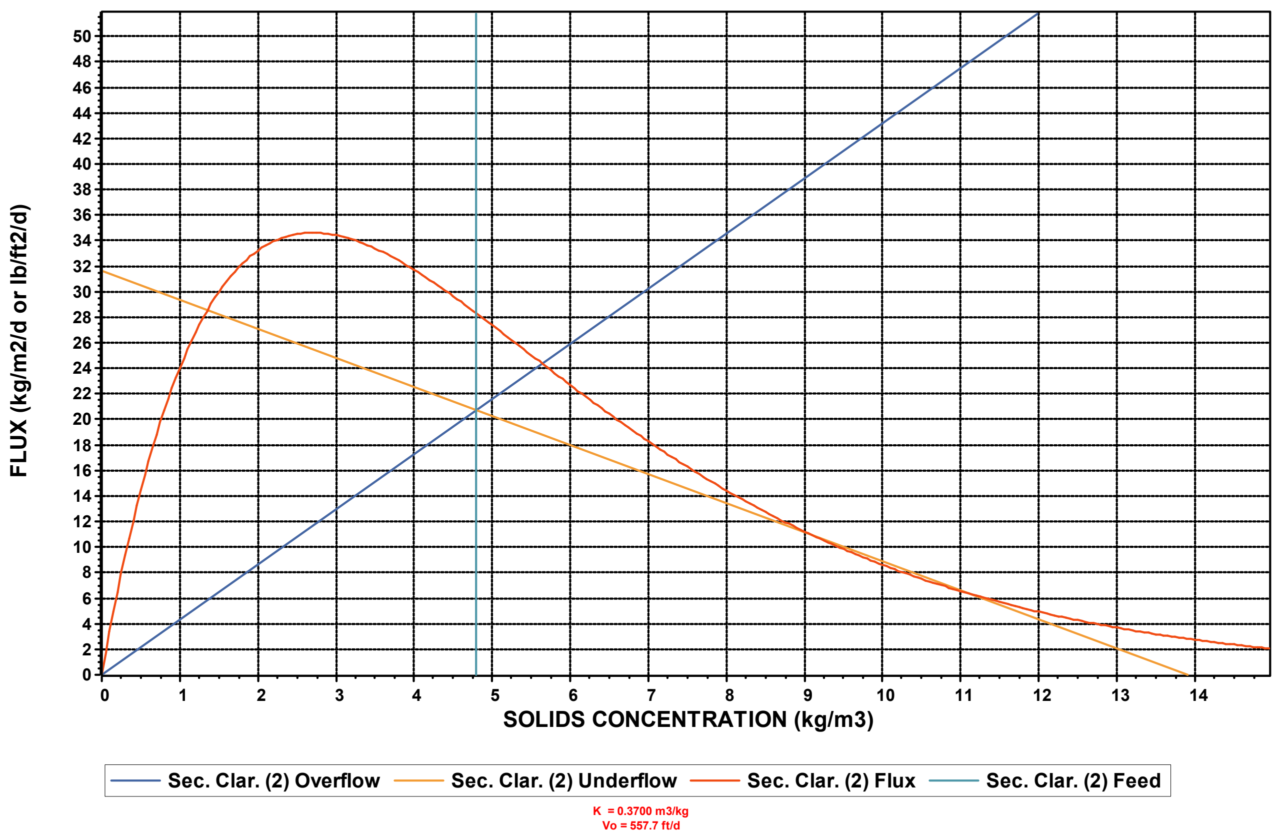

Figure 8-2. State Point Analysis for Side 2 Clarifiers at Projected 2037 Maximum Day Flow.

Re-Aeration Basin

Flow discharged from the secondary clarifiers following the above described process would have DO concentrations of 0.5 mg/L or less. Ideally, effluent discharged to the Clark Fork River should average 6.5 mg/L DO and never drop below 5.0 mg/L. While these values are below the in-stream water quality standard, they have been accepted by MDEQ in the past and no record of adverse effects on the river are known. Therefore, an average effluent concentration of 6.5 mg/L was assumed for this report. BioWin was used to estimate the basin size and air requirement to re-aerate the complete plant effluent flow and the below basin dimensions were determined to be sufficient.

Two options for construction of a re-aeration basin were explored. Re-aeration could be provided prior to UV disinfection by retrofitting the existing primary effluent overflow basin. However, the basin would need to be deeper by about 6 feet and piping connections with large-diameter pipe would need to be moved. The basin is in a congested area and any construction but especially deep construction would present a significant challenge. A second option would provide re-aeration after disinfection by construction of a new basin between clarifier no. 4 and the river bank. This option may allow for use of a smaller basin and less air, as re-aeration through the UV level control gates and the effluent weir would raise DO concentrations somewhat.

As experienced during the 2011 construction of the Headworks Building, deep construction close to the river requires extensive shoring, dewatering, and other construction considerations to complete the work. Given the congested area around the existing primary effluent overflow basin, it was assumed that it would be exceedingly difficult to perform deep construction in this area and construction of a new re-aeration basin was assumed for cost estimating and feasibility considerations. It was also assumed that a new building would be constructed near the re-aeration basin to house the blowers.

Equipment and Structures

The following major structural changes, additions, equipment, and piping would be included in this alternative:

- Existing bioreactor wall configuration changes
- Addition of a third process train on Side 2
- Addition of a fourth clarifier on Side 2
- New Blower Building for re-aeration blowers; 24 ft x 16 ft
- Re-aeration basin; 50 ft x 25 ft x 18 ft deep with baffle wall
- Two new blowers for re-aeration and associated air piping
- One new blower in existing building for bioreactor aeration and associated air piping
- In-basin aeration equipment for bioreactors and re-aeration basin
- New mixers and MLR pumps for all bioreactor basins
- Process piping modifications to existing MLR piping, secondary effluent piping, and new bioreactor and clarifier

SCADA improvements were assumed to include new bioreactor instrumentation and controls for both sides and integrated monitoring and control of all new systems, as well as tying the new controls into the existing SCADA system.

Capital and Operational Costs

The capital cost for this alternative is estimated to be $11,777,000. Additional operational costs are generated by a higher air demand for treating a larger influent load. Energy costs for the additional aeration were estimated to be about $86,000 per year. This additional cost can be expected for all conventional activated sludge processes at the 2037 flows and loads.

Feasibility

The existing facility includes provisions for the addition of a third Side 2 process train and forth clarifier, making siting, piping connections, and process tie-in of the new basins relatively simple. However, the existing bioreactor basin modifications would add complexity as they would require shut down of entire process trains while treatment must continually be provided.

The additional operational costs are proportional to the increase in flows and loads and would not be considered a feasibility concern.

Construction of a re-aeration basin would be challenging but given the relatively small size of the basin, a potential site exists that offers little to no congestion by existing structures and piping. The hydraulic profile calculator was checked to ensure that the WWTP can hydraulically accommodate addition of this basin. Overall, construction of a re-aeration basin was considered reasonably feasible.

8.5.3. Alternative P-3A – Chemical Addition for Alternative P-3

This alternative would be additive to Alternative P-3 to ensure reliable nutrient removal if Alternative P-3 alone cannot achieve the required effluent nutrient concentrations. Alternative P-3A proposes seasonal chemical addition to enhance both nitrogen and phosphorous removal. Alum addition at the end of the post-anoxic zone may be used to achieve lower effluent total phosphorous concentrations. Rapid mixing would need to be provided at the injection point to facilitate contact between the SRP and the alum. The precipitate would settle with the sludge in the clarifiers. Alum sludge is somewhat slow-settling and operating three clarifiers on Side 2 would help facilitate good settling.

For nitrogen removal, a carbon source low in nitrogen and phosphorous would be dosed into the post-anoxic zone to provide a food source for the denitrifying bacteria and achieve better denitrification prior to recycling the flow back to the anaerobic zone or discharging to effluent. One method for adding a carbon source is to utilize step feeding of influent flow, with some flow fed to the head of the process and some flow fed to the post-anoxic zone. Other methods include addition of brewery waste and chemical addition to the post-anoxic zone. The current piping and flow splitting arrangement would require significant changes to allow for step feeding influent, which was not considered further for this study. Consistent availability and quality of brewery waste for fine-tuned nitrogen removal are hurdles that will need to be cleared before consideration of brewery waste in this capacity. For the purposes of this report, only chemical addition was considered but future process decisions may consider step-feeding of influent and addition of brewery waste as a potential alternative.

BioWin uses methanol as a default carbon source; however, there are some considerations advising against methanol. While methanol is inexpensive, it is explosive and requires thorough safety precautions for storage and handling. A methanol carbon source also requires an acclimation period of the biomass as only certain bacteria like methanol. Other carbon sources, such as glycerin-based products, do not require an acclimation period. Micro-C is a proprietary, glycerin-based carbon source with concentrated and consistent carbon content and has been used at the Butte-Silver Bow WWTP with success. Enhanced denitrification in the post-anoxic zone may also improve biological phosphorous removal as experienced at the Butte-Silver Bow WWTP by minimizing the nitrates returned to the anaerobic zone. In addition, glycerin-based carbon sources recycled to the anaerobic zone have been shown to stimulate phosphorous release by augmenting VFAs and other readily available food sources for phosphorous accumulating organisms. The increased bio-P removal observed in Butte during addition of Micro-C may have in part been due to this effect. Modeling does not indicate these relationships for the Missoula WWTP; however, BioWin does not account for effects of an added carbon source to phosphorous removal.

Impact on Effluent Nutrient Concentrations. Chemical addition can achieve the following average effluent nutrient concentrations and mass loads for the maximum month 2037 flow of 14.3 mgd:

- TN: 4.75 mg/L; 566 lb/d with methanol addition
- TP: 0.23 mg/L; 27 lb/d with alum addition

Note that the added chemical quantities were not increased beyond meeting the planning effluent limits. It is possible that lower effluent nutrient concentrations may be achieved by adding higher volumes of either or both chemicals.

Equipment and Structures

Equipment needed for chemical addition would likely include two new buildings for the equipment to minimize feed line lengths to the two bioreactors. Major structures and equipment would be as follows:

- Two buildings; 28 ft x 20 ft
- Two sets of alum and carbon source storage and feed equipment
- Buried small diameter piping and in-basin piping
- Two alum mixers

Capital and Operational Costs

The capital cost of the chemical feed systems as described is estimated to be $933,000. If only an alum feed were needed as discussed for Alternative P-3B, the cost would be $662,000. Additional operational costs would be incurred for chemical use and were estimated to be about $93,000 per season, assuming 100 days of chemical use. Actual use and associated cost may vary as it will depend on the starting point and desired end point concentrations of nitrogen and phosphorous to be achieved by chemical removal.

Feasibility

Locations for a building to house the chemical feed equipment would be available south of clarifier no. 3 for Side 1 and south of clarifier no. 6 on Side 2. Capital costs for these facilities would be small when compared to other alternatives capable of reducing effluent nutrient concentrations below the levels discussed for Alternative P-3 above.

