AVIMOR™
Water Reclamation Company

Master Wastewater Study 2022

PROFESSIONAL ENGINEER
REGISTERED
12246
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STATE OF IDAHO
KASEY K. KETTERLING

Prepared by:
T-O ENGINEERS
332 N. Broadmore Way
Nampa, Idaho 83687
(208) 442-6300

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Introduction

1.1 Background

This Master Wastewater Study was prepared to support the development of the Avimor Planned Community (APC or Avimor) that is being developed by Avimor Development LLC as shown in Figure 1-1. The purpose of this Master Plan is to satisfy the requirements of IDAPA 58.01.16 410.01.

Figure 1-1 Avimor Development

The APC is located at the intersection of Ada County, Boise County, and Gem County between the City of Eagle and Horseshoe Bend. This area was previously used for dryland grazing prior to being acquired for the planned community. Avimor is being developed to include a mixture of homes, trails, parks, community facilities, commercial businesses, and retail stores. The land owned by Avimor is approximately 19,590 ± acres as shown in Figure 1-2.

In 2008, Avimor built the Avimor Water Reclamation Facility (AWRF). The AWRF and the associated wastewater collection system and reuse system are owned and operated by the Avimor Water Reclamation Company LLC. The AWRF is able to produce Class B reclamation water and land-apply when permitted by the Idaho Department of Environmental Quality (DEQ).

Reclaimed water is currently being used during the growing season to irrigate landscaping, parks, and vegetation along roadways. During the non-growing season, the reclaimed water is surface-applied at rapid infiltration (RI) basins for final treatment prior to discharging to the groundwater. The current reuse system includes five RI basins, a water storage tank, and the pressure irrigation system. In addition, when permitted by DEQ, the facility may directly discharge to surface water in the Spring Valley Creek through a Idaho Pollutant Discharge Elimination System (IPDES) permit. The facility is currently reapplying for a new discharge permit as the previous permit expired

May 2, 2021. The facility does not currently discharge to surface water however this option may be implemented as the community develops.

Current residential and commercial development is located at the southeast side of the Avimor property boundary. As of December 2021, there were 664 occupied residential homes, a village community center, commercial offices, a brewery, and a gas station. Expansion and development of the community is an ongoing process based upon consumer demand and planning decisions made by Avimor Development LLC.

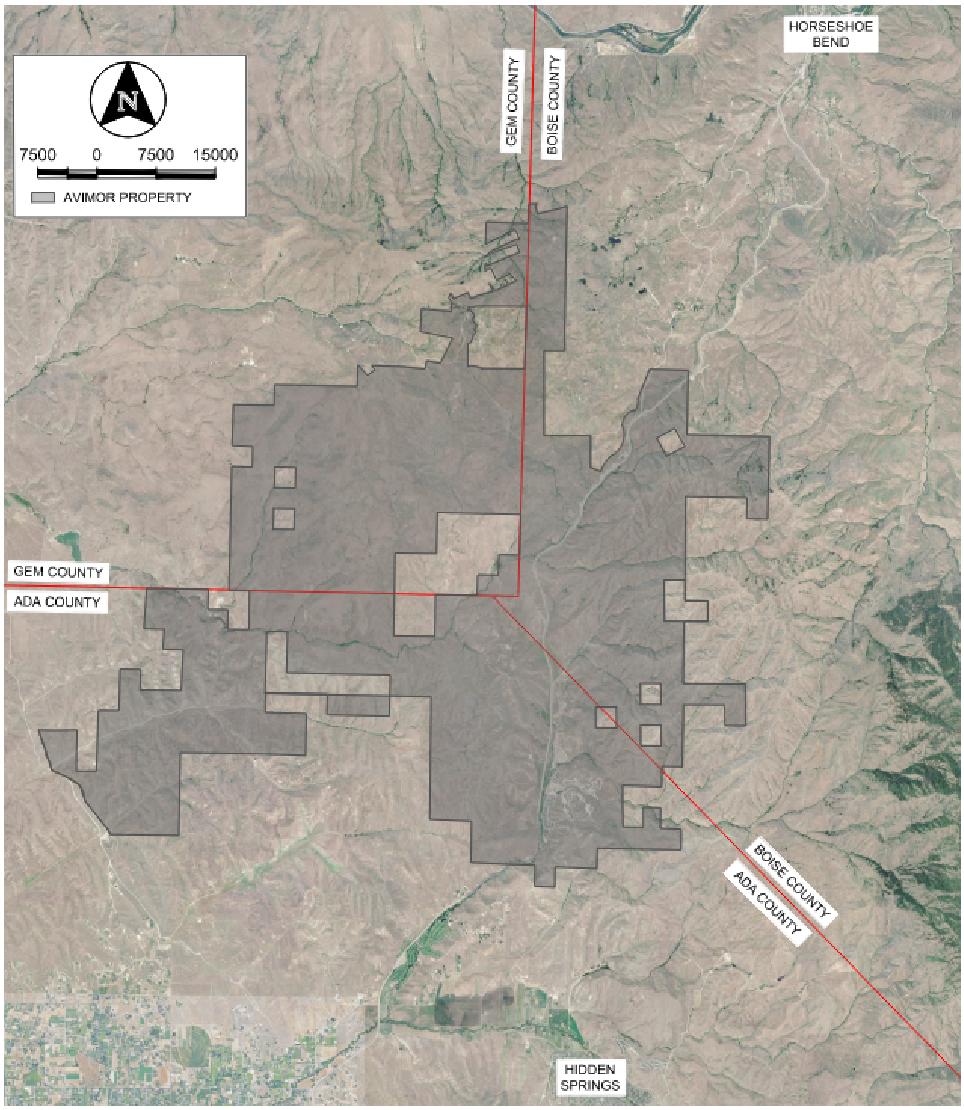


Figure 1-2 Avimor Location Map

The existing topography of the area ranges from relatively flat fields and pasture to moderately steep creek drainage valleys and steep side hills, with some rock outcroppings. Elevation ranges between 3150 and 3520 feet above mean sea level. The portions of the property that will be developed lie generally in the valley bottoms of two separate drainages and their tributaries.

1.2 Reuse Objective

The goal of the reclamation and reuse project is to provide cost-effective water reclamation. Reclaimed water is currently being surface-applied to rapid infiltration beds. Future plans include storing water in the underground aquifer, retrieving the filtered water through wells, and using the groundwater for landscape irrigation during the growing season.

Operational and maintenance activities at the AWRF require sludge to be removed from the facility periodically. Sludge removal is typically performed annually and is expected to be removed in higher frequencies as the community develops. The sludge is dewatered and transported to a landfill for disposal while the filtrate is reintroduced to the headworks. Avimor plans to land apply the filtrate to a proposed parcel of land on the west side of Highway 55. Plants will provide nutrient uptake from the filtrate and reduce levels of nitrogen and phosphorus in the AWRF.

Site Description

2.1 Site Geology

The APC is being developed on approximately 19,590 ± acres in portions of Ada, Gem, and Boise Counties.

The APC site is underlain by two principal bedrock units: granite and granite-like rocks of the Cretaceous-age Idaho Batholith and an assemblage of sedimentary and volcanic rocks of the Quaternary/Tertiary-age Idaho Group. The bedrock units are heavily weathered and generally occur as small, inconspicuous outcrops within mapped areas.

Rocks of the Idaho Batholith consist of light to medium gray granite, granodiorite, and meta-granite. This unit weathers to grass-covered slopes with occasional gray to dark gray outcrops and residual boulders. Areas underlain by the deeply weathered Idaho Batholith rocks can generally be recognized by the presence of light grayish-brown, sandy soil with isolated residual granite boulders. This condition is most apparent on the crest of the ridge west of Hwy 55 and in the upland areas north of Spring Valley Creek. In areas adjacent to faults, the granitic rocks commonly exhibit pockmarked erosional pits and reddish-brown iron staining. Because of the lack of continuous outcrop, the relationship between the sedimentary and volcanic rocks within the Idaho Group is unclear. In the general area of the APC, the Idaho Group consists of light to medium brown mudstone, claystone, and volcanic ash beds divided by medium to dark brown sandstone beds. The Idaho Group includes one significant mappable subunit, or in geologic nomenclature a "member." This subunit is the Pierce Park Sand member. The Pierce Park Sand is composed of medium to coarse well-sorted sand with silty and clayey sand interbeds. Based on field observation, the majority of the Idaho Group consists of the fine-grained units forming smooth hillslopes.

Soil derived from the fine-grained sedimentary units of the Idaho Group is typically medium to dark brown and rich in clay. This soil readily contrasts with the sandy soil developed over the Pierce Park Sand member. This transition is apparent in places along the ridge between Burnt Car Draw and Spring Valley Creek, where the light and dark soil types are clearly exposed adjacent to one another.

The mudstones and claystone of the Idaho Group are divided by volcanic rocks consisting of ash and volcanic mudflows. These volcanic deposits are a result of eruptions from nearby volcanoes that rained down hot ash and wet, ash-rich mudflows. Individual ash and mudflow units within this volcanic series range up to approximately 15 feet thick, but they are generally expressed as 3 to 6-foot thick beds within the fine-grained sedimentary units. The volcanic rocks are deeply weathered and generally outcrop as medium brown to dark greenish-brown, soft ledges. Typical exposures of these rocks are the dark-colored, low outcrops along the north side of Burnt Car Draw. The interbedded relationship between the sediments and volcanic rocks of the Idaho

Group is best illustrated on the prominent hillslope immediately north of where Spring Valley Creek emerges from the confines of the east-west canyon and turns to the south in Spring Valley. Here, the volcanic beds form resistant ribs in the otherwise smooth-weathering, fine-grained sedimentary units of the Idaho Group.

Several hillslopes along the eastern side of Spring Valley are covered with cobbles and gravels rich in quartz, granite, and other resistant bedrock materials. These occur at elevations ranging from approximately 3260 to 3380 feet and rest on Idaho Group sediments. These deposits are believed to be remnants of an ancient river terrace system similar to the prominent better-developed river terraces observed on the south side of the Boise River valley.

The relatively flat area in Spring Valley is made up of unconsolidated sediments recently deposited by Spring Valley Creek. These units are predominantly silt and clay, with frequent fine sand layers.

The most prominent structural elements in the project area are Spring Valley and the steep, granitic uplands to the northeast and south. Northwest-trending faults probably account for positioning of older granitic rocks with younger Idaho Group sediments along Spring Valley Creek canyon and on the southern end of the property (Figure 2-2). These northwest-trending faults are probably related to the Boise Front fault system, a series of faults that define the prominent mountain front observed in the Boise River valley north of Boise.

Less obvious but likely present are north-south trending faults defining the east and west margins of Spring Valley. While the western margin is generally linear, the eastern margin is less distinct, with several prominent, unnamed creeks entering the valley on the east side. Taken together, the high-angle faults define a graben, or down-dropped block, occupied by Spring Valley and surrounded by upthrown blocks forming the highland areas.

Idaho Group rocks within the graben generally dip gently to the center of the graben. This tendency is best illustrated in the prominently bedded hillside on the north side of Spring Valley Creek and in the volcanic units outcropping in Burnt Car Draw. In these locations, the beds are gently tipping downward to the west toward the axis of Spring Valley. No indications of active faulting such as fault scarps, flatirons, or recently displaced strata were observed. No active fault zones were shown mapped in the area in statewide fault-zone maps or other sources reviewed for this project. As a result, no mitigation or remedial design will be required for potential fault rupture.

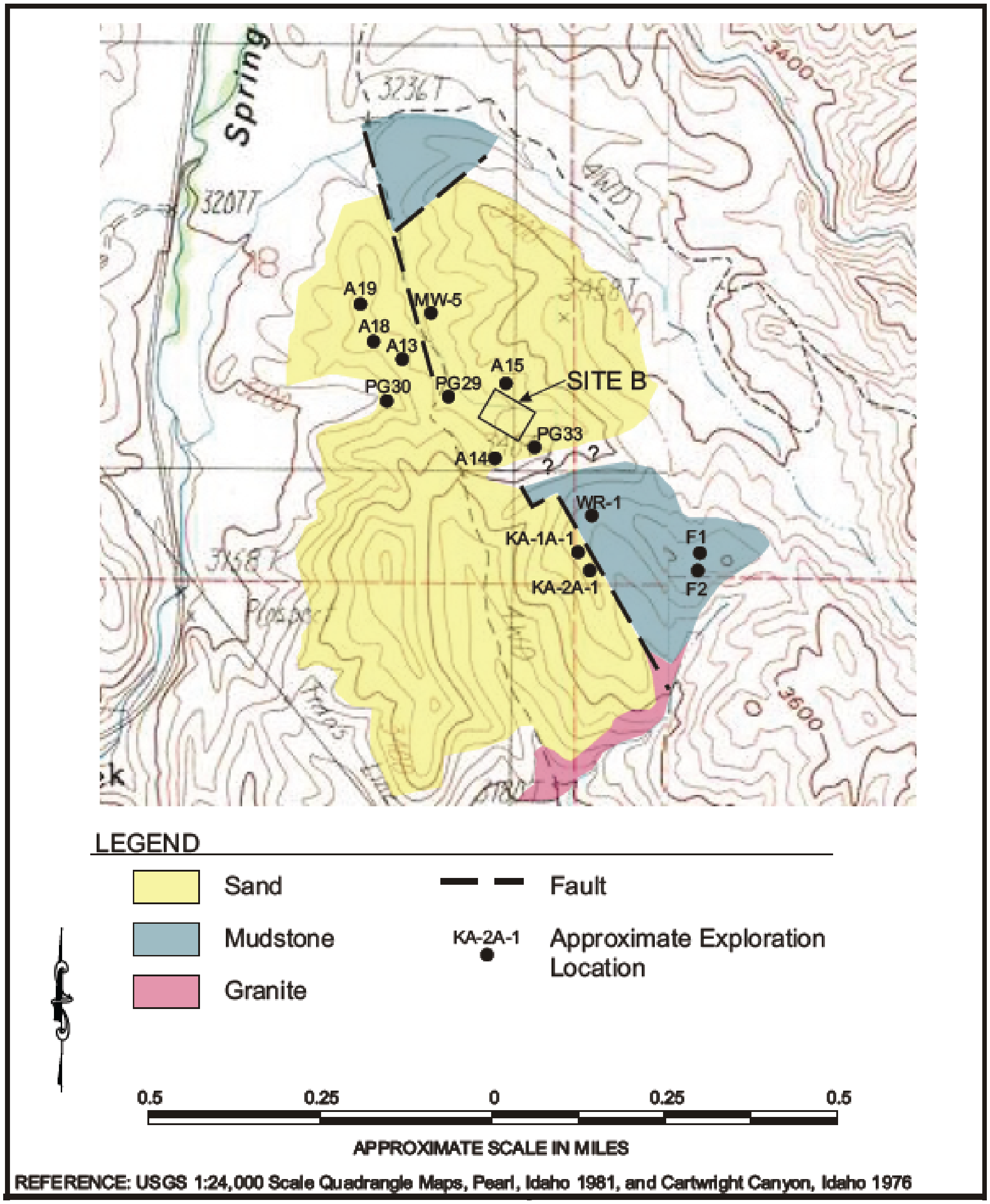


Figure 2-1 Generalized geologic map by Kleinfelder

Flow Projections

3.1 Growth

For the purpose of wastewater planning, the number of residents contributing wastewater to the treatment system needs to be determined. This factor should be based upon occupied homes, not building permits. Since 2008, Avimor has experienced consistent growth. The number of building permits secured during this time exceeds the number of homes that were occupied by residents. As such, the number of building permits should not be used to determine per home flows and other factors. It was determined that water meter records may provide the best evidence for the number of occupied homes.

As of 2021, the total number of residential units is 664 according to Avimor Development LLC. Based on market projections provided by Avimor Development LLC, the planned community is expected to develop at a rate of 200 houses per year therefore, build-out will occur in the year 2059 as shown in Figure 3-1. The number of houses occupied from 2009 to 2021 are based upon the number of houses sold within the community.

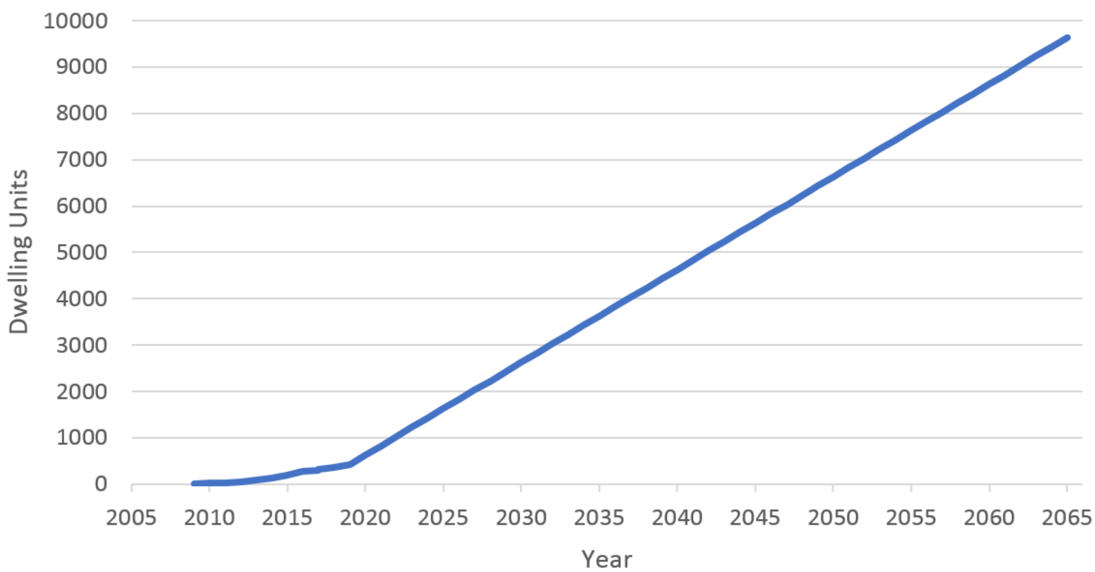


Figure 3-1 Buildout Projection

3.2 Design Parameters

Collection system design is based upon material selection, pipe slope, and sewer flow. For the purposes of this master plan, all flow is routed to the treatment facility with the following general system assumptions:

Pipe material:	SDR 35 PVC / AWWA C900 DR18
Manning's Roughness:	0.010
Hazen Williams C (force mains):	150
Peaking Factor:	2.8
Pipe slope:	minimum, per Ten State Standards*

The design parameters were used to create Table 3-1 for gravity sewer pipe. Maximum average daily flow and peak flows are provided for each pipe diameter along with their respective velocities. The sewer pipe slope may be adjusted to be equivalent to topography in areas with considerable elevation change.

Table 3-1 Gravity Pipe Design Criteria

Pipe Diameter	Minimum Slope	Average Flow Max Ratio	Average Flow Design Capacity	Average Flow Velocity	Peak Flow Max Ratio	Peak Flow Design Capacity	Peak Flow Velocity
(inch)	(ft/ft)	(d/D)	(gal/day)	(ft/sec)	(d/D)	(gal/min)	(ft/sec)
8	0.00400	0.413	229,993	2.62	0.82	447	3.25
10	0.00280	0.413	348,891	2.54	0.82	679	3.16
12	0.00220	0.413	502,889	2.54	0.82	978	3.16
15	0.00150	0.413	752,893	2.44	0.82	1,464	3.03
18	0.00120	0.413	1,095,036	2.46	0.82	2,130	3.06
21	0.00100	0.413	1,507,866	2.49	0.82	2,933	3.10
24	0.00080	0.413	1,925,541	2.43	0.82	3,745	3.03
27	0.00067	0.413	2,412,415	2.41	0.82	4,692	3.00
30	0.00058	0.413	2,972,682	2.40	0.82	5,782	2.99
33	0.00052	0.413	3,629,248	2.43	0.82	7,059	3.02
36	0.00046	0.413	4,304,903	2.42	0.82	8,373	3.01

3.3 Groundwater

Groundwater in this area is relatively deep and not anticipated to impact or contribute to the sewer collection system. Infiltration and inflow (I&I) is assumed to be zero based upon an absence of groundwater and all infrastructure is new construction without storm water connections to the sewer collection system.

3.4 Residential Density

The community residential density is planned to be approximately 0.5 residential units per acre. The developable land is planned with up to 9,190 dwelling units. Actual residential densities will vary across the community based on each area’s land use. The average household density is expected to be 2.5 people per household resulting in a population of approximately 22,975 people in the community.

3.5 Land Use

The Avimor community is comprised of various land uses which will vary in the amount of wastewater generated per acre. A general description is provided as well as the anticipated breakdown of residential and commercial business in each land use.

3.5.1 Village Center

The Village Center district is the heart of the community and the main activity center for Avimor which is designed to accommodate commercial, community, residential, and cultural activities. This district may include shopping, business and professional offices, research and development, hotel and resorts, vineyards and wineries, cultural, educational, civic, community facilities, and parks and recreational facilities to serve the entire Avimor population. Medium to high density residential areas may be included as a secondary use and residential units may be stacked vertically above business uses.

3.5.2 Open Space

Community Open Space is land set aside for recreation, agriculture, habitat, vegetation, scenic, or similar uses and is intended to primarily serve the Avimor residents. Developed Open Space may include public, semi-public, and private recreational facilities, amphitheaters, golf courses, pathways and trails, landscape zones in and adjacent to major roadways including areas outside of a dedicated right-of-way, greenbelts, cultural, community, educational, and quasi-public facilities, equestrian centers, and trailheads, as well as parks, playfields and natural open spaces. Agricultural uses, such as vineyards, wineries and plant nurseries, are also considered Community Open Space. Facilities and tracts of land owned by the Owners' Association are also considered Community Open Space.

Regional Open Space is intended to serve both Avimor residents and the general public and may be adjacent to, or provide connection to, large scale regional open space. Regional Open Space may include many of the amenities provided in Community Open Space as well as active regional parks, trail corridors (such as the proposed Eagle Canyon Regional Park, Trail and Open Space Corridor.) Regional Open Space may be owned and maintained by the City, the Owners' Association, private parties, a land trust, or other conservation group or entity.

3.5.3 Foothills Residential

Foothills Residential districts are intended to provide residential neighborhoods that are one quarter acre in size on average. Foothills Residential districts may also include farmers' market, schools, vineyards, community centers and other complementary uses.

3.5.4 Mixed Use / Commercial

Mixed Use districts are intended to provide a variety and mixture of retail, business, residential and employment opportunities for Avimor and area residents. These districts will accommodate office, flex space, light manufacturing, research and development, shopping, business, lodging, professional and support commercial services, primary, secondary and higher educational facilities, parks and recreation facilities, vineyards and wineries, and residential uses.

3.5.4 Village Residential

Village Residential districts are intended to provide residential neighborhoods with a range of lot sizes and housing types depending on location, site conditions, and market influences to create a community that emphasizes housing diversity. Village Residential districts are comprised of Single-Family Detached, Single-Family Attached and Multi-Family at various densities and mixes. These districts may also include schools, day care facilities, worship sites, parks, playfields, and other recreational facilities, resorts, vineyards and wineries, and other complementary uses. Some village residential uses are also permitted in non-residential land use districts.

3.5.5 Land Use Profile

Table 3-2 is based upon the descriptions provided and is a general assumption for how the land use areas will develop. Each land use has been subdivided by percent into hotel, office, park, residential, restaurant, retail, and miscellaneous for wastewater planning. Characterizing each land use is necessary to determine flow contributions for the community at buildout.

Table 3-2 Land Use Profile

Land Use	Hotel	Office	Park	Residential	Restaurant	Retail	Misc	Storage
Community/ Village Center	10%	20%	10%	5%	20%	20%	5%	0%
Developed Open Space	0%	0%	10%	0%	0%	0%	90%	0%
Foothills Residential	0%	0%	0%	100%	0%	0%	0%	0%
Mixed Use/ Commercial	10%	20%	5%	10%	20%	20%	5%	0%
Multi-Family	0%	0%	0%	100%	0%	0%	0%	0%
Special Areas	0%	0%	0%	0%	0%	0%	100%	100%
Village Residential	0%	0%	0%	100%	0%	0%	0%	0%

3.6 Residential Planning

Residential homes will be developed in areas based upon existing slopes, soils, and planning considerations. Planning densities will vary based upon the land use and market demands at the time of construction.

Sewer flow to the treatment facility is available for the Avimor community from January 2009 to August 15, 2018 and is shown in Figure 3-2. Prior to the treatment facility being started up for the first time in June 2013, all flows were hauled away to be treated at a separate facility not owned by Avimor. The community was unable to produce enough wastewater flows to justify operating the ARWF during the hauling period. Flow data during the hauling period is based on the total number of gallons hauled each month and ARWF operation flow data was measured by a flowmeter. Flowmeter data prior to May 2017 is considered unreliable due to statements made various wastewater land application annual reports, noticeable irregularities in flow, and discrepancies between influent and effluent flow data. The influent flow meter was replaced in May 2017 to correct issues with inaccurate flow readings. It is believed that flows after this date are accurate.

Flow data from July 1, 2017 through August 15, 2018 was chosen for analysis and is shown graphically in Figure 3-3. These dates were selected for analysis because it is after the influent flow meter was replaced and reliable occupied household data is available. NOAA database rainfall data from Boise, ID is shown to demonstrate an absence of impacts to the system flows from inflow and infiltration. The data used to calculate the equivalent residential unit (ERU) flows for this time period are as follows:

Total Flow: 16,907,007 gallons
AWRF Average Daily Flow: 41,136 gallons/day
Occupied Units in July: 294 units
Occupied Units in August: 364 units
Calculated Average ERU Flow: 125 gallons/day

This ERU flow is lower than typical values seen within the Treasure Valley and is due to several factors including home sizes in the community, water conservation measures implemented during construction, hot water recirculation pumps, and the number of people residing in each household. The flow per ERU is assumed to be 160 gpd for planning purposes based upon the historical flow data. Figure 3-3 illustrates the daily flow for the calculated and proposed ERU flows when multiplied by the historical number of occupied households. The assumed value of 160 gpd is larger than the calculated average of 125 gpd. Avimor received approval from DEQ in the 2019 Facility Plan to reduce the current planning figure of 300 gpd/ERU to 160 gpd/ERU based upon record data. The two peaks in flow from September 2017 to October 2017 and March 2018 to May 2018, as shown in Figure 3-3, are due to construction dewatering activities. Avimor staff no longer allows contractors to dewater into the sewer collection system.

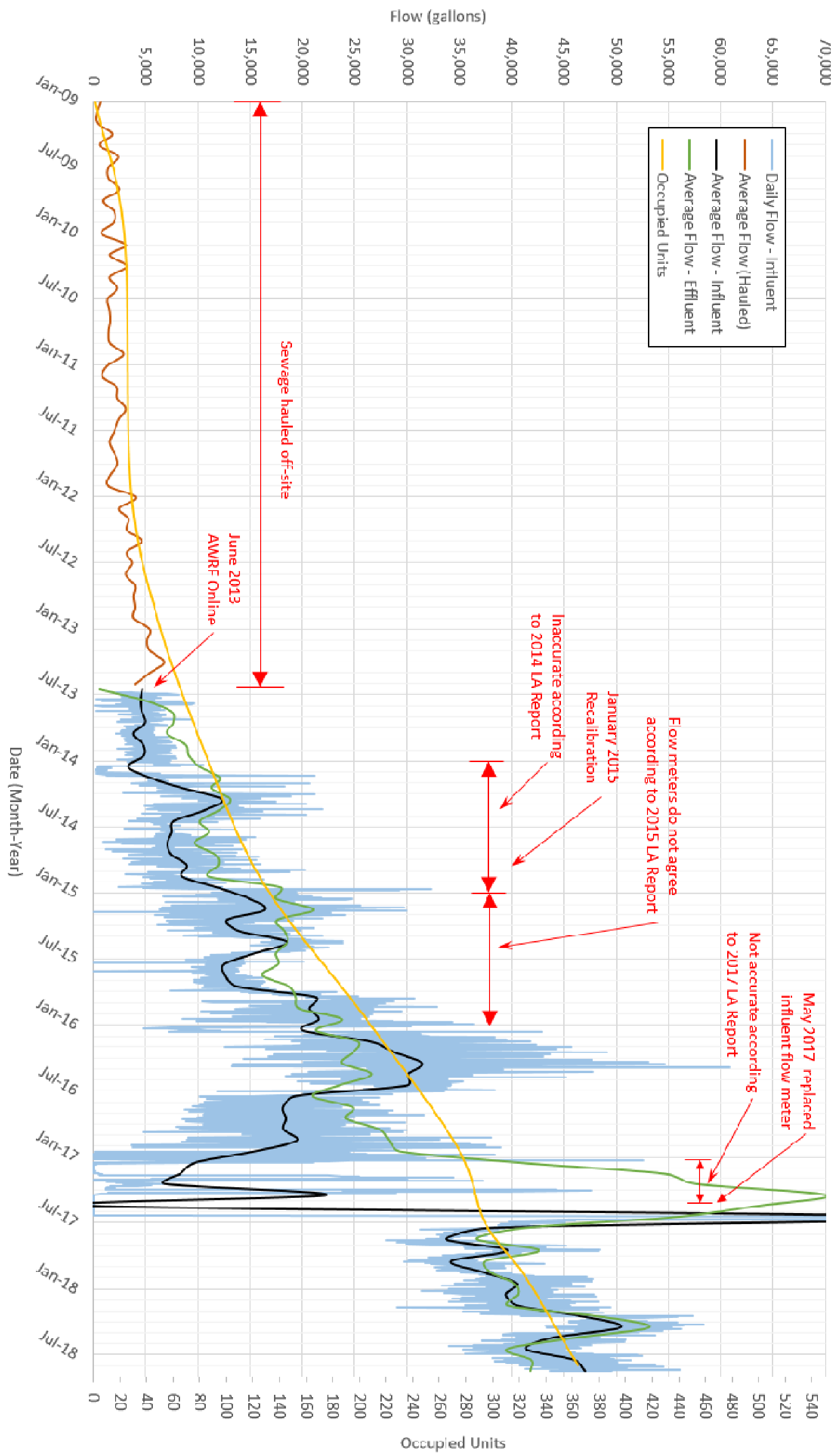


Figure 3-2 AWRF Influent Flow (1-1-2009 to 8-15-2018)

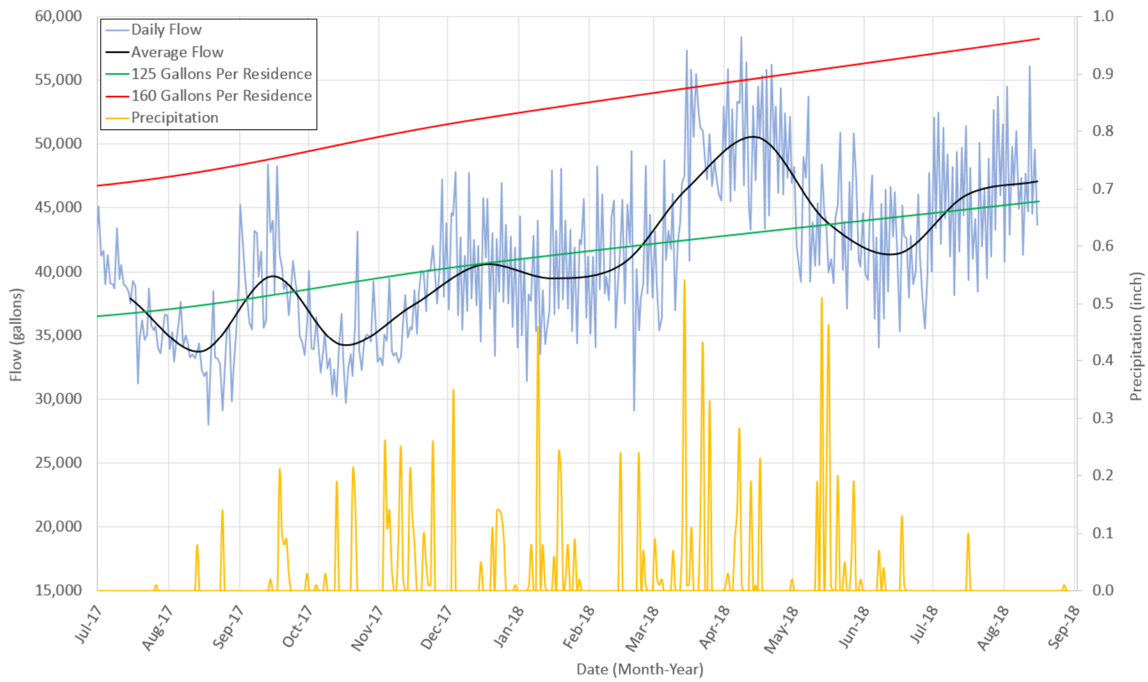


Figure 3-3 AWRF Flow (7-1-2017 to 8-15-2018)

A diurnal curve was developed, using plant influent data, from July 1, 2017 through August 15, 2018 as shown in Figure 3-4. Plant influent data for this time period is nearly exclusively made up of residential units except for the Avimor community center and the Avimor planning office. This diurnal curve shows that a peaking factor of 2.8, as provided in the the design parameters, is a conservative assumption.

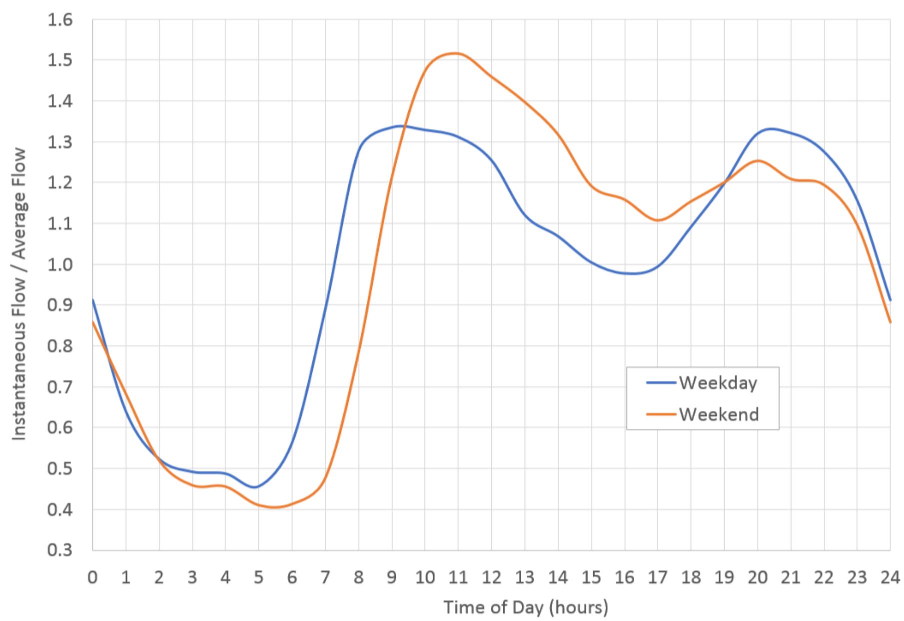


Figure 3-4 Diurnal Curve (7-1-2017 to 8-15-2018)

3.7 Commercial Planning

Commercial development will be located throughout the development and primarily concentrated around the village centers. The only commercial facilities that are currently in operation is a gas station, brewery, the Avimor Community Center, and Avimor planning offices. It is assumed that most of the commercial uses within Avimor will be hotels, offices, restaurants, and retail. Some commercial uses such as a brewery or restaurants will tend to have a larger water usage and associated impacts to the sewer collection system. Without specific information about the commercial uses at this time, broad planning figures were used for this report to estimate future demands as shown in Table 3-3.