Using chemical addition to reduce effluent nutrients would introduce additional operational cost and potential environmental concerns associated with chemical delivery and storage. However, chemical addition would only be required for three months per year, thus limiting additional operational costs and environmental concerns. Onsite chemical storage could be limited to the nutrient removal season, so that tanks would be empty for the remainder of the year,

8.5.4. Alternative P-3B – Tertiary Filtration for Alternative P-3

This alternative may be considered in place of Alternative P-3A or in conjunction with it. It would depend on the success of phosphorous removal by Alternatives P-3 and P-3A, as well as future permit limits. If limits for effluent phosphorous become extremely strict, chemical addition alone may not be sufficient for meeting very low phosphorous limits.

Alternative P-3B proposes effluent filtration to enhance phosphorous removal only. While not addressing nitrogen removal, filtration has been proven to work well for phosphorous removal in combination with alum addition by providing a barrier for solids associated with the precipitated phosphorous. This option would only be considered if bio-P removal plus alum addition (Alternative P-3A) were not able to reliably reduce effluent phosphorous concentrations to below permit limits or if permit limits were lower than the 0.3 mg/L concentration assumed for this report. For this alternative, cloth media filtration was explored. Cloth media filter units offer a phased approach to filtration and installation in two phases was assumed for Missoula. The first phase would have capacity to treat flows up to nine mgd. Flows in excess of nine mgd would bypass filtration and be blended prior to re-aeration and disinfection. The second phase would expand the filtration system to flows up to 14.3 mgd with flows over 14.3 mgd being bypassed and blended with filtered effluent.

Ten years ago, extensive pilot testing was performed at the WWTPs in Spokane and Coeur d'Alene to determine the best filtration method for achieving effluent TP concentrations below 0.02 mg/L, which is an order of magnitude lower than the planning limits assumed for Missoula. Should limits drop this low, results from these pilot studies may be used to develop alternatives for Missoula and pilot testing would be used to identify the best filtration method for the Missoula WWTP.

Impact on Effluent Nutrient Concentrations. The predicted effluent phosphorous concentrations for the filtration system evaluated for this alternative would be expected to reliably be below 0.25 mg/L for the full build-out of the system at the maximum month 2037 flow of 14.3 mgd. At lower flows, lower effluent concentrations would be achieved.

- TN: n/a
- TP: 0.25 mg/L; 30 lb/d

These concentrations and calculated loads for maximum month conditions would be just below the current permit limits but would be expected to be reliably in compliance at average flows.

Equipment and Structures

A filtration building would be required for housing the filters in steel tanks and associated equipment. Yard piping would need to be installed to route secondary effluent from the clarifiers to the new filter building and from the filter building to the re-aeration facility. Pumping of all flow to the filtration facility would be required as the existing plant hydraulic profile does not have enough vertical space to allow for insertion of a major treatment step by gravity flow prior to effluent discharge. Major equipment and structures assumed for this alternative are as follows:

- Filtration building; 40 ft x 24 ft
- Filter equipment
- Filtration System Lift Pumps
- Backwash Waste Pumps
- Process piping and valves

Capital and Operational Costs

The capital costs for this alternative would include the filtration facility ($6,501,000), as well as the alum dosing facilities discussed in Alternative P-3A ($662,000) for a total estimated capital cost of $7,163,000. The most significant operational cost increase for the filtration facility alone would be for pumping the entire plant flow to the filtration facility, which was estimated at about $21,000. Added to this would be approximately $40,000 for the addition of alum for a total operational increase of about $61,000 per year.

Feasibility

Filtration as proposed here, would only address enhanced phosphorous removal with little effect on effluent nitrogen and may not address the needs of the Missoula WWTP as well as other more comprehensive alternatives. When effluent limits dictate ultra-low levels of total phosphorous, filtration becomes a preferred method for ensuring reliable permit compliance. In the case of Missoula, other alternatives evaluated would meet the 0.3 mg/L total phosphorus planning limit while also providing enhanced nitrogen removal.

Siting of a tertiary filtration facility would be difficult given the existing layout and yard piping at the WWTP. If a tertiary facility were to become necessary, evaluation of alternatives that would incorporate existing structures and facilities is recommended in order to avoid costly construction of new buildings on a congested site with limited expansion capacity.

The Missoula WWTP pumps all its flow twice, influent in the headworks and primary effluent. Adding a third pumping location and the associated energy costs would not be in the interest of keeping the facility energy efficient. Other alternatives that do not require pumping of the entire plant flow would have a higher feasibility for implementation.

If water reuse at public sites were a goal for the future, this facility would meet the Circular DEQ-2 (Montana Dept. of Env. Quality, 2016) requirement for tertiary filtration. However, unless the entire plant flow was to be reused, investment into a smaller filtration facility for only a portion of the flow would likely be more feasible.

8.5.5. Alternative P-4 – Membrane Bioreactor in Existing Basins with Change in Process Configuration

Rationale

Membrane bioreactor systems essentially replace secondary clarifiers with membrane filtration. Depending on the membrane type, this may be micro- or ultrafiltration, both of which provide an effective barrier to organisms and solids contained in the mixed liquor. The filtration capabilities of the membranes allow for operating the bioreactor process at higher MLSS concentrations than is possible with conventional clarification. The higher MLSS concentration provides more biomass to treat higher flows and loads in the existing reactor volume and eliminates the need for conventional clarification.

This option was explored for the Missoula WWTP to analyze whether membrane filtration could provide adequate treatment, within the current bioreactor volume without the need for additional treatment trains. For the purpose of this Facility Plan, it was assumed that each side was upgraded separately, rather than combining effluent from both bioreactor sides into a single membrane basin. Given existing RAS piping configurations and challenges associated with routing flow across the site, combining flow from both sides into one membrane basin was not considered further. Upgrading each side separately would also have the potential for project phasing, with installation of one membrane basin first and blending of effluent for partially reduced nutrient concentrations. This phased approach would help facilitate meeting lower permit limits if they are implemented before influent flows and loads have increased to projected levels.

Fine Screening. Membranes require much finer influent screening than conventional processes in order to protect the membranes from stringy or angular materials that could damage the material. As the existing site is congested, it was assumed that it would be possible to locate a secondary influent screening facility on top of the existing primary effluent lift station building. The primary effluent lift pumps would then pump to the fine screens and flow would continue by gravity to the bioreactors. A fine screening facility in this location would receive the lowest possible influent solids concentrations, which would result in the highest possible capacity of any chosen screen type.

Process Configuration. Membrane basins are equipped with aeration to scour and remove biomass from the membrane surface. This results in high dissolved oxygen in the membrane basin, which is returned to the bioreactor with the return activated sludge (RAS). It is undesirable to introduce high-DO RAS into the anaerobic zone located at the head of the treatment process. Therefore, the bioreactor basin and recycle stream configurations were adjusted in the model and an additional recycle stream added to utilize the return DO where it is needed and prevent it from entering zones where it would inhibit treatment. Figure 8-3 shows the modeled bioreactor process configuration. The exact volumes of each zone vary between the two bioreactor sides as existing wall locations were taken into consideration to minimize necessary structural changes.

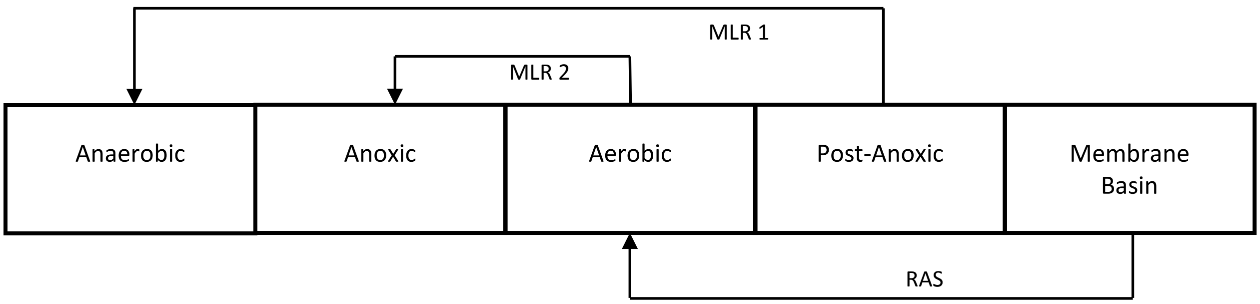

Figure 8-3. MBR Process Configuration

Impact on Effluent Nutrient Concentrations. For annual average 2037 flows and loads, effluent total nitrogen is predicted to be 3.75 mg/L, which would be very near the current limits of technology for nitrogen removal in wastewater treatment. However, modeling shows that effluent TP limits cannot be achieved by biological removal within the existing bioreactor volume with the MBR configuration. While the membranes help filter out any phosphorous associated with particulates, the resulting effluent TP concentrations would only be 1.0 to 2.0 mg/L. BioWin predicts that the anaerobic zone would need to be about twice the existing size for both bioreactor sides to achieve less than 0.3 mg/L effluent TP.

As an alternative to increasing the anaerobic zone, alum addition could be used seasonally to reduce the effluent TP concentration to levels below 0.3 mg/L, as discussed in Alternative P-3A. Another possibility would be to size the anaerobic volume large enough for sufficient biological phosphorous removal at the expense of the anoxic zone and feed a carbon source to the post anoxic zone during the nutrient removal season. Both options would require installation of chemical feed and storage equipment for both bioreactor sides. Because alum is typically less expensive than carbon sources, for this report, alum addition was included in this analysis. With alum addition, the MBR should be able to achieve the following average effluent nutrient concentrations and mass loads for the maximum month 2037 flow of 14.3 mgd:

- TN: 3.85 mg/L; 459 lb/d
- TP: 0.25 mg/L; 30 lb/d

While the nitrogen concentrations and loads would be well in compliance with the planning limits through biological processes alone, the phosphorous quantities were assumed to be achieved by using the minimum amount of alum to just achieve permit compliance.

Equipment and Structures

As mentioned above, a new fine screening facility would be constructed above the existing primary effluent lift station building. Modifications and reinforcement of the existing structure would be required to allow it to support an additional story. Five rotary fine screens would be located in this facility with four units capable of processing the 2037 peak hour flow and a fifth unit for redundancy. During normal operation, two units would be dedicated to Side 1 and two units to Side 2 with one overall spare; piping and valves would allow for use of the spare unit on either side. Also included would be three washer compactors (two duty, one spare), screw conveyor, and a dumpster.

The membranes and associated pumps and piping would be installed in one clarifier on each bioreactor side. It was also assumed that the remaining space within the clarifiers would be sufficient to house the chemical feed equipment for the membranes and for alum addition.

If this alternative is considered further, use of remaining clarifier space may be evaluated for conversion to equalization (EQ) basins. The basins would be mixed but not aerated, would provide additional post-anoxic volume and the ability to shave off peak flows. This may allow for reducing the size of the membrane system. However, placing EQ basins at this point in the process would require additional pumping from the EQ basins to the membrane facilities. An EQ analysis was not completed for this report nor was EQ included further in this analysis.