Table 3-3 Commercial Flow Generation Rates

Flow Generation Rates (gal/acre/day)	Hotel	Office	Park	Restaurant	Retail	Misc	Storage
	2400	750	150	10000	200	800	100

The flows in Table 3-3 are based on the following assumptions:

- Hotel
  IDAPA 58.01.03 - 60 gallons/bed space
  Assumed 40 rooms/acre
- Office
  IDAPA 58.01.03 - 20 gallons/employee
  Assumed 37.5 employees/acre
- Park
  IDAPA 58.01.03 – 5 gallons/person
  Assumed 30 people/acre
- Restaurant
  Maryland Department of the Environment (Rev. June 2011) - 50 gallons/seat
  Assumed 200 seats/acre
- Retail
  Metcalf & Eddy - 1.5 gallons/parking space, 8 gallons/employee
  Assumed 80 parking spaces and 10 employees
- Miscellaneous
  Flow generation rate assumed
- Storage Units
  Flow generation rate assumed

3.8 Future Flows

Three flow conditions are significant to the design of the treatment facility: average daily flow, peak daily flow, and peak hour flow. The average daily flow, an indicator of the typical daily flow rate, is useful in estimating normal operating conditions for the treatment system. The peak daily flow is used as the critical process design condition, and the peak hour flow is used to define the hydraulic capacities of certain elements of the system, including lift stations, screening, and equalization basins.

Future flow will be dependent on the land use flow generation rates for the development key numbered regions that are provided in Appendix B. The commercial flow generation rates and land use flows are also provided in Appendix B of this report. Commercial flow generation rates are used in conjunction with the Table 3-2 (Land Use Profile) to develop non-residential flows in Table 3-5.

The collection and conveyance system has been designed to have 8” to 21” gravity collection lines as shown in the collection system maps provided in Appendix A of this report. The collection lines have been designed using minimum slopes however, they may be downsized if sufficient slope is used during the design to convey all anticipated flows. Estimated flows for each sewer area are shown in Table 3-5 and the extents of each sewer service area are shown in Figure 4-1.

Table 3-4 Sewer Service Area Estimated Flows

Area	Average Daily Flow (gal/day)		Peak Hour Flow (gal/min)	
	Historical Flow 125 gal/du	160 gal/du	Historical Flow 125 gal/du	160 gal/du
1	665672	852061	1294.4	1656.8
2[1]	81146	103866	157.8	202.0
3	306281	392039	595.5	762.3
4[2]	366698	469373	708.6	907.0
5	168899	216191	303.5	388.5
Total	**1,588,696**	**2,033,531**	**3059.8**	**3916.5**

[1]Estimated flow collected and produced in Area 2 not including the total flow produced in Areas 3, 4, & 5.
[2]Estimated flow collected and produced in Area 4, not including the total flow produced in area 5.

Collection System

4.1 Introduction

The proposed collection system is comprised of gravity sewer pipe that will be generally located within roadways and lift stations where it becomes impractical convey sewer with gravity sewer pipe across the existing terrain. The collection system will terminate at a treatment facility located at the southeast part of the development.

4.2 Topographic Service Basins

The sewer service basins within the Avimor planned community will be governed primarily by the natural topography. As the system is expanded across terrain or over the top of topographic features, consideration is given to how these areas will be routed to the sewer system.

There are 5 primary sewer service basins that will serve the Avimor Planned Community as shown in Figure 4-1. The majority of Areas 1 through 3 are able to flow by gravity into the existing trunk mains and eventually into the wastewater treatment facility. Areas 4 and 5 require that the sewer be lifted and pumped to the gravity system. Because of the varying terrain, there will be small areas, such as the outlying reaches of Area 2 and 3, which must be lifted over the ridge into the gravity system. Collection system maps are provided in the Appendix A of this report.

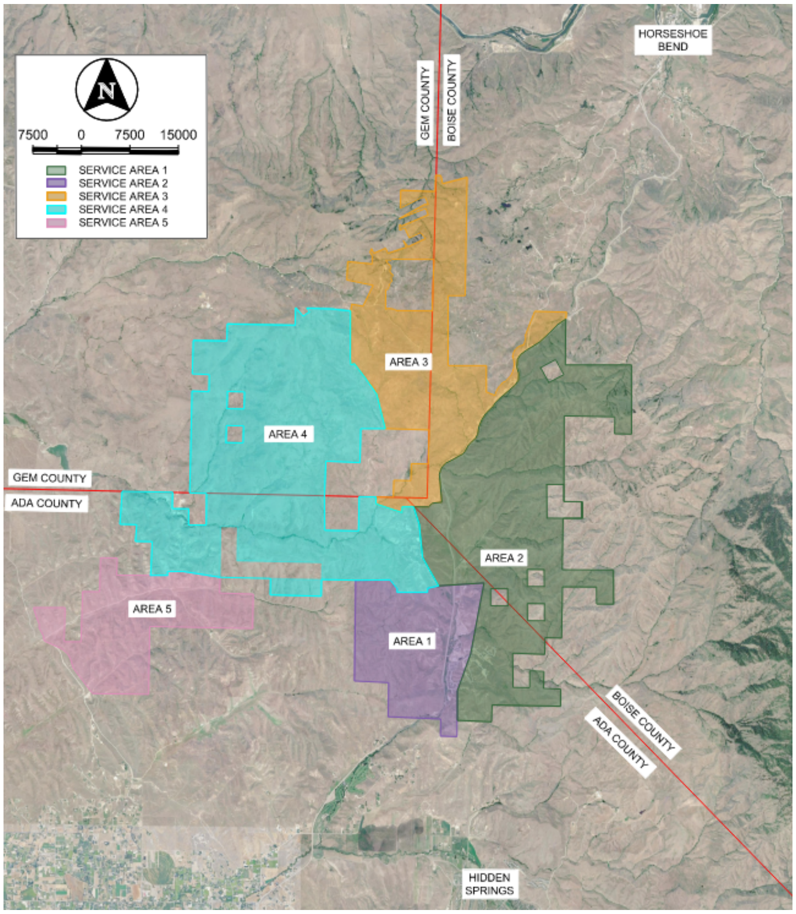


Figure 4-1 Sewer Service Areas

4.3 Trunk Mains

Most of the planned roadways within Avimor will follow the natural topography and will be located along the bottom of draws, especially in high slope areas. These same alignments will typically represent the best location for sewer trunk mains. As the community is developed, priority shall be given to maintaining sewer corridors within the right of way.

Sewer trunk mains have been planned to serve the Avimor Planned Community. The trunk main sizes have been sized based upon anticipated flow for the development areas using minimum slopes. Because the site is relatively hilly with most natural grades in excess of minimum pipe slopes, it is not anticipated that sewer trunk main depths will exceed typical minimum bury depths. The sewer mains may be downsized during design if the proposed slopes are greater than the minimum allowable and there is adequate capacity for all proposed flows using both diversion structure scenarios discussed in Section 4.5. In general, gravity trunk mains are planned to be bell and spigot gasketed SDR-35 PVC or PS46 for large diameter PVC piping. Welded HDPE may be considered when advantageous for force mains or in areas with considerable depth. Final pipe material decisions will be determined by the design engineer.

4.4 Lift Stations

The natural grade of the site does not allow for all sewer service areas to grade by gravity to the treatment facility. Lift stations have been strategically located to minimize the amount of pumping necessary while allowing service to the full development area. Installation of sewer lift stations will result in increased power consumption, increased maintenance, and operational costs. The plan includes two primary regional lift stations and up to seven smaller lift stations. Table 4-1 represents the anticipated flows that will be pumped by the proposed lift stations.

Table 4-1 Lift Station Flow

Lift Station	Sewer Area	Avg. Daily Flow (gpd)	Peak Hour Flow (gpm)
1	5	13,802	26.8
2	5	216,191	388.5
3	5	15,932	31.0
4	4	12,264	23.8
5	4	423,382	791.3
6	4	685,564	1295.5
7	4	685,564	1295.5
8	3	62,536	121.2
9	2	101,712	197.8
10	2	34,692	67.5
11	2	85,904	167.0
12	2	1,181,470	2259.7

4.5 Diversion Structure

A diversion structure is planned at the south end of sewer area 3 to allow for flows to be routed to lift station 1. Buildout of the sewer infrastructure will reroute flows to sewer area 1 as shown in Appendix A of this report. The purpose of the diversion structure is to allow flexibility in sewer operations depending upon the timing of the north and south highway crossings. The necessity for this structure will be entirely dependent upon the order of development within the planned development areas.

4.6 Highway Crossings

The central wastewater treatment facility is located on the east side of Highway 55 and the majority of the property within Avimor is located on the west side of the highway. Currently there are two planned crossings of the highway. One is near the WWTP and the other is North of the community center and gas station just off Highway 55. The timing of these crossings will depend upon the construction of future development. It is anticipated that an open cut of Highway 55 will not be permitted and that crossings will need to be bored. This effort will require coordination with and approval from the Idaho Transportation Department.

Wastewater Treatment and Reuse

5.1 Treatment Facility

Avimor is served by a central wastewater treatment system, Avimor Water Reclamation Facility (AWRF), as shown in Figure 5-1. The facility is owned and operated by the Avimor Water Reclamation Company LLC. The AWRF is planned to serve the entire planned development at final build out and will be upgraded as necessary to keep up with demand. All sewer generated within the planned community will be routed by gravity or force main to this location. The wastewater treatment facility is generally located at a lower elevation than most of the surrounding areas.

The existing treatment system is a membrane bioreactor (MBR) which utilizes biological treatment in combination with in-basin membrane filtration. Plant effluent is discharged as Class B recycled water and pumped up to a water storage tank where it is utilized for reuse.

Figure 5-1 Avimor Water Reclamation Facility

5.2 Wastewater Treatment

A process flow diagram for the AWRF is shown in O&M Manual which is available in Appendix E of this report. The existing wastewater treatment facility is discussed below.

5.2.1 Raw Influent Pump Station

Raw sewage flows by gravity to the raw influent pump station where it is lifted to the wastewater treatment facility. The 1.0 MGD condition for the pump station includes three submersible pumps, each designed to deliver 1,400 gpm and operate in a two (duty) + one (standby) arrangement. The current setup includes two pumps with a one (duty) + one (standby) arrangement. The third pump will be added as part of the Phase 2 expansion

The pumps have variable speed motors that cycle based on liquid level set points. Pump run times and alarm features are monitored in the programable logic controller (PLC) and reported to the SCADA system. The pump station consists of a precast concrete pump vault and precast concrete valve vault.

5.2.2 Headworks

The headworks screening facility is sized to accommodate three fine screens which can handle peak instantaneous flow from the raw influent pump station. There are currently two fine screens with an integral washer and compacter that removes debris larger than 3 mm, washes it to minimize organic components, and compacts it to remove the water and reduce volume. As material accumulates on the screen, the upstream water surface will increase and initiate a cleaning cycle. Two duty screens will be required at 1.0 MGD capacity, with one standby unit for backup. The third screen will be added as part of the Phase 2 expansion. The current capacity of the AWRF headworks is 1.0 MGD.

5.2.3 Equalization and Process Feed Pumping

The equalization tank provides peak flow attenuation at the AWRF. The equalization tank is mixed with a pump and set of mixing nozzles. Two submersible process feed pumps in the equalization basin deliver flow to the activated sludge system. The third pump will be added as part of the Phase 2 expansion. The pumps run on variable frequency drives and are controlled based on the equalization tank level. The equalization tank is constructed with cast-in-place concrete and has a concrete deck designed to support the weight of a membrane module weighing up to 3 tons.

5.2.4 Phosphorus Removal

The influent wastewater is expected to contain a phosphorus concentration of 8 mg/L since testing data is not available at this time. Phosphorus is an environmental concern because it is a nutrient that helps support algae growth in surface waters. The facilities IPDES permit has an effluent limit of 1.2 lb/day limit average monthly limit on phosphorous. The expected facility influent phosphorus loads are as follows:

- Phase 1 - Plant Flow - 0.33 MGD 22.0 lbs/d of P
- Phase 2 - Plant Flow - 0.66 MGD 44.0 lbs/d of P
- Phase 3 - Plant Flow - 1.0 MGD 66.7 lbs/d of P

To remove phosphorus, alum is dosed into the MBR train as a chemical coagulant and is removed with the solids.

5.2.5 Activated Sludge System

The activated sludge system and filtration system are designed to continuously treat the design average daily flow and meet the peak daily flow. The anoxic basins are un-aerated and mixed with deck-mounted mixers. Flows are discharge from the anoxic basin to the aeration basin where the mixed liquor is aerated with fine bubble diffusers. A submersible anoxic recycle pump delivers flow back to the anoxic basins to enhance denitrification. A submersible MBR recycle pump delivers flows to the end of the MBR basins to reduce solids accumulation in that tank. Instrumentation for the activated sludge system includes an online suspended solids analyzer to control overall system sludge ages and a dissolved oxygen meter for aeration basin blower control.

Table 5-1 provides physical descriptions for the system, Table 5-2 provides recycle pump criteria, and Table 5-3 provides air system requirements.

Table 5-1 Basin sizes and hydraulic detention times (HRT) for the AWRF

Basin	L (ft)	W (ft)	H (ft)	Nominal SWD (ft)	Nominal Volume (gal)	No.	Nominal Volume (gal)	HRT (hours)
Anoxic	16	44.5	17	14.00	74,560	1	74,560	5.4
Aeration	14	14	17	13.75	20,159	3	60,475	3.3
MBR	14	14	17	14.00	20,525	3	61,575	3.9

Table 5-2 Process recycle pumping (per train) for the AWRF

Service	Recycle Rate[1]	Capacity[2] (gpm)	No.	Total Flow (gpm)
Anoxic recycle	10x	875	3	2310
MBR recycle	5x (forward)	525	3	1386

1 Max pump sizing based on maximum forward flux rate with all MBR modules in-service. These values represent a 12.9% additional flow capacity and provide for robustness if a single pump fails.

2 Pumps are equipped with variable frequency drives and flow meters. Flow capacities shown are maximums.

The aeration basins are equipped with 9-inch disc fine bubble rubber membrane diffusers in a dedicated grid supplied by a 4-inch drop pipe. Each grid contains 80 diffusers. Average day conditions are estimated to require 115 scfm airflow to each aeration basin. Aeration basin blowers are sized for 500 scfm so the maximum aeration capacity is 167 scfm per basin. Any aeration demand peaking factors beyond these requirements will be absorbed by the aeration capacity in the MBR basins.

Table 5-3 Aeration rate requirements for the AWRF

Service	Air Per Train (scfm)	No. Trains	Air Total (scfm)
Aeration basins[1]	167	3	500
MBR basins[2]	330	3	990
Sludge holding tank[3]	620	1	620

1 65% oxygen demand in aeration basin, assumed 0.4 α factor, 23% SOTE for aged diffuser elements (26% original) and 2 mg/L dissolved oxygen concentration. Air rates above provide sufficient air for average daily loadings at a 2.0 mg/L dissolved oxygen concentration. Peak daily loadings will depress oxygen concentration in the aeration basin down to 0.75 mg/L, driving some of the nitrification reaction to the MBR tank.

2 100 scfm per EK400 plus 10% additional safety factor

3 Based on AOR of 31 lbs/hr average decay over 5-day storage. Minimum air for mixing at 20 to 40 scfm/1,000 cf is 340 to 640 scfm.

5.2.6 Blowers

Five blowers provide air to the treatment facility. One set provides air for the aeration basins and the other set of blowers provides air for the membrane modules as well as for the sludge holding tank. One blower is installed as a redundancy for the system.

In the aeration basins, air provides oxygen for biological reactions. These blowers are equipped with variable frequency drives and controlled based on a dissolved oxygen meter reading and dissolved oxygen setpoint in the PLC.

Membrane modules use air to scour solids away from the membrane surfaces to maintain flux rates through the membranes. When operating, each module needs a minimum 100 scfm and a minimum of 900 scfm is required for the membranes. Because air scour is critical to the operation of these membranes, an additional 10% air flow is provided in the blowers for 1,000 scfm total.

Aeration requirements for the sludge holding tank are greater than for the aeration basin but smaller than for the membranes. The sludge holding tank supply blower is identical to the membrane blowers and discharge into the same header so that the two have a common standby unit.

The membrane/sludge holding tank blowers are positive displacement tri-lobe blowers equipped with variable frequency drives. The sludge holding tank blower is maintained between a minimum setpoint for mixing requirements and what is needed to maintain a dissolved oxygen setpoint. The membrane blowers are controlled to provide sufficient airflow and are positioned in the control building.

5.2.7 MBR Modules and Permeate Pumping

Membrane modules separate the mixed liquor solids from the clear water. The modules lie submerged in the MBR basin, with permeate piping conveying clean water. Air is used to scour solids off the individual membrane faces. The MBR recycle pump ensures that solids do not accumulate in the MBR basin.

The mechanical frame of the module is bolted to the basin floor, with hard-piped air supply for air scour. Each module has two membrane cases that can be removed on a rail system. Permeate water is piped to the side of the MBR basin with 2” pressure/vacuum hoses, where it is hard-piped. An isolation valve for each membrane case is positioned in the pipe chase adjacent to the MBR basin. A monorail mounted on the deck allows the membrane cases to be pulled for inspection and servicing.

Permeate pumps pull water through the membranes. Each suction line is equipped with a vacuum pressure transmitter so that trans-membrane pressure can be monitored for each set. The pumps discharge into a single train discharge header that routes to the plant discharge. The common train discharge header pipe is equipped with a turbidity-monitoring device.

Table 5-4 Membrane design criteria for the AWRF

Element	Units	Phase 1	Phase 2	Phase 3
Effective membrane area per sheet	sf	8.6	8.6	8.6
No. sheets per EK400	No.	400	400	400
Total membrane area per EK400	sf	3,440	3,440	3,440
Average allowable flux rate, per Enviroquip	gfd	12.3	12.3	12.3
Design average allowable flux rate	gfd	12.11	12.11	12.11
Average flow capacity per EK400	gpd/EK400	41,796	41,796	41,796
No. EK400s in project phase	No.	9	18	27
Total flow capacity per design flux rate at 12.3 gfd	gpd	380,808	761,616	1,142,424
Firm capacity, one EK400 out of service each phase at 12.11 gfd	gpd	333,333	666,667	1,000,000

5.2.8 Disinfection

Each membrane permeate header is combined to a common effluent pipeline that discharges into the chlorine contact tank. The contact tank provides 30 minutes of contact time at peak daily conditions. A sodium hypochlorite feed system adds chlorine to the permeate.

5.2.9 Effluent Pumping

Vertical turbine-type effluent pumps deliver disinfected water to a water storage tank which can be routed to the rapid infiltration basins or used in the landscape PI system. In addition to effluent pumping, a smaller duty pump is provided for plant utility water that is used at the influent screens, as spray-down water in the process, for polymer makeup water, or for clean-in-place of the membranes.

5.2.10 Sludge Holding Tank

Waste sludge is pumped to the sludge holding tank where it is aerated and stored until it can be thickened or dewatered. Solids in the activated sludge basins are measured with an online suspended solids analyzer. The sludge holding tank is 46’-6”L × 27’W ×

17'H with a nominal maximum water surface of 14', yielding 131,000 gallons. At the plant capacity of 1.0 mgd, there will be just over 5 days of storage for thin (1.5%) sludge.

5.2.11 Thickening/Dewatering

A 1-m gravity belt thickener was initially provided for waste sludge thickening. The feed pump for the thickener, a submersible centrifugal pump with a variable frequency drive, is flow controlled with a magnetic flow meter. This thickener reduces sludge volumes by 60% to 75%. Current operations at the AWRF include the use of a Flo Trend roll-off container for dewatering. Waste sludge can be hauled off for disposal at larger publicly owned treatment works, disposed of at Idaho Waste Systems' Simco Road Landfill, or Ada County Landfill. Future liquid that is dewatered will be reused at a proposed nursery as discussed in the Reuse section of this report.

5.2.12 Backup Power

Backup power is provided with a generator. Two generators will be used at 1.0 MGD capacity however, only one is currently installed. With both generators, adequate backup power will be available for the plant to operate at 1.0 MGD.

5.2.13 Treatment Facility Capacity

The constructed facilities include the following primary components and associated treatment capacities shown in Table 5-5. The existing infrastructure is configured to allow three projects (initial construction and two expansions) equivalent to approximately 0.33 MGD per phase for a total capacity of 1.0 MGD.

Current permitting through DEQ limits RI bed capacity based upon an assumed per ERU flow of 300 gallons per day. Avimor's historic water usage shows an ERU flow of 125 gallons per day. A planning ERU of 160 gallons per day has been used in this report to size the collection system and show the buildout capacity of the treatment facility. Avimor is in the process of collecting actual RI capacity data that will be used to expand the number of ERU's that can be permitted within Avimor. A new land application permit is planned to be issued to account for the actual capacity of the RI beds.

Table 5-5 Treatment Facility Capacity

Component	Current Capacity	Proposed Capacity
Headworks	1.00 MGD	1.00 MGD
Mechanical Treatment	0.33 MGD	0.33 MGD
Rapid Infiltration Beds	0.19 MGD	0.30 MGD

5.2.14 Phases

Buildout of the existing facility includes three phases of construction however a total of 6 phases of treatment capacity upgrades are planned. The existing building was constructed to have adequate space for all instrumentation needed for the treatment plant expansion up to three (3) mechanical treatment trains. MBR expansions include the construction of reinforced concrete tanks, installation of the MBR, and additional equipment in the treatment building as necessary. Phase 4 will introduce a second treatment facility to the site and it will be upgraded in similar fashion as what is planned for the existing facility with space for additional MBR tanks and equipment. The new facility will be similar to the current treatment plant. Treatment capacity upgrades and expected phase implementation are as shown in Table 5-6. The expected construction dates are based on the expected population growth.

Table 5-6 Treatment Capacity Planning Phases

Phase	Description	Capacity	ERU's (160 gpd)	ERU's (125 gpd)	Expected Construction
Phase 1	Existing Facility	0.19 MGD[1]	1,188	1,520	Complete
Phase 1a	RI Basin Permitting	0.33 MGD	2,083	2,667	May 2021
Phase 2	MBR #1 Expansion #1	0.67 MGD	4,167	5,333	January 2024
Phase 3	MBR #1 Expansion #2	1.00 MGD	6,250	8,000	January 2033
Phase 4	Headworks / MBR #2	1.33 MGD	8,333	10,667	January 2041
Phase 5	MBR #2 Expansion #1	1.67 MGD	10,417	13,333	January 2050
Phase 6	MBR #2 Expansion #2	2.00 MGD	12,500	16,000	January 2058

[1]Capacity limited by RI basin permitted flow.

As the AWRF reaches the existing infrastructure capacity of 1.0 MGD, future expansion of the treatment facility will be necessary to sustained development of the community. Avimor controls all property adjacent to the treatment facility which will allow for future expansion as shown in Figure 5-2 in red. Necessary treatment system expansions will be possible adjacent to the existing facility without any anticipated site area limitations. Future locations for facility expansion will need to take access and grading into account, as much of the area is encumbered with hills.

Figure 5-2 MBR#2 Facility Expansion

5.3 Reuse

Avimor has evaluated long-term options for reuse, recharge, discharge, and storage of wastewater effluent. The AWRF currently treats wastewater to a Class B reuse level (IDAPA 50.01.17). This is a high level of treatment and allows for landscape irrigation of parks and roadside areas during the growing season when not in use by the public.

Current reuse options for Class B effluent include:

- Rapid Infiltration
- Sandy Hill Aquifer
- Landscape Irrigation
- Direct Discharge to Spring Valley Creek (IPDES Permit)

The method used for effluent reuse varies seasonally between landscape irrigation and RI bed infiltration under the Idaho Department of Environmental Quality (DEQ) permit M-211-03 which expired on May 2, 2021. Avimor Water Reclamation Company is currently reapplying for the IPDES permit and will not be performing any surface water discharge until permitted to. Discharge to surface water will be required to meet effluent limits in the new IPDES Permit. Average monthly effluent requirements for Class B reuse are shown in Table 5-7.

Table 5-7 AWRF Class B effluent criteria

Parameter	Permit Limit
Biochemical oxygen demand (BOD)	< 5.0 mg/L (monthly arithmetic mean)
Total Chlorine Residual	> 1 mg/L (after 30-minute contact time at peak flow)
Total nitrogen (TN)	< 8.0 mg/L (monthly arithmetic mean)
Total phosphorus (TP)	< 0.35 mg/L (monthly arithmetic mean)
Turbidity	< 2.0 NTU (daily arithmetic mean) < 5.0 NTU (at any time)
Total coliform	<2.2 tco/100 ml (median number of tco) < 23.0 tco/100 ml (at any time)
tco= total coliform organisms	

Results from effluent testing are available in Figure 5-3 and Figure 5-4. Total coliform is not shown in either figure because it has not been detected above the lower detection limit. Gaps in the lines shown in Figure 5-3 indicates levels of each constituent below the lower detection limit. Monthly sampling data is available in Appendix C of this report.

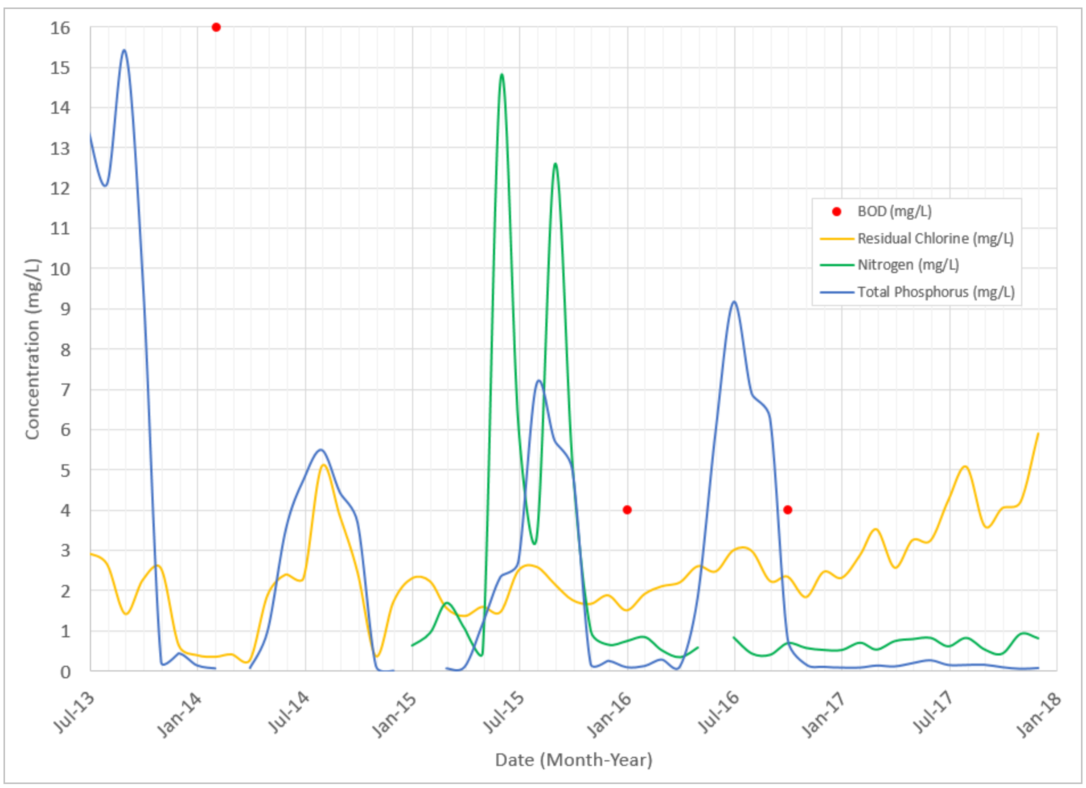


Figure 5-3 Effluent Testing Results

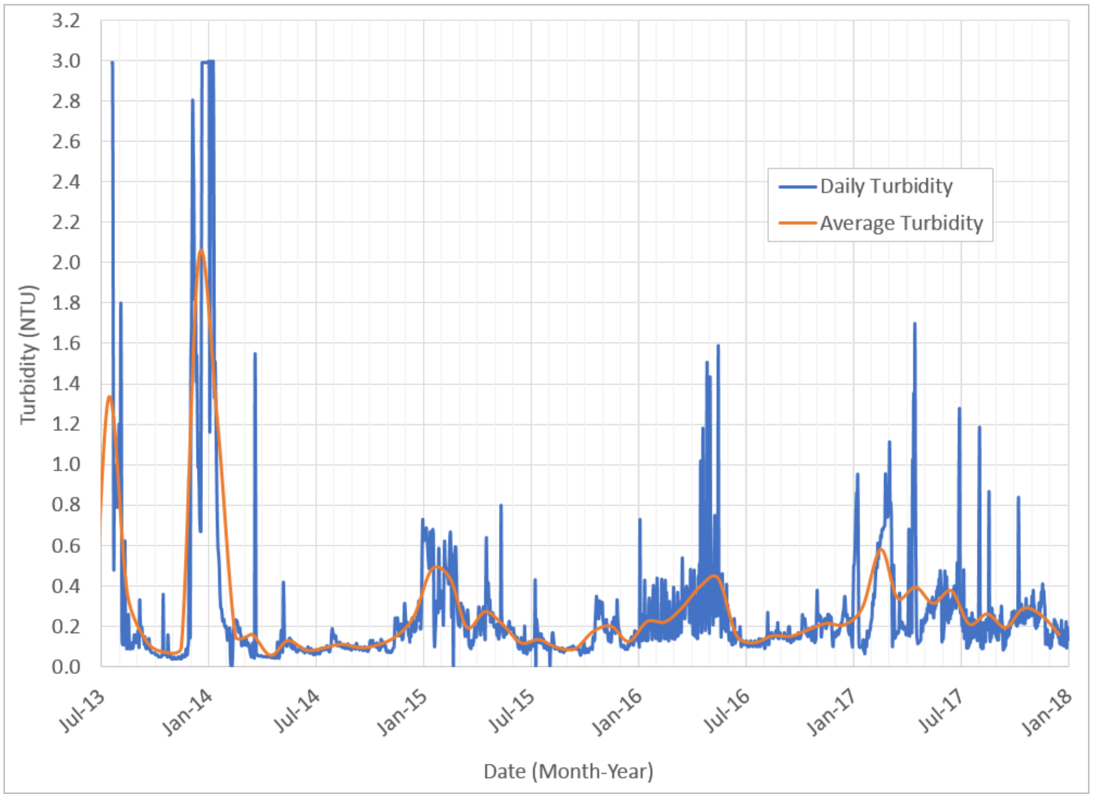


Figure 5-4 Effluent Turbidity

The average monthly effluent requirements for direct discharge to Spring Valley Creek are shown in Table 5-8 as defined in the previous IPDES permit, number ID0028371 that expired on May 2, 2021. The AWRF can meet these limits however the facility is not currently discharging directly to surface water at this time.

Table 5-8 AWRF IPDES effluent criteria

Parameter	Effluent Limits			
	Units	Average Monthly Limits	Average Weekly Limit	Maximum Day Limit
Biological Oxygen Demand (BOD)	mg/L	15	25	-
	lb/day	52	87	-
	% removal	85 min	-	-
Total Suspended Solids (TSS)	mg/L	10	17	-
	lb/day	35	59	-
	% removal	85 min	-	-
pH	s.u	Between 6.5 - 9 at all times		
E. Coli Bacteria	#/100ml	126	-	406
Total Residual Chlorine (TRC)	µg/L	9	-	18
	lb/day	0.03	-	0.06
Total Ammonia (as N)	mg/L	2.4	-	4.7
	lb/day	8	-	17
Total Nitrogen	mg/L	15	-	-
Total Phosphorus (as P)	lb/day	1.2	-	-

5.3.1 Rapid Infiltration Basins (Current)

The rapid infiltration (RI) beds are located northeast of the AWRF in a dry drainage referred to as Broken Horn Draw, as shown in Figure 5-5. Soil and subsoils in this area are coarse sand with high infiltration rates (i.e., over 50 in/hr). Reuse water is surface-applied to a series of flat basins at rates that are below infiltration capacity. This area is designed as dry basins due to the relatively high infiltration rates for Pierce Park Sands.

Figure 5-5 Treatment System & Reuse Map

During non-irrigation seasons, recycled water is pumped to the RI beds where it infiltrates into the aquifer. RI Basins are then discharged into an isolated aquifer that allows in-ground storage of treated wastewater as permitted through DEQ. During the nongrowing season, all effluent is sent to the RI beds.

The current permitted capacity of the RI basin is 0.19 MGD. This rate represents the daily treatment capacity for the planned community. The technical report for Avimor Water Reclamation Company's Reuse Permit M-211-03, produced by Mountain Waterworks dated January 2021 includes an updated analysis of the system's RI basin. The report includes a new RI basin capacity of 0.3 MGD. At this time, the new reuse permit has not been issued by DEQ, but is anticipated to be in 2022. The report states that infiltration testing was performed in the two largest basins in October of 2020 by a materials testing specialist. Based on the testing results, the report proposes an

allowance in the new permit for 36 inches of infiltration per day in the largest basin, and 60 inches per day in remaining basins.
The report also proposes a bed rotation schedule utilizing all 5 RI beds. The rotation occurs by alternating between operational zones A and B. Operational zone A utilizes basins 1 and 2 with a total basin area of 0.37 acres, allowing for a daily infiltration capacity of 0.36 MGD. Operational zone B utilizes basins 3, 4, and 5 with a total basin area of 0.31 acres, allowing for a daily infiltration capacity of 0.3MGD. The lower of the two operational zones' capacity is 0.3MGD in zone A, which will be the limiting factor for the daily inflow rate to RI basins at all times.

5.3.2 Sandy Hill Aquifer

The concept of routing treated effluent from the wastewater treatment plant to the top of Sandy Hill (located West of Highway 55) is being examined. The treated effluent would be injected into the aquifer in effort to recharge the aquifer. The aquifer would then be utilized for collecting water that can be purchased for irrigation purposes. At this time, the concept and process is entering the preliminary design phase. If this option is utilized, Avimor Water Reclamation Company will submit an addendum to this report.

5.3.3 Landscape Irrigation (Current)

Class B reclamation water is being land-applied throughout the community during the growing season to maximizing water use efficiency of fresh water. Fresh and recycled water are blended for irrigation purposes. The primary reuse areas include lawn, landscaped parks, and parkways along roads.

The Class B water is pumped from the ARWF to the reuse water storage tank that is approximately 3,000-feet east of the treatment facility. The water storage tank is connected to a pressure irrigation system that is used throughout the community.