Major equipment and structures for this alternative would be as follows:

Fine Screening Facility

- Structural changes of primary effluent lift station building
- Five fine screens
- Three washer/compactors
- Screw conveyor
- Piping and valves

Each Bioreactor Side

- Structural changes of bioreactor basins
- Structural modifications of clarifier walls
- Building and membrane basin within clarifier
- Manufacturer-supplied membrane system equipment and alum addition equipment
- Piping and Valves for bioreactor modifications, membrane system, yard piping

Capital and Operational Costs

The MBR is the most expensive alternative with an estimated capital cost of $52,220,000. This includes over $10,000,000 for the membrane system with the remainder accounting for clarifier basin modifications, two membrane buildings, and the fine screen facility. Additional operational costs are equally high, largely due to required pumping for the entire effluent flow at a relatively high headloss through the membranes, and pumping of RAS at a rate of four times the influent flow. Estimated operational costs are about $375,000 per year and include the additional aeration requirement for the higher flows and loads at 2037 conditions, membrane cleaning chemicals and alum, and air scour requirement for the membrane cassettes.

Feasibility

The thorough filtration provided by membranes not only removes total phosphorous where it is associated with solids, but also metals and other compounds that have adsorbed to solids or are present as a precipitate. In addition, ultra-filtration, as provided by the membrane system evaluated for this report, also removes bacteria and viruses more effectively than UV disinfection, so that the existing UV system could probably be turned off and only kept for redundancy purposes. This reduction in operational costs was not included in the above analysis as discontinued use of the UV system would have to be approved by MDEQ and therefore is considered somewhat uncertain at this time. This level of filtration would also set up the WWTP for unrestricted water reuse in parks, cemeteries, golf courses, and other public locations. If unrestricted reuse is a goal in the future, an MBR would offer a possible solution.

However, membrane systems are expensive in and of themselves and have a number of requirements, such as fine screening, that add significant ancillary capital costs. Membrane systems also have high operating costs because all the effluent must be pumped through the membranes (high headloss) and RAS flow rates are high, requiring high volume pumping. Membrane cleaning processes include air scour, as well as year-round chemical use to remove scale and fouling from the membrane surfaces.

Hydraulically, Side 2 may be challenged as presented in Chapter 7, if the entire 2037 flow diverted to Side 2 must pass through the existing two bioreactor trains without the hydraulic relief provided by a third train. A more detailed hydraulic analysis would be prudent if this alternative is considered further.

Process modeling did not yield a configuration that would provide adequate biological nutrient removal in the existing bioreactor volume. An MBR would either need another process train or chemical addition to achieve the required phosphorous removal. As shown for Alternative P-3, an additional process train possibly with chemical addition could achieve required effluent quality without the investment into a membrane facility.

And finally, an MBR would increase the operational complexity of the plant more than any of the other alternatives discussed. It is possible that it would require additional plant staff to handle the increased operation and maintenance demands of a membrane system. As long as unrestricted water reuse is not a driving factor in the evaluation of future upgrade alternatives, an MBR facility has low feasibility for implementation.

8.5.6. Alternative P-5 – Switching to an Alternative Treatment Process

The various treatment alternatives evaluated for this Facility Plan all utilize conventional activated sludge processes in different configurations. However, over the past 20 years, advances have been made with alternative forms of activated sludge treatment, particularly with granular activated sludge. In this process, bioreactor conditions and settling are manipulated to produce larger particles within the sludge in which bacteria cluster together to form granules. These granules settle much more quickly, which allows for operation at MLSS concentrations of up to 8,000 mg/L with conventional settling in clarifiers.

Granular activated sludge would have the same advantage as the MBR, which would allow treatment of higher loads by operating with a higher MLSS concentration in the existing bioreactor volume resulting in lower effluent concentrations. In contrast to an MBR, granular activated sludge can operate with the existing clarifiers and does not require effluent filtration. It does require a separate settling column preceding the clarifiers to continuously select for the granules and maintain them in the system. Granular activated sludge also requires a side stream reactor with carbon addition and other process feed streams to prepare the granules for biological phosphorous removal as well as complete denitrification and to culture biomass for continuous augmentation to the main process. This side stream reactor would serve double duty by reducing phosphorous and ammonia in the pressate return stream from the dewatering process as well as feeding and "priming" the granules for optimal performance in the mainstream process.

Another benefit of granular activated sludge is its ability to simultaneously nitrify and denitrify (SND), while skipping the conversion to nitrate (NO_3) for about 90 percent of the nitrogen and going straight from ammonia to nitrite (NO_2) to nitrogen gas. This process is quicker and requires less air (energy) than the full conversion to nitrate. Exact energy savings were not calculated for Missoula, but based on the stoichiometry of the chemistry involved, it is estimated that granular activated sludge would require just under one third less air than conventional activated sludge for the nitrification process. Air required for $cBOD_5$ removal may also be somewhat lower as some of the $cBOD_5$ would be used as a carbon source for some of the denitrification, rather than by dissolved oxygen-consuming bacteria.

Granular sludge has been used in side stream treatment for ammonia successfully for many years. However, side stream treatment systems typically require elevated wastewater temperatures to be efficient. The patented Nereda process uses granular sludge in sequencing batch reactors without additional heat with a plant in design (2018) for the City of Whitefish; however, granular activated sludge is only beginning to be implemented at full scale for a continuous-flow mainstream process. The King County Wastewater Treatment Division has constructed a mainstream granular activated sludge facility at one of their treatment plants. The facility is scheduled to start operation in 2019.

The Missoula WWTP is a continuous flow activated sludge facility and conversion to a sequencing batch reactor would require very significant changes of basin layout and piping. As an immediate upgrade is not needed to meet effluent requirements or growing loads, the Nereda process was not explored further at this time. However, as data becomes available from the King County treatment plant and potentially others, the continuous flow granular activated sludge process may be evaluated further for potential implementation in Missoula.

In the absence of concrete design information for these alternative treatment processes, a feasibility discussion and cost estimates were not developed. It is anticipated that as information becomes available of the next decade, a more complete evaluation will be possible.

8.5.7. Alternative P-6 – Tertiary Treatment for Nutrient Polishing

Rationale

One method for achieving low effluent nutrient concentrations is tertiary treatment which polishes the effluent from the existing treatment process. As described in Alternative P-4, just expanding the existing treatment in its current configuration will not achieve the nutrient removal that will be needed when permit limits are significantly lower and influent loads are higher. Instead of modifying and expanding the bioreactor process, a tertiary treatment step can be added to specifically target the remaining nutrients and remove them. The proposed tertiary treatment would receive settled effluent from the existing treatment system after addition of alum for enhanced phosphorous removal. The tertiary process would denitrify remaining nitrate/nitrite and filter flow for better solids removal. Effluent from the tertiary system would continue to UV disinfection.

Alum would be added in the clarifier splitter boxes where mixers would provide rapid mixing of the alum with the process flow. Much of the phosphorous precipitate would settle in the clarifiers but very fine material would overflow the clarifier weirs and enter the tertiary treatment system.

Influent to the tertiary treatment system would be dosed with a carbon source, typically methanol, to provide food for the denitrifying bacteria contained in the system. Proprietary filter media provides a surface for the bacteria to attach to and remain in the system as flow passes through. Attached growth provides a means for retaining biomass without the need for settling and a recycle stream. Occasional air scour and backwashing will remove excess growth and flush solids filtered from the secondary effluent.

Tertiary treatment may be implemented in stages and used to treat only a portion of the effluent rather than implementing a larger plant upgrade. This approach may be less expensive in the short term and would facilitate adaptation of plant processes to gradually increasing influent flows and loads.

Impact on Effluent Nutrient Concentrations. Predicted effluent concentrations are based on the tertiary treatment system manufacturer's predictions and would have to be refined should this alternative be chosen for detailed evaluation. Tertiary treatment as included for this report should be able to achieve the following average effluent nutrient concentrations and mass loads for the maximum month 2037 flow of 14.3 mgd:

- TN: 5.0 mg/L; 596 lb/d
- TP: 0.25 mg/L; 30 lb/d

These concentrations and calculated loads for maximum month conditions would be just under current permit limits but would be expected to be reliably in compliance at average flows.

Equipment

Pumping. For this alternative, pumping would likely be required from at least one of the bioreactor sides to the tertiary treatment facility to accommodate the headloss experienced through the tertiary treatment.

For this report, it was assumed that pumping of the entire flow would be required and pumping facilities are included in the cost estimate.

Phosphorous Removal. As discussed above, alum dosing facilities would be required to achieve low effluent phosphorous concentrations during the summer months. The chemical feed equipment would be identical to that discussed for Alternative P-3A but for alum only. A building housing a chemical storage tank and duplex pump skid for alum would be needed, as well as alum mixers installed at the alum dosing point. The tertiary treatment system relies on this alum addition for enhanced phosphorous removal and no other treatment elements are needed.

Nitrogen Removal. The evaluated tertiary treatment system would include six media filters located in a concrete tank and equipped with air scour, underdrain, proprietary filter media, and backwash troughs. Influent would be fed to the top of the filters and flow by gravity to the bottom where it would be collected in filter effluent piping. Methanol would be the preferred carbon source for this type of treatment due to characteristics including biomass growth rates and type, which is more tolerant to higher influent dissolved oxygen concentrations than the biomass that prefers other carbon sources. Pound for pound, methanol is less expensive than many other carbon sources but, as discussed previously, methanol does require more extensive storage facilities and handling requirements due to its explosive nature. For this report, methanol was used as the carbon source of choice. The polishing filter would require blowers for air scour, backwash pumps, and pumps for return of the backwash water to the head of the plant.

The following major structures and equipment would be needed for this alternative:

- New building; 148 ft x 84 ft
- Tertiary lift pumps
- Tertiary polishing filter basin; 134 ft x 80 ft x 16.5 ft
- Two buildings for alum dosing; 20 ft x 16 ft
- Filter equipment and methanol feed system
- Blowers
- Backwash pumps
- Backwash waste pumps
- Alum storage and feed equipment

Capital and Operational Costs

Capital cost for the tertiary treatment facility is estimated to be $19,649,000 plus $662,000 for the alum dosing facilities for a total of $20,311,000. The bulk of this cost is in the filter basin, a large filter and equipment building and the filter equipment itself. Operational cost for this facility is estimated to be about $311,000 per year. This includes pumping of the entire plant flow to the filter facility, alum dosing, and the filter system itself as estimated by the manufacturer. The largest portion of this cost is generated by the methanol feed with a manufacturer-estimated annual cost of $242,000.

Feasibility

Siting of a tertiary treatment facility of the estimated size would be very difficult at the Missoula WWTP. The only location large enough to accommodate the facility would be south of the Side 2 clarifiers on the pilot poplar plantation. Siting a tertiary treatment facility here would require pumping all flow from upstream of the UV disinfection facility to the new tertiary facility. Effluent would have to be routed back by gravity flow to the UV disinfection system.

Tertiary treatment does not offer any energy savings in the process, and adds another process. Blower use is intermittent for filter backwash purposes only which requires energy. Pumping of the entire effluent would also add significant cost and operational considerations during high flow events.

Chemical addition of alum would be required to achieve the desired effluent total phosphorous concentrations; however, chemical additional alone would accomplish this with the addition of a third Side 2 process train at a lower capital cost and with better constructability. Compared to other alternatives that require pumping of the entire plant flow, this alternative has lower feasibility for implementation.