During the irrigation season, the effluent is reused to the maximum extent possible for landscaping from the storage tank. Excess reuse water that cannot be used by the landscape irrigation system overflows the storage tank and is diverted to the RI beds.

5.3.4 Direct Discharge

An Idaho Pollutant Discharge Elimination System (IPDES) permit was granted from Region 10 of the EPA for direct discharged to Spring Valley Creek. Discharge was authorized from October 1 through March 31 to Spring Valley Creek and prohibited between April 1 through September 30. The permit expired on May 2, 2021, though Avimor Water Reclamation Company is in the process of applying for a new IPDES permit through DEQ. If granted a new permit, this discharge may be utilized as Avimor's flows increase. There will be no limit on flow however, maximum limits have been established on nutrient loadings including the time of year that discharge is permitted, October 1st through March 31st.

5.3.4 Modifications to the Reuse Permit

Various reuse options were proposed in the October 16, 2006 Master Plan as reuse alternatives that are not planned for the future reuse permit. These reuse alternatives are riparian restoration and agricultural reuse. The riparian rehabilitation area was established and future use of recycled water for this area has been deemed unnecessary. Agricultural reuse was proposed for approximately 137 acres that would was used for grass hay and sod production but changes in land use planning has resulted in its removal.

Landscape irrigation with recycled water will eventually be replaced by water pumped from the confined aquifer beneath the RI beds. Landscape irrigation is currently conducted with a mixture of potable water and reuse water during the irrigation season. The landscape reuse areas are shown in Appendix J of this report. The plan for future reuse is to discharge treated effluent to the RI beds where it will contribute to the underground aquifer. The treated water will be pumped from the aquifer and used for irrigation. Water will be pumped from the wells adjacent to the RI beds and supply the current recycled water tank. New pipe routing from the AWRF to the RI beds will be necessary to allow for this operational change. The proposed routing is shown in Figure 5-6.

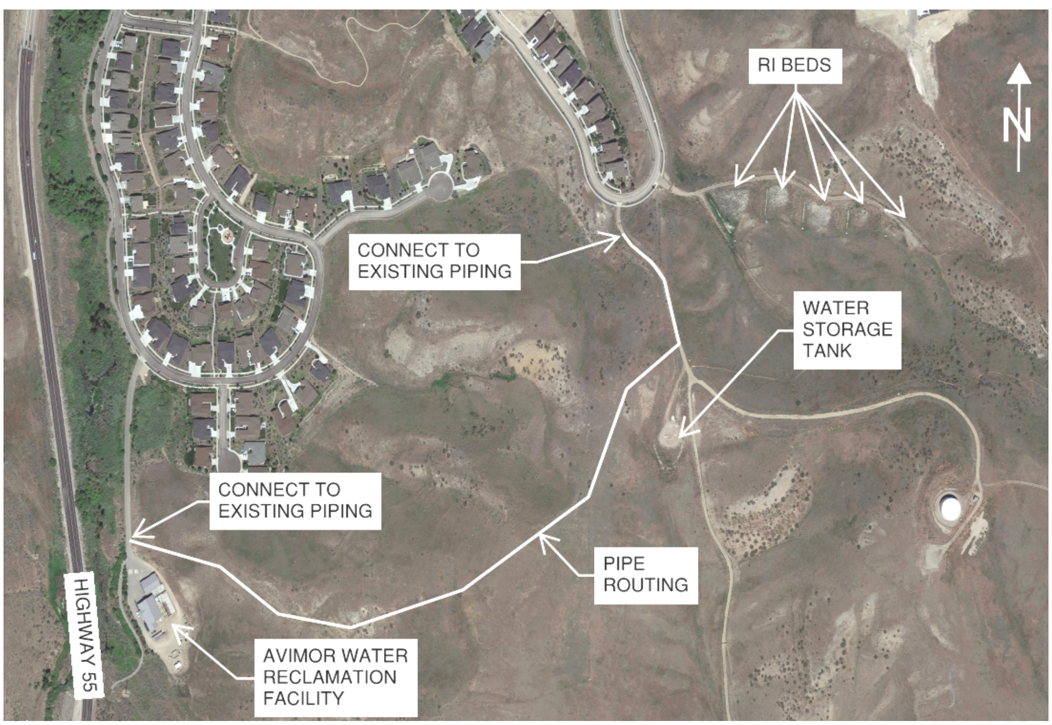


Figure 5-6 Pipe Routing

Liquid that is dewatered from the treatment plant sludge, also known as filtrate, is proposed to be land applied at a proposed tree nursery at the location shown in Figure 5-7. Solids from the AWRF are removed from the MBR tanks and stored in an aerated storage vault. The solids are pumped to the solids handling facility where it is dewatered using a Flo Trend roll-off container. The facility possesses a 1-m gravity belt thickener on site that can be used for dewatering however it is currently non-operational.

The current operation at the treatment facility is to reintroduce the filtrate at the AWRF headworks to be reprocessed. Development of this project requires additional information including the type of trees or plants at the nursery, sampling analysis of the filtrate, and the expected size of the nursery which will be further explored in the project PER and coordination with DEQ.

Figure 5-7 Proposed Nursery Location

Nutrient uptake of trees and plants is based upon type and density. The nursery is currently in the planning stage and this information is currently unknown. For planning purposes, nutrient application rates for non-grazed privately-owned woodlands was used based upon the USDA document titled "Nutrients Available from Livestock Manure Relative to Crop Growth Requirements." The application rates for nitrogen and phosphorus are 100 lbs/acre and 20 lbs/acre respectively, as presented in the USDA document, and was multiplied by 4 since the density of the nursery will be higher than what is observed in woodlands. Table 5-10 provides the acreage necessary to comply with loading rates of nitrogen and phosphorus in the nursery. Filtrate will be trucked over to the nursery as necessary to be applied to the trees and plants. The filtrate will be classified as Class E recycled water which will require buffer zones and fencing to protect the public. Recycled water application at the nursery will only occur during the growing season and reintroduced into the plant headworks in the non-growing season.

Table 5-9 Nursery Loading

Plant Influent MGD	Filtrate MGD	Phosphorus from Filtrate lb/day	Nitrogen from Filtrate lb/day	Acres for Phosphorus(1)	Acres for Nitrogen(1)
0.19	0.0029	3.0	13.5	6.9	6.3
0.33	0.0050	5.2	23.8	12.1	11.0
0.67	0.0100	10.4	47.5	24.1	22.0
1.00	0.0150	15.6	71.3	36.2	33.0
1.33	0.0200	20.9	95.1	48.2	44.0
1.67	0.0250	26.1	118.8	60.3	55.0
2.00	0.0300	31.3	142.6	72.3	66.0

(1) For growing season only - approximately 185 days

Filtrate: 0.015 of Plant Influent, Metcalf & Eddy

Phosphorus - Filtrate Concentration: 125 mg/L, Metcalf & Eddy

Nitrogen - Filtrate Concentration: 570 mg/L, Metcalf & Eddy

Phosphorus - Application Limit: 80 lb/acre/year

Nitrogen - Application Limit: 400 lb/acre/year

5.4 Surface Water Quality - Snake and Boise River TMDLs

Spring Valley Creek eventually drains into Dry Creek which drains into the Boise River. The Lower Boise River Subbasin, hydraulic unit code 17050114, has a total daily maximum load (TMDL) for concentration-based total phosphorus of $\leq$ 0.07 mg/L. This limit is set for the mouth of the Boise River near the City of Parma prior to discharge into the Snake River. Load reductions are required during the critical period established as May 1 through September 30 when dissolved oxygen levels are low.

5.5 Operations Evaluation

Avimor’s wastewater treatment plant is fully operational and functional at this time. There are no known issues that need to be addressed with a capital improvement project. An issue regarding inflow and infiltration into the wastewater system that occurred in 2017 and 2018, but has since been resolved. This issue and corrective action by Avimor is discussed in Section 3.6 of this report. The plant will continue to resolve any minor issues with pumps, valves and consumables related to the MBR with routine maintenance practices. Avimor Water Reclamation Company continues to inspect and maintain operations on the plant to ensure everything remains in compliance.

Conclusion

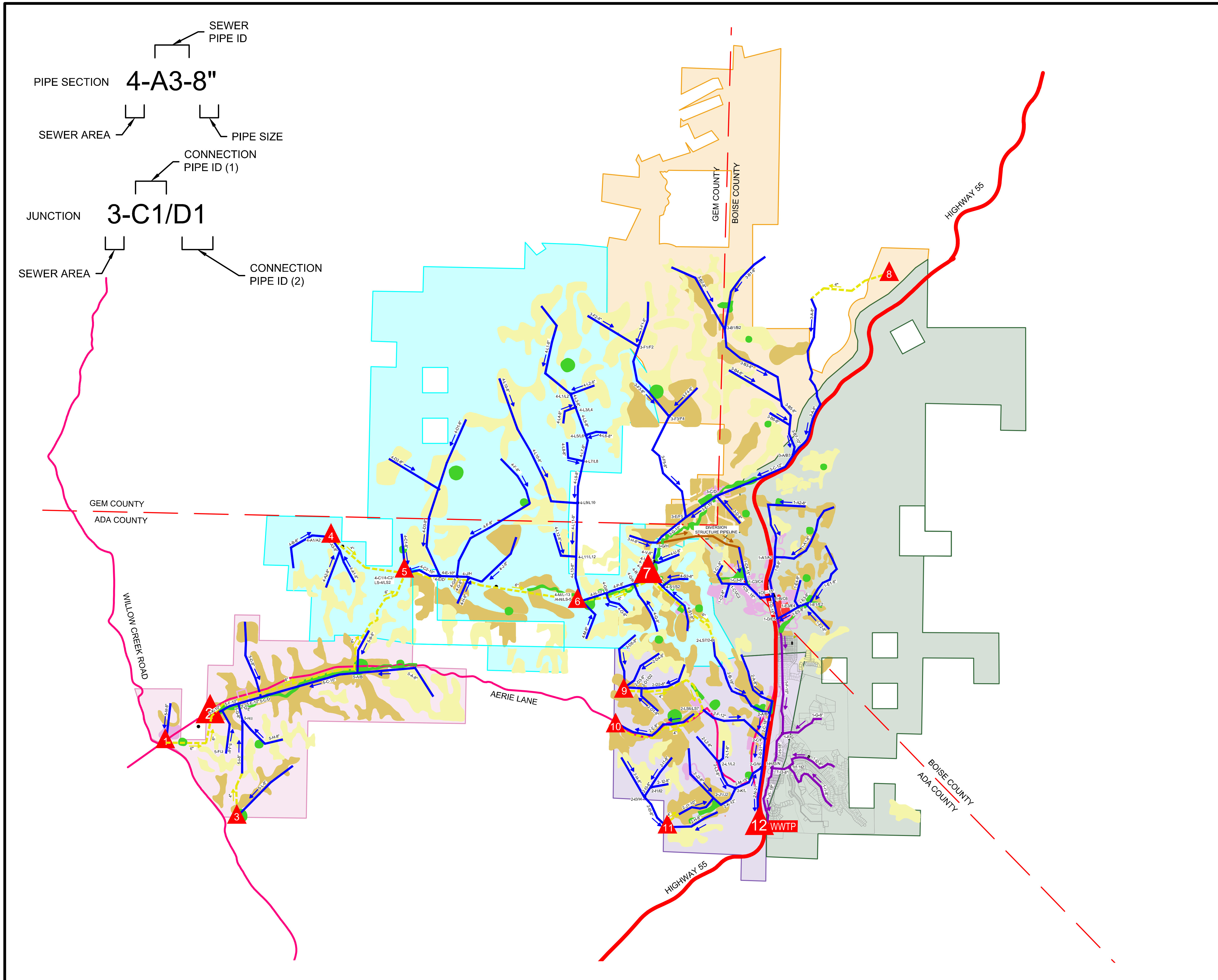
6.1 Considerations

Master planning for a community of this scale is a dynamic process. The assumptions for planned development patterns, density of development, commercial users, and other similar planning figures have been assumed with the understanding that there are a considerable number of unknowns at this time. This master plan is intended to provide guidance to the planning goals of Avimor.

The master plan is a general planning tool intended to highlight planning objectives for the development of Avimor. As demonstrated herein, there is adequate capacity to provide for wastewater collection, treatment, and disposal for the Avimor community. It is the goal of the development team for Avimor to provide adequate planning and construction of infrastructure to serve the community, protect the natural environment, and public health and safety.

Appendix A – Collection System Maps

COVER - SEWER COLLECTION SYSTEM



LEGEND

- MAJOR LIFT STATION
- MINOR LIFT STATION
- WWTP WASTEWATER TREATMENT PLANT
- DIVERSION STRUCTURE PIPELINE
- GRAVITY SEWER
- SEWER FORCE MAIN
- EXISTING GRAVITY SEWER
- ROADWAY
- COUNTY LINE
- SEWER AREA 1
- SEWER AREA 2
- SEWER AREA 3
- SEWER AREA 4
- SEWER AREA 5
- VILLAGE CENTER -VC
- MIXED USE/COMMERCIAL -MUC
- FOOTHILLS RESIDENTIAL - FR
- VILLAGE RESIDENTIAL - VR
- OPEN SPACE - OS

N

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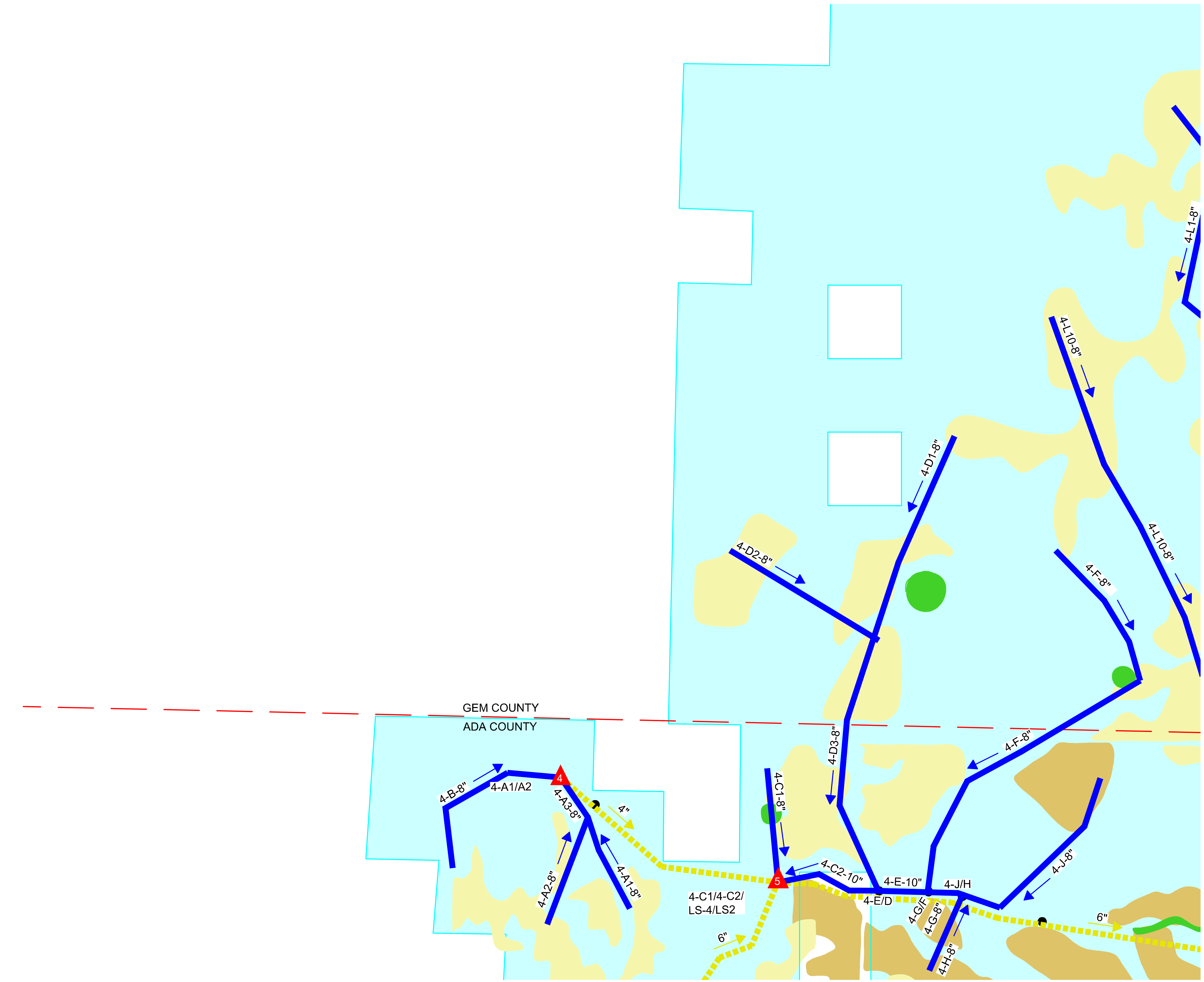
DATE: 4/6/22 JOB: 220063

T-O ENGINEERS
2471 S. TITANIUM PLACE
MERIDIAN, IDAHO 83642
208-323-2288 | WWW.TO-ENGINEERS.COM

EXHIBIT A - SEWER COLLECTION SYSTEM WITH DIVERSION



LEGEND

1	MAJOR LIFT STATION
2	MINOR LIFT STATION
WWTP	WASTEWATER TREATMENT PLANT
	DIVERSION STRUCTURE PIPELINE
	GRAVITY SEWER
	SEWER FORCE MAIN
	EXISTING GRAVITY SEWER
	ROADWAY
	COUNTY LINE
	VILLAGE CENTER -VC
	MIXED USE/COMMERCIAL -MUC
	FOOTHILLS RESIDENTIAL - FR
	VILLAGE RESIDENTIAL - VR
	OPEN SPACE - OS

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2471 S. TITANIUM PLACE
MERIDIAN, IDAHO 83642
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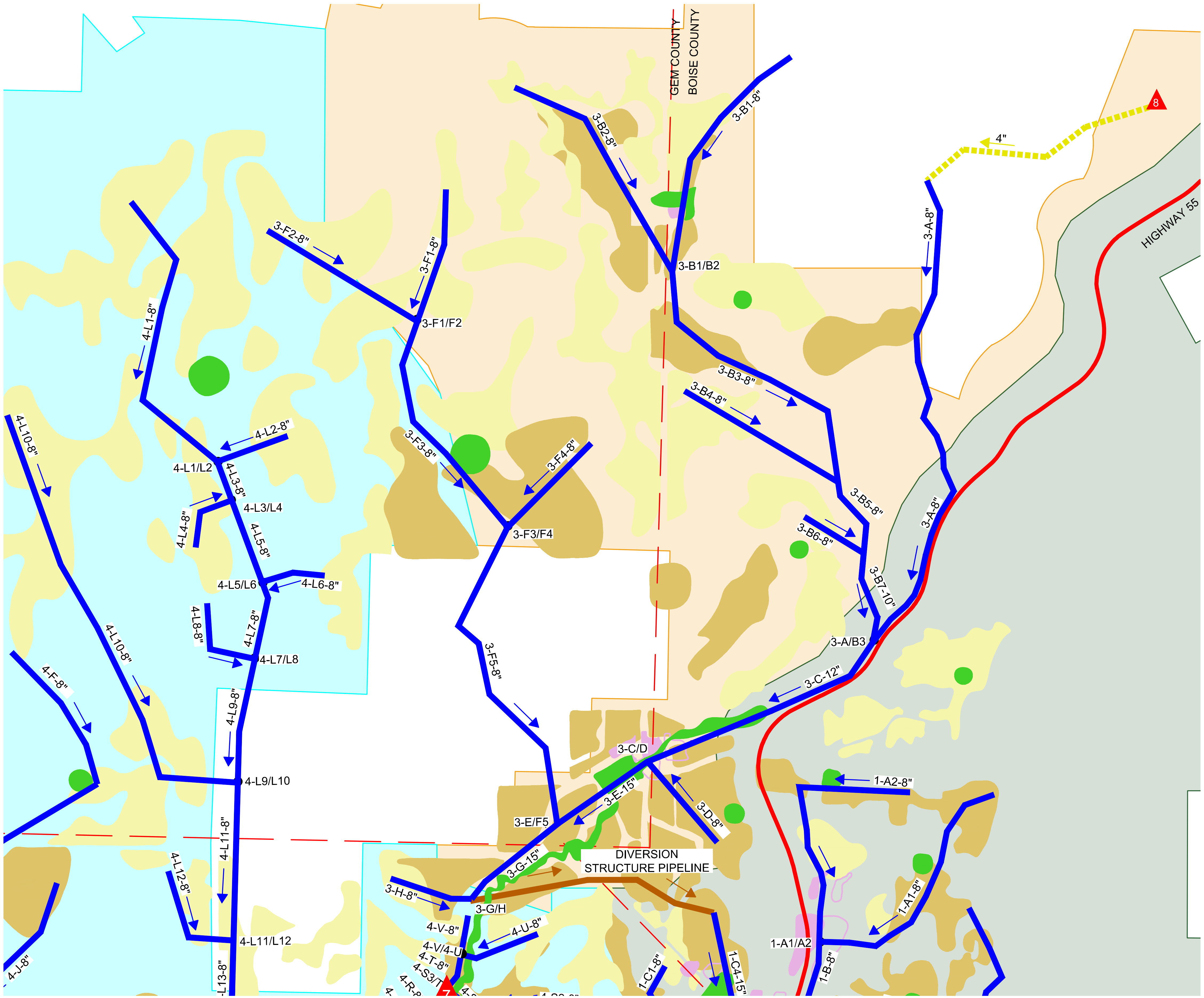
EXHIBIT B - SEWER COLLECTION SYSTEM WITH DIVERSION



LEGEND

- MAJOR LIFT STATION
- MINOR LIFT STATION
- WWTP WASTEWATER TREATMENT PLANT
- DIVERSION STRUCTURE PIPELINE
- GRAVITY SEWER
- SEWER FORCE MAIN
- EXISTING GRAVITY SEWER
- ROADWAY
- COUNTY LINE
- VILLAGE CENTER -VC
- MIXED USE/COMMERCIAL -MUC
- FOOTHILLS RESIDENTIAL - FR
- VILLAGE RESIDENTIAL - VR
- OPEN SPACE - OS

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DATE: 4/6/22 JOB: 220063

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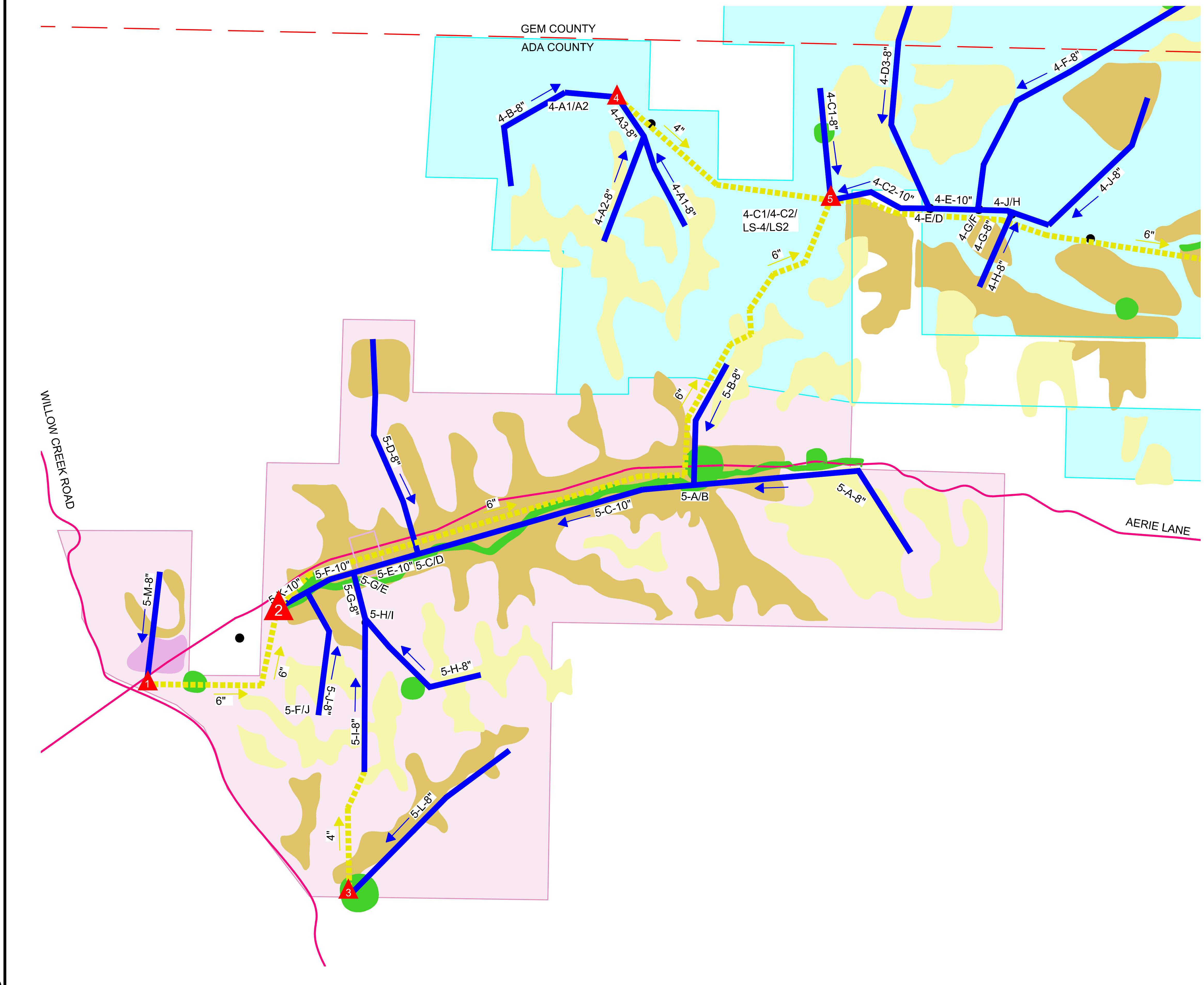
EXHIBIT C - SEWER COLLECTION SYSTEM WITH DIVERSION



LEGEND

- MAJOR LIFT STATION
- MINOR LIFT STATION
- WWTP WASTEWATER TREATMENT PLANT
- DIVERSION STRUCTURE PIPELINE
- GRAVITY SEWER
- SEWER FORCE MAIN
- EXISTING GRAVITY SEWER
- ROADWAY
- COUNTY LINE
- VILLAGE CENTER -VC
- MIXED USE/COMMERCIAL -MUC
- FOOTHILLS RESIDENTIAL - FR
- VILLAGE RESIDENTIAL - VR
- OPEN SPACE - OS

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0 900 1800 3600 5400

DATE: 4/6/22 JOB: 220063

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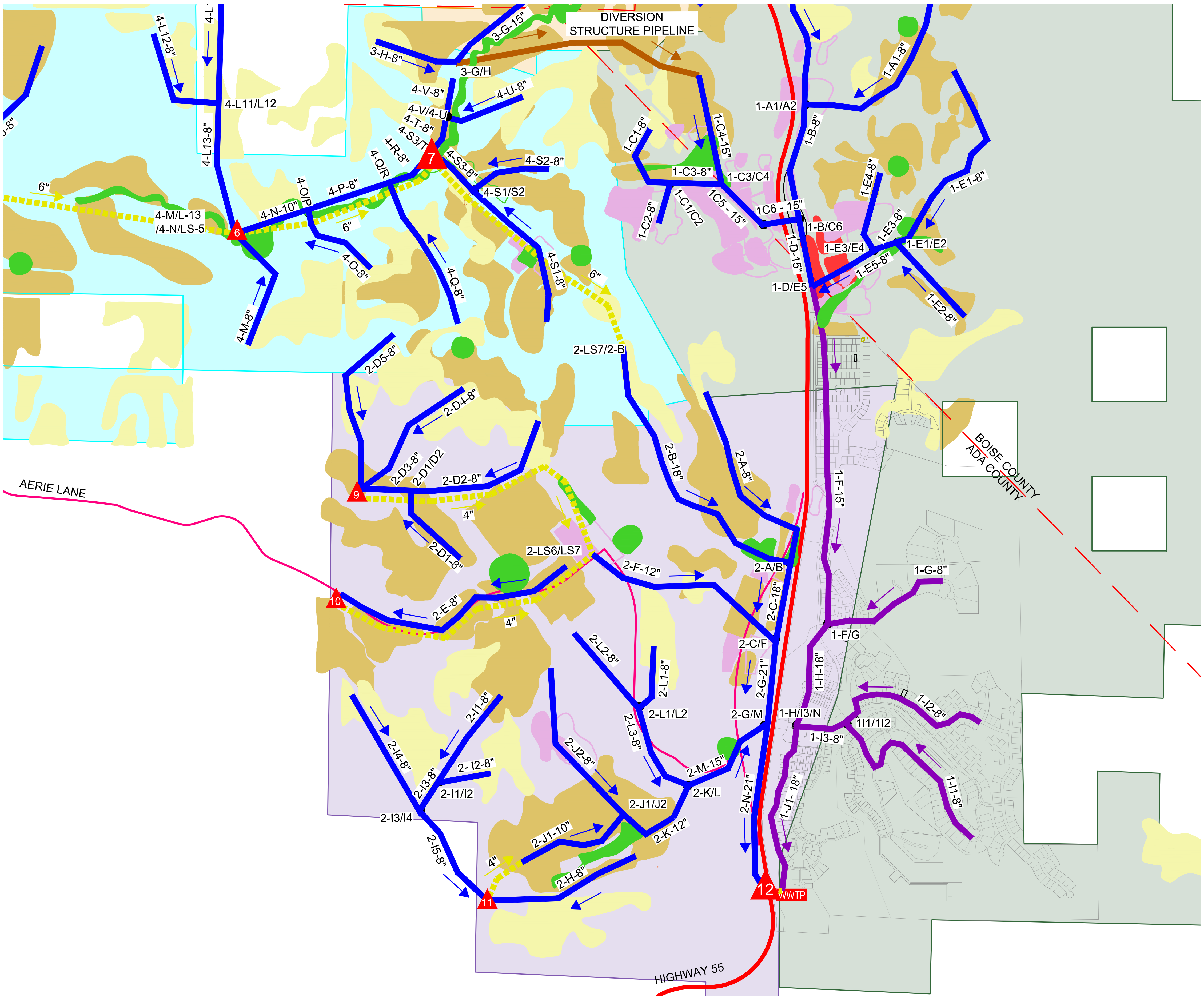
EXHIBIT D - SEWER COLLECTION SYSTEM WITH DIVERSION



LEGEND

- MAJOR LIFT STATION
- MINOR LIFT STATION
- WWTP — WASTEWATER TREATMENT PLANT
- DIVERSION STRUCTURE PIPELINE
- GRAVITY SEWER
- SEWER FORCE MAIN
- EXISTING GRAVITY SEWER
- ROADWAY
- COUNTY LINE
- VILLAGE CENTER -VC
- MIXED USE/COMMERCIAL -MUC
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DATE: 4/6/22 JOB: 220063

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Appendix B – Flow Calculations

HYDRAULIC CALCULATIONS - AREA 1
WITH DIVERSION STRUCTURE

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
1-A1	1B.3 & 1B.4	20147	39
Upstream Flow =		20147	39.2
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		91.2%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
1-A2	1B.4, 1B.5, & 3B.1	28972	56
Upstream Flow =		28972	56.3
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		87.4%	

Junction 1-A1/A2		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		49119	95.5

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
1-B	1B.5	1284	2
Upstream Flow =		50403	98.0
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		78.1%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
1-C1	1-A1	7318	14
Upstream Flow =		7318	14.2
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		96.8%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
1-C2	1-A1	10163	20
Upstream Flow =		10163	19.8
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		95.6%	

Junction 1-C1/C2		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		17481	34.0

Sewer Line 1-C3	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		17481	34.0
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		92.4%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
1-C4	1B-6	48540	94
Upstream Flow =		440579	856.7
Pipe Size (in) = 15			
Remaining Design Capacity Using Minimum Slope =		41.5%	

Junction 1C3-4		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		458060	890.7

Sewer Line 1-C5	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		458060	890.7
Pipe Size (in) = 15			
Remaining Design Capacity Using Minimum Slope =		39.2%	

Sewer Line 1-C6		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		458060	890.7
Pipe Size (in) = 15			
Remaining Design Capacity Using Minimum Slope =		39.2%	

Junction 1-B/C5		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		508463	988.7

HYDRAULIC CALCULATIONS - AREA 1
WITH DIVERSION STRUCTURE

Sewer Line 1-D	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		508463	988.7
Pipe Size (in) = 15			
Remaining Design Capacity Using Minimum Slope =		32.5%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
1-E1	1B.2 & 1B.3	16290	32
Upstream Flow =		16290	31.7
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		92.9%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
1-E2	1-B1, 1B.1a, & 1A.1a	52755	103
Upstream Flow =		52755	102.6
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		77.1%	

Junction 1E1-2		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		69045	134.3

Sewer Line 1-E3		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		69045	134.3
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		70.0%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
1-E4	1B.2	11922	23
Upstream Flow =		11922	23.2
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		94.8%	

Junction 1-E3/E4		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		80966	157.4

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
1-E5	1B.1	**49707**	**96.7**
Upstream Flow =		130673	254.1
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		43.2%	

Junction 1-D/E5		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		639136	1242.8

Sewer Line	Land Use Symbol	ADF (gal/day)	Peak Flow (gal/min)
1-F	FR	1727	3.4
1-F	MF	16627	32.3
1-F	CVC	22982	44.7
1-F	DOS	4337	8.4
1-F	SD	10800	21.0
1-F	202 Homes	32320	62.8
1-F		**88793**	**172.7**
Upstream Flow =		727929	1415.4
Pipe Size (in) =	15		
Existing Slope =	0.0015		
Remaining Pipe Capacity =		3.3%	

Sewer Line	Existing Infrastrure	ADF (gal/day)	Peak Flow (gal/min)
1-G	96 Homes	15360	29.9
1-G		**15360**	**29.9**
Upstream Flow =		15360	29.9
Pipe Size (in) =	8		
Existing Slope =	0.0050		
Remaining Pipe Capacity =		93.3%	

Junction 1-F/G		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		743289	1445.3

Sewer Line	Land Use Symbol	ADF (gal/day)	Peak Flow (gal/min)
1-H	CVC	28191	54.8
1-H	47 Homes	7520	14.6
1-H		**35711**	**69.4**
Upstream Flow =		779001	1514.7
Pipe Size (in) =	18		
Existing Slope =	0.0050		
Remaining Pipe Capacity =		28.9%	

Sewer Line	Existing Infrastrure	ADF (gal/day)	Peak Flow (gal/min)
1-I1	169 Homes	27040	52.6
1-I1		**27040**	**52.6**
Upstream Flow =		27040	52.6
Pipe Size (in) =	8		
Remaining Pipe Capacity =		88.2%	