8.5.8. Alternative D-2 – New Outfall in Different Receiving Water

Current and future nutrient limits for the Missoula WWTP apply during the summer months, which coincide with irrigation practices. While expanding the poplar farm to allow for more land application is not considered feasible, there may be other ways to use the effluent for irrigation. This alternative explores discharge of all or a portion of the effluent to a local irrigation ditch. Effluent would be pumped to an irrigation ditch and used by irrigators as a supplement to existing irrigation water supply. A cursory review of local irrigation ditches identified two potential discharge locations within one-half to three miles from the WWTP, the Flynn Lowney Ditch (Hellgate Irrigation Company) and the Grass Valley French Ditch (Grass Valley Irrigation Company). The latter would be downgradient from the plant, while the former would require pumped flow. Issues to be considered include the following:

- What flow would be acceptable to the irrigation companies and what are anticipated withdrawal rates?
- Would there be water rights issues associated with diverting water from the Clark Fork River to an irrigation ditch?
- Where does the ditch eventually discharge and how would addition of the WWTP effluent to the ditch affect the water quality of the final ditch discharge to a state surface water?
- How would this solution be affected by MPDES permitting and other DEQ requirements?
- What types of effluent requirements from regulations and from irrigation companies would need to be met? Would additional treatment steps, such as tertiary filtration be required?
- Who withdraws water from the ditch and what would it take to obtain buy-in from all water users?
- Would there be a public perception issue associated with this option?
- Would pumping costs to the irrigation ditch be reasonable when compared to operating tertiary treatment or other nutrient reduction methods?

- Power cost is estimated to be about $4,000 - $5,000 per season for half of the 2037 average flow or 5.6 mgd.
- What is the feasibility of constructing a gravity main in the floodplain to reach the downgradient discharge location? Constructability, longevity, and permitting requirements would need to be considered.

At this time, there are too many uncertainties and unknowns to develop details and costs associated with this alternative. A detailed analysis, including research into these questions would be needed in order to determine if discharge of part or all of the effluent to an irrigation ditch would be a feasible option for reducing nutrient load to the Clark Fork River. While Morrison-Maierle has performed this type of analysis for other communities in Montana, it is beyond the scope of this report and this alternative was not developed further.

8.6. ALTERNATIVES FOR COMPLIANCE WITH E. COLI LIMITS AT 2037 FLOWS

This scenario presents upgrade options for the UV system that can expand the disinfection capacity of the existing system to 2037 flows without investment into a complete new system.

8.6.1. Alternatives Summary

- Alternative E-1A – Replacement of UV Lamps for Higher System Capacity
    - This alternative would use a newer lamp style that offers a higher UV dose while using the existing modules.
- Alternative E-1B – Installation of Redundant System in Spare Channel
    - This alternative would add a second system identical to the existing system.

8.6.2. Alternative E-1A – Replacement of UV Lamps for Higher System Capacity

Rationale

The capacity of the existing UV disinfection system does not meet 2037 maximum day or higher flows. Based on the manufacturer's (Trojan) information, the UV system was designed to provide treatment for flows up to 13.8 mgd. In addition, Trojan stated that the system sizing had some conservatism built in and is capable of providing reliable disinfection for flows up to about 15.5 mgd with one bank out of service. This would provide fully redundant treatment for the 2037 maximum month flows but would not do so at the 2037 maximum day flow of 17.4 mgd. In addition, it is unclear if the hydraulic capacity of the level control gates can handle flows above about 14 mgd. At flows above 14 mgd, disinfection may be incomplete as flow begins to bypass over the top of the lamps due to high headlosses even if both UV banks are turned on. As flows increase and maximum day events over 15.5 mgd become more frequent, increased disinfection capacity may be needed.

Plant staff have expressed the desire to postpone a complete UV system upgrade for as long as possible in hopes that LED-based technology will be available when an upgrade becomes necessary. The following may provide a compromise that would allow the WWTP to reliably provide disinfection at higher flows at a lower cost than a complete upgrade.

Trojan offers a newer lamp type with a higher output than the currently used lamp type which can be retrofitted into the existing system. These newer lamps would provide disinfection capacity for flows up to about 23 mgd. The retrofit would require replacement of the lamps and lamp holders; the ballasts are the same for both lamp types and do not need replacement. The newer lamp type is more expensive than the old, which increases replacement costs. However, using this lamp type in conjunction with upgrading the level control gates, as described in Alternative E-4, may allow for postponing a complete replacement of the UV system until reliable LED-based technology or other disinfection technology is available. An LED-based system would provide lower power demands and avoid creating mercury-laden waste. An alternative disinfection system to be considered may include ozonation.

Trojan also offers other upgrades for the automatic wiper and control systems that could be implemented as needed. However, for this report, only the lamp/lamp holder replacement was considered.

Equipment

As described in Chapter 7, the existing UV system consists of two UV banks, each with 24 modules and four lamps per module, for a total of 96 lamps. Lamp and lamp holder replacement would be performed on the standby bank with the duty bank in service. Plant personnel would be able to perform this work with guidance from Trojan.

Capital and Operational Costs

The cost for the replacement lamps, lamp holders, and labor by plant personnel is estimated to be $152,000. The cost for replacement of the level control gates would be $340,000. The new lamps are expected to use more energy than the existing lamps but the difference to the current system was not quantified.

Feasibility

Given a capacity of about 15.5 mgd, an upgrade to the UV system is not immediately needed. However, the high feasibility of a system upgrade offers a relatively low-cost option if complete replacement is to be postponed to allow for development of LED-based UV technology or other technology like ozonation. Challenges associated with replacement of the level control gates are discussed in Alternative E-4.

8.6.3. Alternative E-1B – Installation of Redundant System in Spare Channel

Rationale

A second option for providing increased UV disinfection and system hydraulic capacity would be installing a second system in the spare UV channel. This second system would exactly mirror the existing system, with the exception of using newer electronics and controls. The system would include its own system control center and automatic cleaning controls and would not tie into the existing control system.

Coordination of the two systems would occur via plant SCADA. This option would relieve the hydraulic headlosses experienced through the UV banks during high flows as each channel would only see half the flow. This solution would not change the headloss introduced by the level control gates. Level control gate replacement is discussed separately.

Equipment

As mentioned above, this alternative would include installation of a fully redundant system consisting of the same components as the existing system. The UV lamps would be the same as the existing ones, bringing the full design capacity to 27.4 mgd, not accounting for the conservatism built into the system. The level control gates would need to be replaced with this alternative to ensure that the control gate hydraulic capacity matches the treatment capacity.

Capital and Operational Costs

The cost for the redundant system is estimated to be $784,000. The cost for replacement of the level control gates would be $340,000. The system would use the same type of UV lamps, so power cost would be the same as the existing system, except if two, instead of one bank were operated, it would double.

Feasibility

The capital costs alone render this alternative as not feasible at this time. Since upgrading the system with the newer lamp type and the level control gates would provide adequate treatment through most of the planning period, installation of a redundant UV bank as an option to increase disinfection capacity is not recommended.

8.7. ALTERNATIVES FOR MEETING HYDRAULIC CAPACITY REQUIREMENTS

The plant currently experiences hydraulic limitations for very low flows in the Headworks and will experience hydraulic limitations when flows approach the 2037 peak hour flows. The plant influent lift pumps were originally sized to meet a low flow of approximately 4 mgd based on data available at the time and absent the ability to measure influent flow. However, at times, the plant experiences flows as low as 2 mgd for short periods of time, typically between 3:00 AM and 5:30 AM. The existing influent pumps as currently operated, pump near the shutoff point which causes cavitation and the pumps have experienced increased maintenance and shorter service life than would be expected.

In the spring of 2018, the plant experienced high flows of 20.5 mgd associated with area flooding and related I&I and residential sump pump discharge. This flow exceeds the plant design peak hour flow of 19.2 mgd as well as the 2037 maximum day of 17.4 mgd. The plant experienced overflow between the UV effluent box and the primary effluent overflow basin and all three influent pumps were operated, leaving no redundancy. The flood was measured as a 50-year event; therefore, it is prudent to identify hydraulic and equipment needs to meet these high flow conditions to allow the City to plan for implementation of improvements.

8.7.1. Alternatives Summary

- Alternative E-2 – Influent Lift Pumping Improvements
- Alternative E-3 – Improvements to Increase Overall Plant Hydraulic Capacity
  - Replacement of 36-inch effluent pipe with 60-inch pipe
  - Installation of weir gate between the UV effluent box and overflow basin
- Alternative E-4 – Replacement of UV System Level Control Gates
  - Installation of a UV system effluent weir or other level control rated for higher peak flows

8.7.2. Alternative E-2 – Influent Lift Pumping Improvements

Development of options for addressing the high and low flow capacity issues experienced with the influent lift pumps have been ongoing in a separate effort. Currently explored solutions include three options:

1. Pump replacement with pumps that handle a wider flow range
2. Piping and valve modifications allowing for a recycle stream to the influent wet well during low influent flows to prevent pump cavitation to mitigate low flow issues
3. Replacement of plug valves with gate valves with lower headloss and replacement of short radius bends with long radius bends to mitigate high capacity limitations

Option 1 would be implemented pump by pump without the need for bypass pumping. It would provide a flow range of 2,000 gpm (2.88 mgd) to 7,500 gpm (10.8 mgd) per pump. The existing pumps are rated for a low flow of 4,000 gpm and do not perform as intended for high flows. Some of the underperformance at high flows is likely due to high headloss generated by the existing plug valves. However, the same high headloss allows the existing pumps to operate at lower than intended flows. Option 3, replacement of the high-head plug valves with lower-headloss gate valves would help the existing pumps achieve higher flows but would also exacerbate the low flow issues of cavitation and pump wear. Option 2, recycling flow through the influent wet well during low flows, would provide higher flows for the pumps to mitigate low flow performance issues, but would introduce energy inefficiencies by pumping a portion of the flow twice for short durations.

Option 1 will be implemented in the near future and has been preceded with the replacement of the existing VFDs to allow for use of larger motors with the new pumps. Options 2 and 3 are still under consideration and may be considered after performance of the new pumps has been studied.

The City is planning for pump replacement within the next five years. The cost for pump replacement was estimated to be $195,000 if all pumps were replaced at the same time. Operational costs for the replacement pumps are expected to be very similar as for the existing pumps, assuming similar pump and motor efficiencies and the same water horsepower.

8.7.3. Alternative E-3 – Improvements to Increase Overall Plant Hydraulic Capacity

Description

Effluent Pipe. Replacement of the 36-inch effluent pipe from the effluent structure to the headwall with a 60-inch pipe would reduce headloss during flows exceeding 25 mgd by eight to nine inches. This difference is significant as it would avoid submerging the effluent weir and ensure accurate flow measurement during peak hour flows.

UV Influent Channel / Primary Effluent Overflow Basin. The dividing wall between the UV influent channel and the primary overflow basin required sandbagging during the 2018 high flows of up to 20 mgd to prevent secondary effluent from entering the primary overflow basin. This wall is meant to serve as an overflow from the primary effluent to the outfall during emergency flow conditions when partial bypass of the secondary treatment system is necessary. Therefore, increasing the wall height permanently would not be a desired option. Installation of a stop log or adjustable weir gate and grating for operator access would allow for secondary treatment bypass while also preventing backflow during extreme high flow events. For this report, stop logs were not further considered because they tend to allow a significant amount of seepage. During an extended high flow event, the basin could fill up and the higher water elevation may impact operation of the primary effluent wet well; therefore, installation of an adjustable weir gate was used as a solution. Additional hydraulic analysis is advised prior to gate selection to determine the required gate width for potential overflow events.