Sewer Line	Existing Infrastrure	ADF (gal/day)	Peak Flow (gal/min)
1-I2	136 Homes	21760	42.3
1-I2		**21760**	**42.3**
Upstream Flow =		21760	42.3
Pipe Size (in) =	8		
Existing Slope =	0.0060		
Remaining Pipe Capacity =		90.5%	

Junction 1-I1/I2		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		48800	94.9

HYDRAULIC CALCULATIONS - AREA 1
WITH DIVERSION STRUCTURE

Sewer Line 1-I3		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		48800	94.9
Pipe Size (in) =	8		
Remaining Pipe Capacity =		78.8%	

Junction 1-H/I3	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	827801	1609.6

Sewer Line	Land Use Symbol	ADF (gal/day)	Peak Flow (gal/min)
1-J	144 Homes	23040	44.8
1-J	CVC	160	0.3
1-J	Gas Station	1060	2.1
1-J		**24260**	**47.2**
Upstream Flow =		852061	1656.8
Pipe Size (in) =	18		
Existing Slope =	0.0050		
Remaining Pipe Capacity =		22.2%	

HYDRAULIC CALCULATIONS - AREA 2
WITH DIVERSION STRUCTURE

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2-A	1A.6	18469	35.9
Upstream Flow =		18469	35.9
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		92.0%	

Junction 2-LS7/2-B		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		685564	1295.5

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2-B	1A.5 & 1A.6	16886	32.8
Upstream Flow =		702451	1328
Pipe Size (in) = 18			
Remaining Design Capacity Using Minimum Slope =		37.6%	

Junction 2-A/B		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		720920	1364.2

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2-C	1A.7	20429	40
Upstream Flow =		741349	1403.9
Pipe Size (in) = 18			
Remaining Design Capacity Using Minimum Slope =		34.1%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2-D1	1A.10	28205	54.8
Upstream Flow =		28205	54.8
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		87.7%	

HYDRAULIC CALCULATIONS - AREA 2
WITH DIVERSION STRUCTURE

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2-D2	1A.11	23522	45.7
Upstream Flow =		23522	45.7
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		89.8%	

Junction 2-D1/D2		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		51727	100.6

Sewer Line 2-D3		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		51727	100.6
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		77.5%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2-D4	1A.11	23522	45.7
Upstream Flow =		23522	45.7
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		89.8%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2-D5	1A.11	26463	51.5
Upstream Flow =		26463	51.5
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		88.5%	

Lift Station 9		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		101712	197.8

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2-E	1A.9 & 1A.10	34692	67
Upstream Flow =		34692	67.5
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		84.9%	

Lift Station 10	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	34692	67.5

Junction 2-LS9/LS10	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	136404	265.2

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2-F	1A.6 & 1A.9	17788	35
Upstream Flow =		154192	299.8
Pipe Size (in) =	12		
Remaining Design Capacity Using Minimum Slope =		69.3%	

Junction 2-C/F	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	895541	1703.7

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2-G	1A.7	19627	38
Upstream Flow =		915169	1741.9
Pipe Size (in) =	21		
Remaining Design Capacity Using Minimum Slope =		40.6%	

HYDRAULIC CALCULATIONS - AREA 2
WITH DIVERSION STRUCTURE

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2-H	1A.8	32877	64
Upstream Flow =		32877	63.9
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		85.7%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2-I1	1A.9	26964	52.4
Upstream Flow =		26964	52.4
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		88.3%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2-I2	1A.9	10786	21.0
Upstream Flow =		10786	21.0
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		95.3%	

Junction 2-I1/I2	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	37750	73.4

Sewer Line 2-I3		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		37750	73.4
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		83.6%	

HYDRAULIC CALCULATIONS - AREA 2
WITH DIVERSION STRUCTURE

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2-I3	1A.10	15278	30
Upstream Flow =		15278	29.7
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		93.4%	

Junction 2-I3/I4	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	53028	103.1

Sewer Line 2-I5	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	53028	103.1
Pipe Size (in) = 8		
Remaining Design Capacity Using Minimum Slope =	76.9%	

Lift Station 11	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	85904	167.0

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2-J1	1A.8	21918	43
Upstream Flow =		107822	209.7
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		53.1%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2-J2	1A.9	107857	209.7
Upstream Flow =		107857	209.7
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		53.1%	

HYDRAULIC CALCULATIONS - AREA 2
WITH DIVERSION STRUCTURE

Junction 2-J1/J2		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		215679	419.4

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2-K	1A.8	10959	21.3
Upstream Flow =		226638	440.7
Pipe Size (in) = 12			
Remaining Design Capacity Using Minimum Slope =		54.9%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2L-1	1A.9	18336	35.7
Upstream Flow =		18336	35.7
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		92.0%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2L-2	1A.9	14021	27.3
Upstream Flow =		14021	27.3
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		93.9%	

Junction 2-L1/L2		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		32357	62.9

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
2L-3	1A.8	7306	14
Upstream Flow =		39663	77.1
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		82.8%	

Junction 2-K/L		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		266301	517.8

HYDRAULIC CALCULATIONS - AREA 2
WITH DIVERSION STRUCTURE

Sewer Line 2-M		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		266301	517.8
Pipe Size (in) =	15		
Remaining Design Capacity Using Minimum Slope =		64.6%	

Junction 2-G/M	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	1181470	2259.7

Sewer Line 2-N		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		1181470	2259.7
Pipe Size (in) =	21		
Remaining Pipe Capacity =		22.9%	

Lift Station 12	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	1181470	2259.7

HYDRAULIC CALCULATIONS - AREA 3
WITH DIVERSION STRUCTURE

Lift Station 8	Land Use Symbol	ADF (gal/day)	Peak Flow (gal/min)
LS8	NOS	62356	121.2
Upstream Flow =		62356	121.2

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
3-A	3B.4	14108	27.4
Upstream Flow =		76464	148.7
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		66.8%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
3-B1	3B.4	4709	9.2
Upstream Flow =		4709	9.2
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		98.0%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
3-B2	3G.2	40022	77.8
Upstream Flow =		40022	77.8
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		82.6%	

Junction 3-B1/B2		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		44732	87.0

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
3-B3	3B.4	26522	51.6
Upstream Flow =		71254	138.5
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		69.0%	

HYDRAULIC CALCULATIONS - AREA 3
WITH DIVERSION STRUCTURE

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
3-B4	3B.3	9024	17.5
Upstream Flow =		9024	17.5
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		96.1%	

Junction 3-B3/B4	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	80278	156.1

Sewer Line 3-B5	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	80278	156.1
Pipe Size (in) = 8		
Remaining Design Capacity Using Minimum Slope =	65.1%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
3-B6	3B.2	9132	17.8
Upstream Flow =		9132	17.8
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		96.0%	

Junction 3-B5/B6	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	89410	173.9

Sewer Line 3-B7	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	89410	173.9
Pipe Size (in) = 10		
Remaining Design Capacity Using Minimum Slope =	74.4%	

Junction 3-A/B7	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	165873	322.5

HYDRAULIC CALCULATIONS - AREA 3
WITH DIVERSION STRUCTURE

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
3-C	3B.2	**13697**	**26.6**
Upstream Flow =		179571	349.2
Pipe Size (in) =	12		
Remaining Design Capacity Using Minimum Slope =		64.3%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
3-D	1B.7	**56686**	**110.2**
Upstream Flow =		56686	110.2
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		75.4%	

Junction 3-C/D		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		236257	459.4

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
3-E	1G.1	32981	64.1
Upstream Flow =		269238	523.5
Pipe Size (in) =	15		
Remaining Design Capacity Using Minimum Slope =		64.2%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
3-F1	3G.4	**6396**	**12.4**
Upstream Flow =		6396	12.4
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		97.2%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
3-F2	3G.3 & 3G.7	18030	35
Upstream Flow =		18030	35.1
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		92.2%	

HYDRAULIC CALCULATIONS - AREA 3
WITH DIVERSION STRUCTURE

Junction 3-F1/F2		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		24426	47.5

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
3-F3	3G.3	18136	35.3
Upstream Flow =		42562	82.8
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		81.5%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
3-F4	3G.1 & 3G.3	36571	71
Upstream Flow =		36571	71.1
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		84.1%	

Junction 3-F3/F4		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		79133	153.9

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
3-F5	3G.3	**5850**	**11.4**
Upstream Flow =		84983	165.2
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		63.1%	

Junction 3-E/F5		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		354221	688.8

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
3-G	1A.2 & 1G.1	21976	43
Upstream Flow =		376197	731.5
Pipe Size (in) = 15			
Remaining Design Capacity Using Minimum Slope =		50.0%	

HYDRAULIC CALCULATIONS - AREA 3
WITH DIVERSION STRUCTURE

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
3-H	1A.2	**15842**	**31**
Upstream Flow =		15842	30.8
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		93.1%	

Junction 3-G/H	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	392039	762.3

HYDRAULIC CALCULATIONS - AREA 4
WITH DIVERSION STRUCTURE

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-A1	2A.7	2109	4.1
Upstream Flow =		2109	4.1
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		99.1%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-A2	2A.7	7910	15.4
Upstream Flow =		7910	15.4
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		96.6%	

Junction 4-A1/A2		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		10020	19.5

Sewer Line 4-A3		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		10020	19.5
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		95.6%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-B	2A.7	2244	4.4
Upstream Flow =		2244	4.4
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		99.0%	

Lift Station 4		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		12264	23.8

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-C1	2A.12	7423	14.4
Upstream Flow =		19687	38.3
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		91.4%	

HYDRAULIC CALCULATIONS - AREA 4
WITH DIVERSION STRUCTURE

Junction 4-C1/4-C2/LS4/LS2	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	423382	791.3

Lift Station 5	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	423382	791.3

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-J	2A.11 & 3G.6	13633	26.5
Upstream Flow =		13633	26.5
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		94.1%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-H	2A.8	87133	169.4
Upstream Flow =		87133	169.4
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		62.1%	

Junction 4-H/J	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	100766	195.9

Sewer Line 4-G		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		100766	195.9
Pipe Size (in) =	10		
Remaining Design Capacity Using Minimum Slope =		71.1%	

Sewer Line	Devleopment Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-F	3G.6 & 3G.8	5940	11.6
Upstream Flow =		5940	11.6
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		97.4%	

Junction 4-G/F		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		106707	207.5

Sewer Line 4-E		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		106707	207.5
Pipe Size (in) =	10		
Remaining Design Capacity Using Minimum Slope =		69.4%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-D1	3G.8 & 3G.9	23090	44.9
Upstream Flow =		23090	44.9
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		90.0%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-D2	3G.10	5376	10.5
Upstream Flow =		5376	10.5
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		97.7%	

Junction 4-D1/D2		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		28466	55.3

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-D3	3G.10	25569	49.7
Upstream Flow =		54034	105.1
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		76.5%	

Junction 4-E/D3		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		160741	312.6

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-C2	2A.6 & 2A.8A	14499	28.2
Upstream Flow =		175240	340.7
Pipe Size (in) =	10		
Remaining Design Capacity Using Minimum Slope =		49.8%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-L1	3G.5, 3G.7, & 3G.8	16374	32
Upstream Flow =		16374	31.8
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		92.9%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-L2	3G.5	5880	11.4
Upstream Flow =		5880	11.4
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		97.4%	

Junction 4-L1/L2		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		22254	43.3

Sewer Line 4-L3		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		22254	43.3
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		90.3%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-L4	3G.5	2572	5.0
Upstream Flow =		2572	5.0
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		98.9%	

Junction 4-L3/L4		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		24826	48.3

Sewer Line 4-L5		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		24826	48.3
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		89.2%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-L6	3G.3	11115	21.6
Upstream Flow =		11115	21.6
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		95.2%	

HYDRAULIC CALCULATIONS - AREA 4
WITH DIVERSION STRUCTURE

Junction 4-L5/L6		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		35941	69.9

Sewer Line 4-L7		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		35941	69.9
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		84.4%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-L8	3G.5	1654	3.2
Upstream Flow =		1654	3.2
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		99.3%	

Junction 4-L7/L8		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		37595	73.1

Sewer Line 4-L9		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		37595	73.1
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		83.7%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-L10	3G.6 & 3G.8	8875	17.3
Upstream Flow =		8875	17.3
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		96.1%	

Junction 4-L9/L10		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		46470	90.4

Sewer Line 4-L11		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		46470	90.4
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		79.8%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-L12	3G.6 & 2A.11	6075	11.8
Upstream Flow =		6075	11.8
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		97.4%	

Junction 4-L11/L12		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		52545	102.2

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4L-13	2A.9 & 2A.9a	20605	40.1
Upstream Flow =		73150	142.2
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		68.2%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-M	2A.9, 2A.9a, & 2A.10	64222	125
Upstream Flow =		64222	124.9
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		72.1%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-U	1A.3	**13330**	20.2
Upstream Flow =		13330	20.2
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		94.2%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-V	1A.2	9505	18.5
Upstream Flow =		9505	18.5
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		95.9%	
4-V/4-U		(gal/day)	(gal/min)
Upstream Flow =		22835	38.7

Sewer Line 4-T		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		22835	38.7
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		91.3%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-S1	1A.3 & 1A.4	67359	131
Upstream Flow =		67359	131.0
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		70.7%	

HYDRAULIC CALCULATIONS - AREA 4
WITH DIVERSION STRUCTURE

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-S2	1A.3	22603	43.9
Upstream Flow =		22603	43.9
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		90.2%	

Junction 4-S1/S2	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	89962	174.9

Sewer Line 4-S3		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		89962	174.9
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		60.9%	

Junction 4-S3/T	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	112797	213.7

Sewer Line 4-R		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		112797	213.7
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		52.2%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-Q	2A.10	1651	3.2
Upstream Flow =		1651	3.2
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		99.3%	

Junction 4-Q/R	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	114448	216.9

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-P	2A.9a	3109	6.0
Upstream Flow =		117557	222.9
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		50.2%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-O	2A-10	1297	2.5
Upstream Flow =		1297	2.5
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		99.4%	

Junction 4-O/P		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		118854	225.4

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
4-N	2A.9	5956	11.6
Upstream Flow =		124810	237.0
Pipe Size (in) = 10			
Remaining Design Capacity Using Minimum Slope =		65.1%	

Junction 4-M/L-13/4-N/LS-5		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		685564	1295.5

Lift Station 6 4-M/L-13/4-N/LS-5		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		685564	1295.5

Lift Station 7		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		685564	1295.5

HYDRAULIC CALCULATIONS - AREA 5
WITH DIVERSION STRUCTURE

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
5-A	2A.5	52316	101.7
Upstream Flow =		52316	101.7
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		77.3%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
5-B	2A.6	7276	14.1
Upstream Flow =		7276	14.1
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		96.8%	

Junction Node 5-A/B	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	59592	115.9

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
5-C	2A.2	97248	189.1
Upstream Flow =		156839	305.0
Pipe Size (in) =	10		
Remaining Design Capacity Using Minimum Slope =		55.1%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
5-D	2A.2	18060	3.2
Upstream Flow =		18060	3.2
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		99.3%	

Junction Node 5-C/D	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	174900	308.2

HYDRAULIC CALCULATIONS - AREA 5
WITH DIVERSION STRUCTURE

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
5-E	2A.2	6946	13.5
Upstream Flow =		181846	321.7
Pipe Size (in) =	10		
Remaining Design Capacity Using Minimum Slope =		52.6%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
5-L	2A.4	15932	31.0
Upstream Flow =		15932	31.0
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		93.1%	

Lift Station 3		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		15932	31.0

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
5-I	2A.3	3939	7.7
Upstream Flow =		3939	7.7
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		98.3%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
5-H	2A.3	4829	9.4
Upstream Flow =		4829	9.4
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		97.9%	

HYDRAULIC CALCULATIONS - AREA 5
WITH DIVERSION STRUCTURE

Junction Node 5-H/I		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		8769	17.0

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	2A.2	8769	17.0
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		96.2%	

Junction Node 5-G/E		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		190614	338.7

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
5-F	2A.2	6946	13.5
Upstream Flow =		197561	352.2
Pipe Size (in) = 10			
Remaining Design Capacity Using Minimum Slope =		48.1%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
5-J	2A.3	3939	7.7
Upstream Flow =		3939	7.7
Pipe Size (in) = 8			
Remaining Design Capacity Using Minimum Slope =		98.3%	

Junction 5-F/J		ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =		201500	359.9

HYDRAULIC CALCULATIONS - AREA 5
WITH DIVERSION STRUCTURE

Sewer Line	Development Key Nnumber	ADF (gal/day)	Peak Flow (gal/min)
5-K	2A.3	890	1.7
Upstream Flow =		202390	361.6
Pipe Size (in) =	10		
Remaining Design Capacity Using Minimum Slope =		46.7%	

Sewer Line	Development Key Number	ADF (gal/day)	Peak Flow (gal/min)
5-M	2A.1	13802	26.8
Upstream Flow =		13802	26.8
Pipe Size (in) =	8		
Remaining Design Capacity Using Minimum Slope =		94.0%	

Lift Station 1	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	13802	26.8

Lift Station 2	ADF (gal/day)	Peak Flow (gal/min)
Upstream Flow =	216191	388.5

Appendix C – AWRF Flow and Sampling

AWRF Influent Flow

Month - Year	Average Monthly Flow (gal/day)	Maximum Monthly Flow (gal/day)	Total Monthly Flow (gal/day)
Jan-13	-	-	-
Feb-13	-	-	-
Mar-13	-	-	-
Apr-13	-	-	-
May-13	-	-	-
Jun-13	4699	8130	32895
Jul-13	4517	9584	140035
Aug-13	4650	6922	144136
Sep-13	4982	9374	149463
Oct-13	3863	7142	119744
Nov-13	4784	7941	143527
Dec-13	4853	7941	150437
Jan-14	3354	10528	103968
Feb-14	5496	21183	153887
Mar-14	8553	20649	265149
Apr-14	12296	21140	368869
May-14	10694	21909	331518
Jun-14	7708	15875	231247
Jul-14	7533	15512	225986
Aug-14	7112	14583	220469
Sep-14	7487	15808	224619
Oct-14	8969	20295	278027
Nov-14	8518	18786	255546
Dec-14	11837	32231	331424
Jan-15	14911	24733	447334
Feb-15	16439	29960	443866
Mar-15	12775	24066	396029
Apr-15	14008	21949	420239
May-15	18550	23876	575055
Jun-15	15505	22534	465149
Jul-15	12566	20109	376978
Aug-15	12612	16168	378360
Sep-15	13724	17439	411734
Oct-15	21311	30758	660648
Nov-15	20640	32848	619186
Dec-15	21579	36393	668945
Jan-16	20057	42923	621756
Feb-16	26532	42292	769433
Mar-16	28759	49117	891525
Apr-16	31493	60833	944792
May-16	30138	47815	934272
Jun-16	30075	38348	902237
Jul-16	19353	33561	599940

Aug-16	18145	28627	562505
Sep-16	18420	31644	552604
Oct-16	18190	29982	563901
Nov-16	19457	38021	583721
Dec-16	15394	39018	477226
Jan-17	9757	52538	273193
Feb-17	8341	34462	225199
Mar-17	6843	37198	205281
Apr-17	22260	47571	267117
May-17	0	0	0
Jun-17	81460	163037	1629194
Jul-17	37880	45094	1174286
Aug-17	33708	38927	1044943
Sep-17	39629	48402	1188859
Oct-17	34237	43187	1061345
Nov-17	37307	47200	1119196
Dec-17	40516	47806	1256005
Jan-18	39462	48099	1223335
Feb-18	40607	49459	1136986
Mar-18	46491	37198	205281
Apr-18	50502	58412	1515072
May-18	43892	53741	1360640
Jun-18	41386	47759	1241593
Jul-18	45980	53746	1425395
Aug-18	47051	56122	705762

AWRF Effluent Flow

Month - Year	Average Monthly Flow (gal/day)	Maximum Monthly Flow (gal/day)	Total Monthly Flow (gal/day)
Jan-13	-	-	-
Feb-13	-	-	-
Mar-13	-	-	-
Apr-13	-	-	-
May-13	-	-	-
Jun-13	1919	2449	3838
Jul-13	13617	25885	149783
Aug-13	11569	17721	231387
Sep-13	12091	19736	229722
Oct-13	9923	18726	218314
Nov-13	13220	22853	264391
Dec-13	11765	26829	282369
Jan-14	10148	21894	314591
Feb-14	12115	31023	339212
Mar-14	13486	38899	350628
Apr-14	17055	29763	392261
May-14	16961	28125	373149
Jun-14	16055	24359	305042
Jul-14	15576	29700	342681
Aug-14	16633	29008	299401
Sep-14	20575	47270	349781
Oct-14	19489	30882	370293
Nov-14	19469	34826	330966
Dec-14	29198	44017	554757
Jan-15	27031	52594	540612
Feb-15	30967	62387	588365
Mar-15	25946	41435	544868
Apr-15	24920	43538	548230
May-15	23024	56696	575591
Jun-15	20169	32973	524383
Jul-15	23844	33094	548407
Aug-15	22768	33772	500901
Sep-15	25506	33295	561125
Oct-15	24997	42335	599921
Nov-15	28044	59541	588930
Dec-15	32077	70010	737774
Jan-16	28713	78453	660407
Feb-16	34950	82584	733942
Mar-16	32104	79646	770496
Apr-16	35718	82866	714363
May-16	30571	46740	825406
Jun-16	30671	43906	705429

Jul-16	25970	42246	649247
Aug-16	28473	42909	768772
Sep-16	27894	45129	725243
Oct-16	32630	49337	848387
Nov-16	40439	93229	849219
Dec-16	36387	90041	909686
Jan-17	42093	63211	1262791
Feb-17	54627	77560	1529551
Mar-17	57592	68322	1785352
Apr-17	70154	106616	2104621
May-17	63528	81707	1969356
Jun-17	55509	75278	1665273
Jul-17	41225	53062	1277979
Aug-17	36668	47903	1136696
Sep-17	42646	53570	1279383
Oct-17	37498	52618	1162452
Nov-17	38011	48708	1140328
Dec-17	40277	51085	1248596

AWRF Influent BOD_5

Month - Year	BOD5 (mg/L)
Jan-13	-
Feb-13	-
Mar-13	-
Apr-13	-
May-13	-
Jun-13	-
Jul-13	139
Aug-13	<61
Sep-13	198
Oct-13	353 & 417
Nov-13	-
Dec-13	158
Jan-14	73
Feb-14	398
Mar-14	257
Apr-14	259
May-14	91
Jun-14	-
Jul-14	93
Aug-14	126
Sep-14	226
Oct-14	254
Nov-14	227
Dec-14	290
Jan-15	136
Feb-15	116
Mar-15	159
Apr-15	176
May-15	167
Jun-15	273
Jul-15	264
Aug-15	200
Sep-15	218
Oct-15	266
Nov-15	286
Dec-15	375
Jan-16	862
Feb-16	279
Mar-16	328
Apr-16	275
May-16	186
Jun-16	218
Jul-16	202

Aug-16	198
Sep-16	168
Oct-16	156
Nov-16	104
Dec-16	233
Jan-17	810
Feb-17	359
Mar-17	667
Apr-17	183
May-17	164
Jun-17	182
Jul-17	179
Aug-17	296
Sep-17	264
Oct-17	242
Nov-17	227
Dec-17	169

AWRF Effluent Sampling & Analysis

Month - Year	BOD (mg/L)	COD (mg/L)	TKN (mg/L)	Nitrate (mg/L)	Ammonia (mg/L)	Total P (mg/L)	Free Chlorine (mg/L)	Total Coliform (MPN/100 mL)	Average Monthly Turbidity (NTU)	Max Monthly Turbidity (NTU)
Jan-13	-	-	-	-	-	-	-	-	-	-
Feb-13	-	-	-	-	-	-	-	-	-	-
Mar-13	-	-	-	-	-	-	-	-	-	-
Apr-13	-	-	-	-	-	-	-	-	-	-
May-13	-	-	-	-	-	-	-	-	-	-
Jun-13	-	-	-	-	-	-	-	-	-	-
Jul-13	< 3	< 20.0	0.64	31.30	< 0.04	13.40	2.92	< 2	1.33	2.99
Aug-13	< 3	< 20.0	0.96	46.60	< 0.04	12.10	2.65	< 2	0.36	1.80
Sep-13	< 3	< 20.0	1.70	48.60	< 0.04	15.40	1.41	< 2	0.13	0.33
Oct-13	< 3	20.7	1.05	27.95	< 0.04	9.69	2.26	< 2	0.07	0.36
Nov-13	-	-	0.47	2.92	-	0.22	2.54	< 2	0.09	0.97
Dec-13	< 3	43.8	14.75	0.79	11.40	0.44	0.63	< 2	2.03	2.99
Jan-14	< 5	64.5	6.09	0.38	9.30	0.14	0.38	< 2	1.23	3.00
Feb-14	16	58.2	3.33	0.03	3.60	0.06	0.34	< 2	0.15	0.23
Mar-14	< 3	34.2	12.60	< 0.02	11.4	< 0.05	0.41	< 2	0.16	1.52
Apr-14	< 3	25.4	5.11	0.09	8.18	0.07	0.27	< 2	0.05	0.12
May-14	< 3	< 20.0	1.00	2.84	< 0.04	0.99	1.88	< 2	0.13	0.41
Jun-14	< 3	< 20.0	0.66	2.68	< 0.04	3.52	2.40	< 2	0.08	0.10
Jul-14	< 3	< 20.0	0.75	3.44	< 0.04	4.75	2.31	< 2	0.09	0.19
Aug-14	< 3	< 20.0	0.85	3.30	< 0.04	5.49	5.09	< 2	0.11	0.17
Sep-14	< 3	< 20.0	0.52	3.13	0.08	4.43	3.84	< 2	0.09	0.12
Oct-14	< 3	< 20.0	0.35	2.67	< 0.04	3.68	2.44	< 2	0.11	0.14
Nov-14	< 3	29.7	0.59	4.87	< 0.04	0.12	0.36	< 2	0.15	0.31
Dec-14	-	-	-	-	-	-	1.72	< 2	0.25	0.73
Jan-15	< 3	< 20.0	0.84	1.09	< 0.04	< 0.05	2.30	< 2	0.48	0.69
Feb-15	< 3	< 20.0	0.44	1.55	< 0.04	< 0.05	2.23	< 2	0.43	0.67
Mar-15	< 3	< 20.0	0.41	2.10	< 0.04	0.06	1.56	< 2	0.19	0.32
Apr-15	< 3	< 20.0	0.70	2.43	< 0.04	0.10	1.36	< 2	0.27	0.64
May-15	< 3	< 20.0	0.58	3.99	< 0.04	1.15	1.59	< 2	0.21	0.80
Jun-15	< 3	< 20.0	0.53	3.47	< 0.04	2.34	1.46	< 2	0.12	0.20
Jul-15	< 3	< 20.0	0.53	2.70	< 0.04	2.70	2.49	< 2	0.13	0.43
Aug-15	< 3	< 20.0	0.71	4.89	< 0.04	7.12	2.59	< 2	0.09	0.17
Sep-15	< 3	< 20.0	0.54	3.16	< 0.04	5.72	2.15	< 2	0.09	0.10
Oct-15	< 3	< 20.0	0.75	11.90	< 0.04	5.00	1.76	< 2	0.17	0.35
Nov-15	< 3	< 20.0	0.80	1.89	< 0.04	0.18	1.66	< 2	0.20	0.33
Dec-15	< 3	< 20.0	0.83	5.38	< 0.04	0.25	1.87	< 2	0.12	0.18
Jan-16	4	-	0.62	2.20	< 0.04	0.09	1.50	< 2	0.22	0.73
Feb-16	< 3	-	0.83	6.65	< 0.04	0.12	1.91	< 2	0.22	0.44
Mar-16	< 3	-	0.54	3.72	< 0.04	0.28	2.10	< 2	0.29	0.54
Apr-16	< 3	-	0.44	4.03	< 0.04	0.11	2.20	< 2	0.40	1.50
May-16	< 3	-	0.93	3.82	< 0.04	1.82	2.60	< 2	0.43	1.59

Jun-16	< 3	-	0.82	3.75	< 0.04	6.01	2.47	< 2	0.15	0.31
Jul-16	< 3	-	0.97	4.59	0.17	9.17	3.00	< 2	0.12	0.14
Aug-16	< 3	-	0.59	4.53	< 0.04	6.89	2.98	< 2	0.15	0.27
Sep-16	< 3	-	0.67	5.29	< 0.04	6.28	2.23	< 2	0.15	0.20
Oct-16	4	-	0.71	5.03	< 0.04	0.79	2.34	< 2	0.18	0.38
Nov-16	< 3	-	0.68	6.82	< 0.04	0.16	1.83	< 2	0.21	0.26
Dec-16	< 3	-	0.70	3.45	< 0.04	0.10	2.46	< 2	0.21	0.35
Jan-17	< 3	< 20.0	1.55	4.34	1.99	0.08	2.31	< 2	0.29	0.95
Feb-17	< 3	< 20.0	0.63	3.91	< 0.04	0.08	2.88	< 2	0.58	0.95
Mar-17	< 3	< 20.0	0.67	3.30	0.06	0.13	3.52	< 2	0.34	1.11
Apr-17	< 3	< 20.0	0.45	2.67	< 0.04	0.11	2.56	< 2	0.39	1.70
May-17	< 3	< 20.0	0.45	5.01	< 0.04	0.19	3.25	< 2	0.31	0.48
Jun-17	< 3	< 20.0	0.69	5.68	0.59	0.26	3.24	< 2	0.37	1.25
Jul-17	< 3	< 20.0	0.66	3.66	0.08	0.15	4.24	< 2	0.21	0.48
Aug-17	< 3	< 20.0	0.41	6.99	< 0.04	0.15	5.07	< 2	0.26	1.17
Sep-17	< 3	< 20.0	0.49	5.93	< 0.04	0.15	3.60	< 2	0.19	0.30
Oct-17	< 3	< 20.0	0.55	6.05	< 0.04	0.09	4.05	< 2	0.29	0.84
Nov-17	< 3	< 20.0	0.55	5.83	< 0.04	0.05	4.20	< 2	0.25	0.41
Dec-17	< 3	< 20.0	0.55	5.22	< 0.04	0.07	5.90	< 2	0.16	0.24

Surface Water Sampling & Analysis Results

Serial Number	Location	Date	Ammonia (mg/L)	Nitrate-nitrite (mg/L)	TKN (mg/L)	T-P (mg/L)	Ortho-P (mg/L)	Chloride	TSS (mg/L)	TDS (mg/L)	Temp (C)	PH	DO (mg/l)	Flow (cfs)
SW-021101	WQ-1	22-Apr-2013	<0.04	0.02	0.25	0.17	0.13	11.00	-	-	10.2	6.95	9.9	4.00
		12-Nov-2013	<0.04	0.55	0.26	0.18	0.15	18.00	-	-	7.7	7.65	9.1	0.20
		17-Apr-2014	<0.04	0.27	0.32	0.19	0.15	6.00	-	-	10.3	7.74	10.2	4.00
		20-Nov-2014	<0.04	0.44	0.21	0.26	0.18	21.00	-	-	3.8	7.46	9.6	0.50
		21-Apr-2015	0.83	0.17	13.00	<0.04	0.29	0.15	< 3	-	9.4	7.69	9.6	98.00
		24-Nov-2015	0.28	0.40	26.00	<0.04	0.34	0.19	< 3	-	3.9	7.20	6.7	30.20
		6-Apr-2016	<0.04	0.04	0.30	0.16	0.14	7.00	ND	-	9.3	7.90	9.6	31.10
		9-Nov-2016	<0.04	0.03	0.40	0.39	0.27	33.00	7	-	7.7	7.83	6.3	0.53
		26-Apr-2017	<0.04	0.30	0.45	0.23	0.18	10.00	19	-	9.5	9.63	N/A	N/A
		25-Oct-2017	<0.04	0.12	0.70	0.45	0.25	44.00	17		8.4	7.64	N/A	N/A
SW-021102	WQ-2	22-Apr-2013	<0.04	0.15	0.29	0.10	0.07	3.00	-	-	10.0	6.90	8.3	2.00
		12-Nov-2013	<0.04	2.67	0.18	0.19	0.20	6.00	-	-	9.9	6.88	4.6	0.01
		17-Apr-2014	<0.04	0.35	0.35	0.11	0.09	1.00	-	-	10.2	7.31	8.4	1.00
		20-Nov-2014	<0.04	1.85	0.37	0.23	0.21	7.00	-	-	6.4	6.99	7.9	0.00
		21-Apr-2015	0.45	0.14	2.00	<0.04	0.20	0.11	< 3	-	10.4	7.23	8.3	93.00
		24-Nov-2015	Dry	-	-	-	-	-	-	-	-	-	-	-
		6-Apr-2016	<0.04	0.02	0.30	0.07	0.08	2.00	ND	-	9.7	8.04	9.4	19.40
		9-Nov-2016	<0.04	2.03	0.37	0.27	0.23	9.00	ND	-	9.6	7.92	6.5	0.42
		26-Apr-2017	<0.04	0.02	0.41	0.17	0.12	1.00	26	-	9.2	8.24	N/A	N/A
		25-Oct-2017	<0.04	3.39	0.22	0.19	0.19	9.00	< 3	-	11.9	7.51	N/A	N/A
SW-021103	WQ-3	22-Apr-2013	Dry	-	-	-	-	-	-	-	-	-	-	-
		12-Nov-2013	Dry	-	-	-	-	-	-	-	-	-	-	-
		17-Apr-2014	Dry	-	-	-	-	-	-	-	-	-	-	-
		20-Nov-2014	Dry	-	-	-	-	-	-	-	-	-	-	-
		21-Apr-2015	Dry	-	-	-	-	-	-	-	-	-	-	-
		24-Nov-2015	Dry	-	-	-	-	-	-	-	-	-	-	-
		6-Apr-2016	Dry	-	-	-	-	-	-	-	-	-	-	-
		9-Nov-2016	Dry	-	-	-	-	-	-	-	-	-	-	-
		26-Apr-2017	Dry	-	-	-	-	-	-	-	-	-	-	-
		25-Oct-2017	Dry	-	-	-	-	-	-	-	-	-	-	-
SW-021104	WQ-4	22-Apr-2013	Dry	-	-	-	-	-	-	-	-	-	-	-
		12-Nov-2013	Dry	-	-	-	-	-	-	-	-	-	-	-
		17-Apr-2014	Dry	-	-	-	-	-	-	-	-	-	-	-
		20-Nov-2014	Dry	-	-	-	-	-	-	-	-	-	-	-
		21-Apr-2015	Dry	-	-	-	-	-	-	-	-	-	-	-
		24-Nov-2015	Dry	-	-	-	-	-	-	-	-	-	-	-
		6-Apr-2016	Dry	-	-	-	-	-	-	-	-	-	-	-
		9-Nov-2016	Dry	-	-	-	-	-	-	-	-	-	-	-
		26-Apr-2017	Dry	-	-	-	-	-	-	-	-	-	-	-
		25-Oct-2017	Dry	-	-	-	-	-	-	-	-	-	-	-