Equipment

Effluent Pipe. The replacement effluent pipe would be 60-inch PVC where buried and transitioning to concrete where exposed. Installation of a duckbill valve to prevent wildlife and fish from entering this large pipe is recommended. Headloss of these valves is minimal but would need to be explored further during design to ensure that the headloss gains for the larger pipe size are not negated by this valve.

UV Influent Channel / Primary Effluent Overflow Basin. For this report, a 6-foot slide gate with a 1-foot travel distance was used. This travel distance would allow for opening the gate to the elevation of the top of the wall and closing it to the elevation of the surrounding basin walls. The wall height to the right and left of the slide gate, as well as the fully closed gate elevation would be the same as that of the remaining basin walls. Grating and steps would be provided for operator access to the gate operators.

Capital and Operational Costs

The capital costs for replacing the effluent pipe and installing the weir gate was estimated to be $316,000.

Feasibility

Removal and replacement of the effluent pipe extending into the river bed would require bypass pumping of the entire plant effluent to a location somewhat removed from the outfall. The construction would also include excavation and placement of fill below the ordinary high water mark of the Clark Fork River. Both of these activities would have regulatory implications involving routine application for a number of permits and MDEQ approval. A visual screen of the pipe may be needed to avoid complaints from river recreationists. Implementation of this alternative would largely be driven by need and feasibility concerns would need to be resolved during the pre-design phase.

Installation of the weir gate would not require bypass pumping as the top of the overflow wall is dry during normal operations and the overflow basin can be isolated and pumped down to allow for installation of the gate on the side of the overflow basin. Implementation of this alternative would be straight forward and no other feasibility concerns exist.

8.7.4. Alternative E-4 – Replacement of UV System Level Control Gates

Rationale

The UV disinfection system water level is controlled by automatic level control gates. These gates are spring-loaded flap gates that respond to the varying head exerted by varying water levels in the channel to open more or less, controlling the flow volume through the gates. Based on manufacturer’s information, the existing gates were designed to pass 15.5 mgd; however, when the UV system was installed, the design peak hour flow of the plant was 19.2 mgd. Replacement of these level control gates is treated as a separate alternative because it may become necessary prior to upgrading the UV system as described in Alternative E-1.

Complete disinfection is only required up to maximum day flow and not during peak hour flows; however, water should not reach the ballast boxes at the top of the UV modules because leaks in the boxes could cause electrical damage. As mentioned above, during the spring of 2018 at flows of around 19 mgd, the water level reached the ballast boxes of the upstream UV bank, which indicates that the level control gates hydraulic capacity is somewhat less than 19.2 mgd. The UV system manufacturer strongly recommends taking measures to prevent flows from reaching the ballast boxes again. Plant flows should be monitored to determine the frequency and duration of peak flows that cause the channel water level to get close to the ballast boxes. If these peak flows increase in frequency and duration, planning for replacement of the level control gates with level control capable of maintaining the channel level at good operating levels through a wider range of flows should begin. If high flows are encountered that are likely to touch or overtop the ballast boxes, plant staff should consider pulling one of the UV modules from the channels to reduce headloss and protect the equipment from electrical shorts and failure.

Equipment and Structural Modifications

Trojan would provide new automatic level control (ALC) gates and accessories. Trojan has also provided information regarding the replacement of the ALC gates with larger capacity ALC gates. Replacement of the gates would require significant structural modifications to the channel. These modifications would include saw cutting of the concrete ramp supporting the existing gates to lower the ramp elevation and extending the ramp into the UV effluent box by two feet.

These changes would only be possible while bypass pumping to divert flow from the channel. In addition, for the duration of this work, an alternate source of disinfection would need to be provided, likely consisting of chlorination and dechlorination. Disinfected effluent would be discharged downstream of the effluent weir.

Capital and Operational Costs

The capital cost for this alternative is estimated to be $340,000. Most of the cost for this alternative is for bypass pumping and providing alternate disinfection during construction.

Feasibility

If hydraulic conditions necessitate gate replacement, feasibility is driven by need rather than cost or constructability considerations. However, having to provide temporary disinfection for the entire plant flow would present a challenge and likely be associated with significant cost. Bypass pumping of the entire plant flow would also add significant cost. Both would encourage postponement of the decision to upgrade the level control gates for as long as possible while monitoring the disinfection market for new technologies that may invite replacement of the entire system prior to influent flow increases that would necessitate upgrading the UV level control gates.

8.7.5. Alternative E-5 – Side 2 Primary Effluent Lift Pump Replacement

Rationale

As discussed in Chapter 7, the smaller of the three Side 2 primary effluent lift pumps significantly reduces the firm capacity of the lift station to less than current peak hour flows. Replacement of the smaller pump with one equally sized to the others would increase the firm capacity of the lift station to about 13.4 mgd and to about 17.5 mgd with all three pumps operating. These capacities could be higher if the wet well level were allowed to rise to reduce static head during higher flows with higher friction losses and it may be possible to pump the full 2037 peak hour flow, albeit without redundancy. Further planning should be initiated as flows increase and peak flows exceeding the firm capacity of the lift station become more frequent.

Equipment

Replacement of the smaller pump with a pump identical to the two larger pumps would be prudent as it would ensure seamless operation when more than one pump is operating. Pump replacement would also require replacement of the pump VFD with a larger one and potentially other electrical and control changes to accommodate the larger pump.

Capital and Operational Costs

The capital cost for this alternative is estimated to be $108,000. Operational costs would not change because plant staff have not been using the smaller pump and the new pump will be the same size as the two larger existing pumps.

Feasibility

This pump replacement has high feasibility as it offers a relatively simple measure to increase plant capacity at a pinch point.

8.8. EQUIPMENT REPLACEMENT DUE TO AGE OR PERFORMANCE ISSUES

Chapter 7 identified two pieces of equipment that are at or nearing the end of their intended service life, including one of the Side 1 primary effluent lift pumps and the DAF thickeners. Replacement of the Side 1 primary effluent lift pump would be scheduled and handled as part of routine equipment replacement when plant staff determine that the existing pump is no longer reliable and is not further discussed here. Replacement of the thickening system was examined here as multiple options exist. The DAF thickeners may be replaced with like equipment or with a different thickening system. Cost effectiveness with respect to capital and operational costs were examined, as well as effects on TWAS storage capacity and dewatering coordination with digested sludge.

As discussed in Chapter 7, the ducking scum skimmers in the bioreactors have never functioned as intended, making manual skimming necessary on a daily basis to remove scum. Replacement with a different skimming system is briefly discussed below.

8.8.1. Alternatives Summary

- Alternative E-6 – WAS Thickening Equipment Replacement
    - Alternative E-6A – Replacement with new DAF Units
        - This alternative would replace like with like without changes to the thickening process or ancillary equipment
    - Alternative E-6B – Replacement with Volute Thickeners
        - This alternative examines use of volute thickeners in place of the DAF units.
- Bioreactor Scum Skimming Discussion

8.8.2. Alternative E-6A – Replacement of DAF Thickeners with New DAF Thickeners

Rationale

Replacement of the existing DAF thickeners with two new units would be straightforward. The plant is currently only utilizing one of their two units, which would allow for phased implementation while keeping one unit operational. It is conceivable that only one new unit is purchased, while one existing unit is maintained for redundancy until plant staff determine that the older unit is no longer reliable as a backup or multiple units are needed to increase capacity. However, for the purpose of this report, replacement of both DAF units was assumed.

With this replacement, WAS and thickened WAS operations would continue as currently operated without change in piping, flow streams, and TWAS storage. While the TWAS tank does have some excess storage capacity, it mainly serves as equalization for the TWAS. At increasing sludge production rates, the solids processing capacities would have to be upgraded to accommodate larger solids streams and no change in TWAS storage is anticipated.

Equipment

Replacement of only both DAFs unit was included in this alternative. While the existing air-entraining (Edur) pumps are in good working order and have sufficient capacity for current sludge flows, they do not offer additional capacity. In order to provide a system with similar capacity to the volute thickener alternative discussed below, an additional air entraining pump was included in this alternative. The following equipment was included for this alternative:

- Two DAF thickeners
- One air-entraining (Edur) pup

Capital and Operational Costs

Purchase and installation of a rectangular DAF thickener of the type currently operated at the WWTP is estimated to cost approximately $1,104,000. Operational costs are assumed to the same as current costs.

Feasibility

Thickening through dissolved air floatation does not require addition of chemicals to aid in coagulation or the associated equipment and chemical storage. Equipment replacement of like with like is typically straightforward and does not require engineering services or MDEQ review. However, while historically reliable, DAF thickeners have been replaced in many plants with newer technology that can be less energy intensive, simpler to operate, or otherwise benefit operations at the WWTP. In addition, if future operations require a higher capacity, the existing building space would likely not have the footprint to accommodate larger units. Other thickening technology typically requires a smaller footprint and higher capacity could be provided in the same space. Given the projected growth, investing in a technology that is limited in its expansion capacity is not recommended.

8.8.3. Alternative E-6B – Replacement of DAF Thickeners with Volute Thickeners

Rationale

Replacement of the existing DAF units with a different thickening technology offers options for energy conservation, optimization of operator utilization, and potential changes in the coordination of thickening and dewatering of WAS. Volute thickeners have been used for this application for many years and are a similar technology to the plant's volute press installed in 2016 for digested and thickened waste activated sludge dewatering. The volute thickeners are fully automated and allow for unattended operation. The operation involves dosing sludge with polymer in a flocculation tank where it is mixed and then sent to the volute thickener. Considerable energy savings would be possible with a volute thickener because it does not require operation of air entrainment pumps. Power consumption of a volute thickener and polymer system combined is approximately 5 hp compared to 30 hp for the existing air entraining pumps alone. For this alternative, replacement of both DAF units with two volute thickeners was assumed.

Unlike the existing DAF units, which do not require chemicals, the volute thickener requires polymer addition to achieve proper thickening. The amount of polymer needed is difficult to estimate but for this report was based on an average use provided by the manufacturer. Costs for polymer were loosely estimated and exceed the costs for the energy savings. However, when thickened sludge is sent directly to the dewatering press, polymer is added only once, and therefore does not constitute an additional cost to current plant operations.

New volute thickeners may be installed in the same location as the existing DAF units or adjacent to the dewatering press in the Biosolids Handling Building. If installed in the DAF locations, piping, valves, pumping, TWAS storage, and blending with digested sludge would remain as currently operated. However, building modifications may be necessary to create access for chemical delivery and storage of polymer. Cost for these modifications was included in the cost estimate.

When pumping to the TWAS tank, the tank would serve as a “pass-through” unit only. Pumped WAS volumes would be higher than the currently pumped thickened WAS volumes. WAS would need to be processed by the thickening and dewatering system at about the flow rate it would be pumped to the TWAS tank because the relative equalization provided by the TWAS tank would be much smaller for a higher flow rate. Capacity of the existing pumps appears to be adequate for the higher WAS flow rate to the thickener.

If installed in the Biosolids Handling Building, one thickener would be installed as a “piggy-back” to the existing dewatering press, discharging dewatered sludge directly to the press. A second dewatering press would be installed with a second “piggy-back” thickener. This installation would require some modifications to the existing dewatering press, as well as an additional polymer system and piping and valve modifications. Digested sludge may continue to be discharged directly to the dewatering press or allowed to run through the thickener. Piping configurations would need to be fine-tuned if blending of WAS and digested sludge was desired.

The existing chemical storage room in the Biosolids Handling Building would house the new polymer feed skid. New piping would be required to route polymer to the new thickening/dewatering unit while only minor modifications to existing piping would be necessary to feed the thickener installed with the existing volute press.

Equipment

Replacement of both DAF units with two volute thickeners was assumed. For the installation in the Biosolids Handling Building, an additional dewatering press was also added. For cost estimating purposes, installation in the DAF building and the Biosolids Handing Building were compared.

- Installation in DAF building:
    - Two volute thickeners (as a “piggy-back” to the existing and one new dewatering press)
    - Two polymer feed systems

- Installation in Biosolids Handling Building:
    - Two volute thickeners (as a "piggy-back" to the existing and one new dewatering press)
    - One polymer feed system
    - One dewatering press

Capital and Operational Costs

The capital cost for this system is estimated to be $931,000 if installed in the space of the existing DAF unit and $1,890,000 if installed in the Biosolids Handling Building. The latter is more expensive because it also includes a second dewatering press and associated additional electrical and controls cost. However, there are savings associated with the latter option for being able to tie into the existing polymer feed system, rather than having to modify the building near the DAF units to create access for chemical delivery and storage. Power costs would be lower for both options by about $13,000 per year when compared to the DAF operation. Installation in the DAF building may add additional polymer cost up to $25,000 per year for potentially having to dose polymer twice. Installation in the Biosolids Handling Building would not introduce significant additional polymer cost as polymer only has to be dosed once into the thickening/dewatering system. Exact amounts will need to be determined be testing once the system is installed.

For the combined thickening/dewatering installation in the Biosolids Handling Building, the TWAS tank may be bypassed and blowers turned off, reducing current operational costs by the energy required to operate the blowers. This amount was not quantified as it is uncertain at this time if complete bypass of the TWAS tank will ultimately be possible or desired.

Feasibility

Volute thickeners have a much smaller footprint than the existing DAF basins, easily allowing for installation of a thickener, polymer feed system, and chemical storage within the space of one DAF unit. However, building modifications may be necessary to provide access for chemical tote delivery. Installation of the volute thickener in the Biosolids Handling Building would allow for use of the same polymer as currently used for the dewatering press and take advantage of the existing chemical storage area and access. Installation of a second dewatering press would provide complete redundancy by volute press, rather than having to operate the centrifuge. The volute press offers significant benefits over the centrifuge with respect to energy consumption, balancing return streams to the process, reduced operator time, and increased operator comfort and safety. In addition, polymer use would be more efficient for the combined thickener/dewatering installation in the Biosolids Handling Building because sludge would only need to be dosed once.

If routine operations were to thicken and dewater WAS at the rate it is generated, the operating level in the TWAS tank could be minimized, requiring less blower power and conserving energy. With the combined thickening/dewatering installation in the Biosolids Handling Building, the TWAS tank may be bypassed completely, conserving blower power in its entirety. The low operational requirements combined with operator familiarity with this type of equipment would ease the transition from the DAF thickeners. Overall, the volute thickener has higher feasibility for implementation than replacement with a new DAF thickener.

8.8.4. Bioreactor Scum Skimming Discussion

As discussed in Chapter 7, the ducking skimmers in the Side 2 bioreactors have never quite functioned as intended and bioreactor scum has been allowed to flow to the clarifiers. According to plant staff, the skimming mechanism could be repaired, which would require a crane to lift the skimmers up, adjust skimmer height, replace the bushings, and re-install them. This would allow for wasting of bioreactor scum before it flows into the bioreactor effluent channels and to the clarifiers. Options for alternative bioreactor skimming equipment were pursued. However, a review of scum skimming equipment for bioreactor basins yielded little useful information. Ducking skimmers are still sold in the wastewater market. However, a number of plants in Montana have been at one time equipped with ducking skimmers that did not perform as advertised and were subsequently either removed or still require manual assistance and operator ingenuity to make them work. Water spray is used in some plants to move scum through the system but in most cases, the scum is simply allowed to leave the bioreactor with the process flow to the clarifiers. Innovative or feasible solutions that would improve bioreactor scum removal at the Missoula WWTP were not identified.

The plant reportedly does not experience foaming events and since its upgrade in 2004, the inability to remove bioreactor scum has not negatively affected the process. Given the successful operation without bioreactor scum skimming, another option would be to remove the skimmers completely and allow scum to flow freely to the bioreactor effluent channel and the clarifiers.

Another consideration is the disposal of the Side 2 bioreactor scum. Scum would be collected in scum pits and flow to the scum pumps. These pumps are plumbed into the Side 2 WAS piping, which includes a high spot along its route to the WAS thickening equipment. As scum generally contains foamy materials, it was found that pumping scum through this line causes air lock at the high spot, requiring days of flushing with WAS before proper WAS flow is re-established. Should the Side 2 bioreactor skimming equipment be repaired or replaced, consideration would need to be given to the installation of a dedicated scum line to the destination of choice. At this time, no recommendation for repair, replacement, or removal of the Side 2 ducking scum skimmers is being made. Plant staff preferences will need to be considered for this decision.

8.9. ALTERNATIVE FEASIBILITY SUMMARY

Table 8-5 summarizes the discussed alternatives with respect to cost, estimated effluent quality, and other feasibility considerations. Priority ranks were assigned to those alternatives with recommended implementation within the next five to ten years. A low priority would place implementation of an alternative more than seven years into the future, while a high priority would place it within the next five years. All alternatives are subject to continued evaluation with respect to actual changes in plant flows and loads and ultimately plant staff will need to decide when implementation of an alternative will be realized. The priority ranking in Table 8-5 simply provides guidance for considering the urgency of potential upgrades.

Table 8-5: Alternative Feasibility Summary

Alternative	Capital Cost	Operational Cost [1]	Estimated Effluent Nutrients [2] (mg/L / lb/d)	Construct-ability	Feasibility	Priority [3]
Alternatives for Compliance with Current Nutrient Limits at 2037 Flows and Loads						
Alternative P-1 - Operational Changes	$256,000	$0	TN: 6.54 / 780 TP: 0.52 / 62	Very High	Very High	Low
Alternative P-2 - Side Stream Treatment for Nutrients	$7,470,000	$124,000	TN: 9.9 / 1,181 TP: 0.42 / 50	High	Moderate	--
Alternative D-1 - Poplar Farm Expansion	Not determined	$2,000-$3000	See Table 8-4 [4]	Very Low	Low	--
Alternatives for Compliance with Lower Nutrient Limits at 2037 Flows and Loads						
Alternative P-3 - Change in Process Config. and Addition of Process Train and Clarifier	$11,777,000	$86,000	TN: 5.23 / 624 TP: 0.27 / 32	High	Very High	--
Alternative P-3A - Chemical Addition for Alternative P-3	$933,000	$93,000	TN: 4.75 / 566 TP: 0.23 / 27	High	High	--
Alternative P-3B - Tertiary Filtration for Alternative P-3	$7,163,000	$61,000	TN: n/a TP: 0.25 / 30	Low	Moderate	--
Alternative P-4 - MBR in Existing Basins with Change in Process Configuration	$52,220,000	$375,000	TN: 3.85 / 459 TP: 0.25 / 30	Moderate	Low	--
Alternative P-5 - Switching to an Alternative Treatment Process	Not determined	Not determined	Not determined	Not determined	Moderate	--
Alternative P-6 - Tertiary Treatment for Nutrient Polishing	$20,311,000	$311,000	TN: 5.0 / 596 TP: 0.25 / 30	Low	Low	--
Alternative D-2 - New Outfall in Different Receiving Water	Not determined	$4,000-$5,000	Not determined	Not evaluated	Not evaluated	--

Alternative	Capital Cost	Operational Cost [1]	Estimated Effluent Nutrients [2] (mg/L / lb/d)	Construct-ability	Feasibility	Priority [3]
Alternatives for Compliance with E. coli Limits at 2037 Flows						
Alternative E-1A - Replacement of UV Lamps for Higher System Capacity	$152,000	Not determined	--	Very High	Moderate	Moderate
Alternative E-1B – Installation of Redundant System in Spare Channel	$784,000	Not determined	--	Very High	Moderate	discarded
Alternatives for Meeting Plant Hydraulic Capacity Requirements						
Alternative E-2 - Influent Lift Pumping Improvements	$195,000	$0	--	High	Very High	High
Alternative E-3 - Overall Plant Hydraulic Capacity Improvements	$316,000	$0	--	Moderate	Moderate	Moderate
Alternative E-4 - Replacement of UV System Level Control Gates	$340,000	$0	--	Low	Moderate	Moderate
Alternative E-5 – Side 2 Primary Effluent Lift Pump Replacement	$108,000	$0	--	Very High	High	High
Equipment Replacement Due to Age or Performance Issues						
Alternative E-6A - WAS Thickening Replacement with New DAF Units	$449,000	$0	--	High	Moderate	discarded
Alternative E-6B - WAS Thickening Replacement with Volute Thickener [5]	$453,000 $418,000	$12,000	--	High Very High	Very High	High

1. Listed operational costs do not include existing chemical, power, and other energy costs. High level estimates were developed for a cursory comparison of O&M costs.
2. Based on 2037 maximum month flow of 14.3 mgd. Diversion to poplar farm is not accounted for in the listed nutrient mass quantities.
3. Only alternatives with recommended implementation in the next 5-10 years were assigned a priority rank.
4. Effluent nutrient load reductions shown in Table 8-4 would subtract from the nutrient loads listed in this Table 8-5 if poplar irrigation was combined with the evaluated alternatives.
5. Listings for installation in DAF location on top; installation as piggy-back to volute press on the bottom.

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8.10. WWTP PLANNING TRIGGER CONDITIONS

Evaluation of the existing plant shows that its equipment and process capacities are adequate to treat current and near future influent flows and loads to the current effluent requirements. Near future average influent flows would be approximately 9.2 mgd or about half of the projected increase for 2037. As previously addressed, permit limits are not expected to change for the next two permit cycles, at which time, flows are projected to still be below the design capacity of the existing plant. BioWin process modeling output predicts that the existing plant will begin to approach its treatment capacity for meeting the current permit limits for total nitrogen and total phosphorous when average influent flows reach about 9.2 mgd. With the diversion of irrigation water to the poplar farm, overall permit compliance is predicted to be achievable for those conditions. However, planning for expansion, upgrades, or alternative permit compliance should commence at that time to stay ahead of changes in permitting and increases in flows and loads beyond the existing plant capacity. Certain events or conditions should trigger this planning as follows:

- Trigger Event 1: Listing of the Clark Fork River on the TMDL schedule published by MDEQ:
    - Listing is anticipated five to ten years from now.
    - Communication with MDEQ should start as soon as the re-development of the Clark Fork TMDL is scheduled to ensure the City is well-informed of the progress and potential outcomes of the TMDL development and its implications for future MPDES permits.
    - As soon as anticipated outcomes become apparent, their implications for plant treatment capacity should be evaluated and appropriate options identified.
    - The current version of Circular DEQ-12B must be examined at that time to determine any additional considerations for the Missoula WWTP.
    - As discussed in more detail in Chapter 2, river modeling and nutrient trading are options that may be pursued at that time.

- Trigger Event 2: Increase of average influent flows to about 9.2 mgd and/or increase of average influent loading halfway to the projected 2037 loads and/or the year 2027:
    - Increases in flow and load should be compared to the predictions in this report and updated if necessary.
    - Hydraulic capacity should be re-evaluated, starting with the locations identified in this report, and planning for improvements should begin.
    - Treatment capacity should be evaluated and planning for potential expansion alternatives should start. Treatment technology and processes available at that time should be reviewed and any new treatment or disposal options should be included in an alternatives analysis.

Both trigger conditions must be considered in concert as both may occur close to each other and any response would likely need to address increasing flows and loads as well as lower effluent limits for nutrients. These trigger events were incorporated into the preliminary implementation schedule presented at the end of this Chapter. In addition, an update to the Facility Plan is advisable every ten years, which may coincide with at least one of these trigger events and allow for an overall re-evaluation of plant capacities, future flow and load projections, and available technology and options at that time.

8.11. RESILIENCY CONSIDERATIONS

Over the past decade, "resiliency" has become a new emphasis among utilities across the United States. It refers to a utility's preparedness for and ability to cope with extreme events including droughts, floods, earthquakes, forest fires, and software hackers among others. Extreme weather events, often linked to climate change, have been growing in frequency, magnitude, and duration and increasingly require changes in planning approaches for future investments. For Missoula, the recent high flow event of 2018 is likely to be counted among these more frequent extreme events and suggests that it would be prudent to plan for higher peak flows of longer durations than previously encountered even if annual average flows are not increasing at a corresponding rate.

Minimizing inflow and infiltration (I&I) can go a long way to make a treatment plant more resilient to extreme rain and snow melt events. If inflow as caused by rain events creates peak flows at the plant, City planning may need to include measures that would slow runoff or retain it to avoid slug flows into the collection system. Measures taken by other communities include encouraging home owners to build rain gardens for roof runoff, placing pervious pavement in parking lots, or designing parks and other public spaces for maximum storm water retention. These measures are consistent with the city of Missoula's Storm Water Division best management practices to reduce both storm water flow and the pollutants it carries to the aquifer, streams and the Clark Fork River. Other measures include sealing manhole lids, identifying and eliminating roof gutter connections to the sewer, and continuous public education about not connecting basement sump pumps to the sewer system.

Forest fires are not limited to the forests as recent experience in Colorado and California has shown. When forest fires encroach on communities, power supply can be compromised. Connection of a plant to more than one substation and presence of backup generators can increase resiliency to disasters affecting power supply. The co-generation facility at the Missoula WWTP is a significant asset in this context as it provides an independent if relatively small source of power.

Threats from hackers either trying to steal intellectual property or sabotaging a system have increasingly been publicized in industry and commerce. It may only be a question of time before systems used to operate water and wastewater systems become a target. Having a response strategy to a compromised plant control system would increase plant resiliency to a cyber threat.

Many cities and states in the US have included resiliency planning into their routine operations and literature about their efforts abounds. Water and Environment Federation (wef.org) offers access to a number of articles on resiliency topics and EPA (https://www.epa.gov/waterresilience) offers guidance to assessing risk, building resilience, and develop emergency response plans. The Safe Drinking Water Act and American Water Infrastructure Act require water systems serving 3,300 or more people to prepare an emergency response plan and a resilience plan, respectively. Guidelines for these plans may be used to prepare similar plans as applicable to the wastewater system.

8.12. PRIORITY LIST AND SUGGESTED IMPLEMENTATION SCHEDULE

The following presents a suggested implementation/action schedule. As stated above, almost all alternatives recommended for implementation within the next ten years are subject to evaluation relative to actual plant flows and plant staff will make the final decisions as to scheduling alternative implementation.

Ongoing	• Monitor the Montana 303(d) list for any new impairment listings for the Clark Fork River
2020-2024	• Replacement/modification of equipment as scheduled on the City of Missoula CIP
2022	• Check the MDEQ schedule for TMDL development to see if the Middle Clark Fork is included for development of new nutrient TMDLs; call DEQ staff to confirm; repeat 5 years later if not listed

Trigger Event 1: MDEQ listing of Clark Fork for Nutrient TMDL development	Trigger Event 2: Annual average flows are nearing 9.2 mgd or 2027	Trigger Event 1 and/or 2 or 2027
• Establish ongoing communication with appropriate MDEQ personnel to be well-informed on progress and permit implications of new TMDLs • Check on Circular DEQ-12B and current Variance nutrient limits • Evaluate the value of a river study for showing impact of WWTP influent on river quality to be used in application for individual Variance	• Evaluate the plant's nutrient removal performance • Re-evaluate projected growth rates and associated rates of increase in plant flow and load	• Begin planning for plant upgrades to ensure continued adequate nutrient removal capacity • Select a nutrient treatment upgrade strategy and initiate the funding/pre-design/ design process

APPENDIX 8-1

WASTEWATER TREATMENT PLANT ALTERNATIVES COST ESTIMATES

MISSOULA WASTEWATER FACILITY PLAN - WASTEWATER TREATMENT PLANT
Probable Capital Cost for Evaluated Alternatives
April 2019

Alternative P-1 - Operational Changes

Item		Cost
Buildings		--
Basins and Tanks		--
Equipment	$	121,000
Piping and Valves		--
Bypass Pumping		--
Dewatering		--
Electrical	$	10,000
Instrumentation & Controls	$	3,000
Subtotal	**$**	**134,000**
General Conditions	$	21,000
Contingency[1]	$	101,000
Total	**$**	**256,000**

Alternative P-2 - Side Stream Treatment for Nutrients

Item		Cost
Buildings	$	288,000
Basins and Tanks	$	114,000
Equipment	$	3,112,000
Piping and Valves	$	50,000
Bypass Pumping	$	8,000
Dewatering	$	5,000
Electrical	$	287,000
Instrumentation & Controls	$	72,000
Subtotal	**$**	**3,936,000**
General Conditions	$	591,000
Contingency[1]	$	2,943,000
Total	**$**	**7,470,000**

Alternative P-3 - Change in Process Configuration and Addition of Process Train and Clarifier

Item		Cost
Buildings	$	77,000
Basins and Tanks	$	3,164,000
Equipment	$	1,246,000
Piping and Valves	$	561,000
Bypass Pumping	$	75,000
Dewatering	$	250,000
Electrical	$	430,000
Instrumentation & Controls	$	403,000
Subtotal	**$**	**6,206,000**
General Conditions	$	931,000
Contingency[1]	$	4,640,000
Total	**$**	**11,777,000**

Alternative P-3A - Alternative P-3 Plus Chemical Addition

Item		Cost
Buildings	$	344,000
Basins and Tanks		--
Equipment	$	69,000
Piping and Valves	$	25,000
Bypass Pumping		--
Dewatering		--
Electrical	$	35,000
Instrumentation & Controls	$	18,000
Subtotal	**$**	**491,000**
General Conditions	$	74,000
Contingency[1]	$	368,000
Total*	**$**	**933,000**

*For alum only: $662,000

Alternative P-3B - Alternative P-3 Plus Tertiary Filtration

Item		Cost
Buildings	$	345,000
Basins and Tanks		
Equipment	$	1,840,000
Piping and Valves	$	673,000
Bypass Pumping*	$	150,000
Dewatering	$	50,000
Electrical	$	245,000
Instrumentation & Controls	$	123,000
Subtotal	**$**	**3,426,000**
General Conditions	$	514,000
Contingency[1]	$	2,561,000
Total**	**$**	**6,501,000**

*Includes alternate disinfection; **Does not include alum dosing facility

Alternative P-4 - MBR in Existing Basins with Change in Process Configuration

Item		Cost
Buildings	$	7,517,000
Basins and Tanks	$	3,086,000
Equipment	$	11,823,000
Piping and Valves	$	1,895,000
Bypass Pumping	$	50,000
Dewatering	$	200,000
Electrical	$	1,966,000
Instrumentation & Controls	$	983,000
Subtotal	**$**	**27,520,000**
General Conditions	$	4,128,000
Contingency[1]	$	20,572,000
Total	**$**	**52,220,000**

1. This is a Class 5 cost estimate and a 65% contingency was applied to account for unknown future bidding climates, changes in material costs, details not included, and other unknowns. Costs are expressed in 2019 dollars.
2. This is a Class 3 cost estimate and a 25% contingency was applied to account for unknown future bidding climates, changes in material costs, details not included, and other unknowns. Costs are expressed in 2019 dollars.

MISSOULA WASTEWATER FACILITY PLAN - WASTEWATER TREATMENT PLANT
Probable Capital Cost for Evaluated Alternatives
April 2019

Alternative P-6 - Tertiary Treatment for Nutrient Polishing

Item		Cost
Buildings	$	3,268,000
Basins and Tanks	$	2,680,000
Equipment	$	2,795,000
Piping and Valves	$	674,000
Bypass Pumping	$	120,000
Dewatering	$	50,000
Electrical	$	575,000
Instrumentation & Controls	$	192,000
Subtotal	**$**	**10,354,000**
General Conditions	$	1,554,000
Contingency[1]	$	7,741,000
Total*	**$**	**19,649,000**

*Does not include alum dosing facility

Alternative E-1A - Replacement of UV Lamps for Higher System Capacity

Item		Cost
Buildings		--
Basins and Tanks		--
Equipment	$	105,000
Piping and Valves		--
Bypass Pumping		--
Dewatering		--
Electrical		--
Instrumentation & Controls		--
Subtotal	**$**	**105,000**
General Conditions	$	16,000
Contingency[2]	$	31,000
Total	**$**	**152,000**

Alternative E-1B - Installation of Redundant System in Spare Channel

Item		Cost
Buildings		--
Basins and Tanks		--
Equipment	$	521,000
Piping and Valves		--
Bypass Pumping		--
Dewatering		--
Electrical	$	16,000
Instrumentation & Controls	$	8,000
Subtotal	**$**	**545,000**
General Conditions	$	82,000
Contingency[2]	$	157,000
Total	**$**	**784,000**

Alternative E-2 - Influent Lift Pumping Improvements

Item		Cost
Buildings		--
Basins and Tanks		--
Equipment*	$	128,000
Piping and Valves		--
Bypass Pumping		--
Dewatering		--
Electrical	$	5,000
Instrumentation & Controls	$	2,000
Subtotal	**$**	**135,000**
General Conditions	$	21,000
Contingency[2]	$	39,000
Total	**$**	**195,000**

*Pump and motor replacement only

Alternative E-3 - Overal Plant Hydraulic Capacity Improvements

Item		Cost
Buildings		--
Basins and Tanks		--
Equipment	$	41,000
Piping and Valves	$	35,000
Bypass Pumping	$	40,000
Dewatering	$	50,000
Electrical		--
Instrumentation & Controls		--
Subtotal	**$**	**166,000**
General Conditions	$	25,000
Contingency[1]	$	125,000
Total*	**$**	**316,000**

*For gate, grating, stairs only: $80,000.

Alternative E-4 - Replacement of UV System Level Control Gates

Item		Cost
Buildings		
Basins and Tanks	$	25,000
Equipment	$	34,000
Piping and Valves		--
Bypass Pumping*	$	150,000
Dewatering		--
Electrical		--
Instrumentation & Controls		--
Subtotal	**$**	**179,000**
General Conditions	$	27,000
Contingency[1]	$	134,000
Total	**$**	**340,000**

*Includes alternate disinfection

1. This is a Class 5 cost estimate and a 65% contingency was applied to account for unknown future bidding climates, changes in material costs, details not included, and other unknowns. Costs are expressed in 2019 dollars.
2. This is a Class 3 cost estimate and a 25% contingency was applied to account for unknown future bidding climates, changes in material costs, details not included, and other unknowns. Costs are expressed in 2019 dollars.

MISSOULA WASTEWATER FACILITY PLAN - WASTEWATER TREATMENT PLANT
Probable Capital Cost for Evaluated Alternatives
April 2019

Alternative E-5 - Side 2 Primary Effluent Lift Pump Replacement

Item		Cost
Buildings		--
Basins and Tanks		--
Equipment	$	52,000
Piping and Valves		--
Bypass Pumping		--
Dewatering		--
Electrical	$	20,000
Instrumentation & Controls	$	2,000
Subtotal	**$**	**74,000**
General Conditions	$	12,000
Contingency[2]	$	22,000
Total	**$**	**108,000**

Alternative E-6A - WAS Thickening Replacement with New DAF Units

Item		Cost
Buildings		--
Basins and Tanks		--
Equipment	$	230,000
Piping and Valves	$	6,000
Bypass Pumping		--
Dewatering		--
Electrical		--
Instrumentation & Controls		--
Subtotal	**$**	**236,000**
General Conditions	$	36,000
Contingency[1]	$	177,000
Total	**$**	**449,000**

Alternative E-6B1 - WAS Thickening Replacement with Volute Thickener in DAF Bldg

Item		Cost
Buildings		--
Basins and Tanks		--
Equipment	$	220,000
Piping and Valves	$	18,000
Bypass Pumping		--
Dewatering		--
Electrical		--
Instrumentation & Controls		--
Subtotal	**$**	**238,000**
General Conditions	$	36,000
Contingency[1]	$	179,000
Total	**$**	**453,000**

Alternative E-6B2 - WAS Thickening Replacement with Volute Thickener in Solids Bldg

Item		Cost
Buildings		--
Basins and Tanks		--
Equipment	$	202,400
Piping and Valves	$	18,000
Bypass Pumping		--
Dewatering		--
Electrical		--
Instrumentation & Controls		--
Subtotal	**$**	**220,000**
General Conditions	$	33,000
Contingency[1]	$	165,000
Total	**$**	**418,000**

1. This is a Class 5 cost estimate and a 65% contingency was applied to account for unknown future bidding climates, changes in material costs, details not included, and other unknowns. Costs are expressed in 2019 dollars.
2. This is a Class 3 cost estimate and a 25% contingency was applied to account for unknown future bidding climates, changes in material costs, details not included, and other unknowns. Costs are expressed in 2019 dollars.

WASTEWATER FACILITY PLAN

CHAPTER 9 - SUMMARY OF RECOMMENDATIONS AND CAPITAL IMPROVEMENTS PLAN

Morrison Maierle

engineers • surveyors • planners • scientists

CHAPTER 9 SUMMARY OF RECOMMENDATIONS AND CAPITAL IMPROVEMENTS PLAN

9.1. COLLECTION SYSTEM RECOMMENDATIONS

Detailed recommendations for the collection system are presented in Chapter 6. The collection system recommendations were developed based on the modeling results and input from City staff and include piping and lift station improvements. Collection system piping improvements identified for near-term implementation include sections of inadequate slope and/or diameter that showed potential for surcharging under existing or near-term conditions.

Collection system piping recommendations, developed with input from City staff, also include implementation of an annual program to replace or rehabilitate aging sewer mains starting with 0.5 percent of the total length of the system per year, to be increased gradually until attaining at least 1.0 percent of total length per year. This approach results in sewer main renewal on a 100-year schedule. Annual replacement of aging mains is an essential component of a robust sewer utility and contributes to overall reliability, decreases occurrence of reactive and unbudgeted main replacement, and proactively accommodates improvements in capacity where needed. The annual replacement program will ultimately help offset current utility costs associated with emergency repairs. The cost of 0.5 percent per year of the total length of the system represents approximately 44 percent of the collection system CIP and 27 percent of the total CIP expenditure over five years.

9.2. WWTP RECOMMENDATIONS

Detailed treatment system recommendations are presented in Chapter 8 and include only a few alternatives recommended for implementation within the next five years. Four of these alternatives are for replacement of aging, undersized, or underperforming equipment. In addition, three alternatives address hydraulic pinch points in the plant that were problematic during the high flows of the spring of 2018 or were identified by hydraulic calculations. Implementation of the hydraulic mitigation alternatives will depend on the frequency of high flows capable of causing the described problems. Implementation within the next five years would help ensure that future high flow events do not cause the problems encountered during the spring of 2018 and prevent issues predicted by the hydraulic calculations.

Since the future regulatory landscape is uncertain and the facility is capable of meeting all current permit limits, no treatment improvement alternatives are recommended in the near-term. Rather, as discussed in Chapter 2, it is recommended that developments within the MDEQ regarding revisions to the Middle and Lower Clark Fork TMDL and the Nutrient Standard Variance process be closely monitored while keeping track of emerging technologies for nutrient removal as they continue to develop. . A renewed analysis evaluating responses to these developments will be needed eventually but is not expected to become relevant within the next seven to ten years.

9.3. CAPITAL IMPROVEMENTS PLAN

Following development of the recommendations, the City prepared a 5-Year Capital Improvements Plan which integrates the higher priority, near-term recommendations with the City's current priorities.

The Capital Improvements Plan presented in this chapter includes recommendations that were identified in this Facility Plan to have high priority, resulting in a recommendation to be implemented within the next five years. City staff used these recommendations to develop solutions that address identified deficiencies and dovetail with overall City planning and development. Table 9-1 lists the resulting improvements and estimated costs. Improvements recommended by the Facility Plan are shown in bold blue font. Note that the costs shown for the improvements recommended by the Facility Plan were developed based on the same methodology as others presented herein and represent Class 3 to 5 cost estimates with a large margin of error. In addition, a cost projection factor of three percent per year was applied to costs for years 2021 through 2024 to account for inflation.

Table 9-1: Capital Improvements Plan, 2020-2024

	Capital Improvements	Total	FY20 [2]		FY21		FY22		FY23		FY24	
			CIP	Sewer Dvlpmt Fund	CIP	Sewer Dvlpmt Fund	CIP	Sewer Dvlpmt Fund	CIP	Sewer Dvlpmt Fund	CIP	Sewer Dvlpmt Fund
Wastewater Treatment Plant	Wastewater Lab Expansion	$295,000		$295,000								
	Influent Pump Replacement	$204,545	$100,000		$51,500		$53,045					
	Wastewater Facility Roof Replacement	$150,000	$150,000									
	UV Lamp Upgrade	$156,560			$156,560							
	Primary Effluent Overflow Basin Adjustable Weir	$82,400			$82,400							
	Atlas Copco Compressor Replacement	$41,200			$41,200							
	Side 2 Primary Effluent Lift Pump	$114,577					$114,577					
	UV Level Control Gates	$360,706					$360,706					
	Miscellaneous Improvements	$1,035,071	$269,502		$231,750		$206,876		$169,373		$157,571	
	60" Effluent Pipe	$257,884							$257,884			
	Thickening and Solids Handling Upgrade	$2,127,212									$2,127,212	
	Treatment Plant Total	**$4,825,155**	**$519,502**	**$295,000**	**$563,410**		**$735,204**		**$427,256**		**$2,284,783**	
Garden City Compost	Trommel Screen	$355,000	$355,000									
	Replace #350 Cat Loader (not leased)	$248,000	$248,000									
	Miscellaneous Improvements	$90,300	$80,000		$10,300							
	Garden City Compost Phase 2	$4,713,369							$458,945		$3,545,353	$709,071
	Garden City Compost Total	**$5,406,669**	**$683,000**		**$10,300**				**$458,945**		**$3,545,353**	**$709,071**
Collection System	***Collection System Immediate Priorities***											
	University FM Replacement	$284,000		$284,000								
	DJA Building Sewer Extension (W. Broadway & Maple)	$336,000		$336,000								
	Reserve St LS Replacement	$1,038,000	$519,000	$519,000								
	Grant Cr LS Improvements	$488,230	$34,000		$131,325	$322,905						
	Momont #2 LS Replacement	$638,178			$51,500		$164,461	$422,217				
	STEP Decommissioning Projects											
	Southpointe/Marias STEP Decommissioning	$180,860	$180,860									
	Lameroux Ln & Birdie Ct STEP Decommissioning	$306,157			$306,157							
	Maloney Ranch/Bigfork Rd STEP Decommissioning	$239,395							$239,395			
	DJ Drive/Linda Vista Blvd STEP Decommissioning	$282,836					$282,836					
	Longer Term Priorities											
	Fort Missoula Lift Station Rehabilitation	$193,084					$193,084					
	Upstream of Momont #1 LS Gravity Main Upsizing	$1,248,910									$424,629	$824,280
	East Broadway LS Force Main Extension	$1,174,900							$399,466	$775,434		
	River Front Triangle Main Upsize	$175,100				$175,100						
	South Ave Gravity Main Extension	$412,000				$412,000						
	Infiltration/Inflow Study	$103,000			$103,000							
	Annual Needs											
	Sewer Main Rehabilitation Program [3]	$6,727,737	$1,267,200		$1,305,216		$1,344,372		$1,384,704		$1,426,245	
	Miscellaneous System Improvements	$1,327,284	$250,000		$257,500		$265,225		$273,182		$281,377	
	Collection System Total	**$15,155,670**	**$2,251,060**	**$1,139,000**	**$2,154,698**	**$910,005**	**$2,249,978**	**$422,217**	**$2,296,746**	**$775,434**	**$2,132,251**	**$824,280**
Wastewater/Compost Utility Total		**$25,206,982**	**$3,453,562**	**$1,434,000**	**$4,675,108**	**$910,005**	**$2,985,182**	**$422,217**	**$3,182,948**	**$775,434**	**$5,835,175**	**$1,533,351**
	CIP Total / Sewer Development Fund Total	$20,131,975 / $ 5,075,007										
	Cost Projection Factor (3% / year) [1]		1.000		1.030		1.061		1.093		1.126	

[1] Estimates of probable cost in 2019 dollars. Costs presented include the cost projection factor of 3% inflation per year.
[2] Fiscal year 2020 from July 2019 - June 2020
[3] Assumes 0.5% of total collection system (1.6 miles of gravity main) replacement per year at a cost of $150 per lineal foot.

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