Supplemental Material
Item E.2. Sewer

SEWER UTILITY MASTER PLAN

Incline Village General Improvement District

March 2024

Prepared for:

INCLINE VILLAGE
GENERAL IMPROVEMENT DISTRICT
ONE DISTRICT – ONE TEAM

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Sewer Utility Master Plan

Prepared for:

Incline Village General Improvement District

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EXECUTIVE SUMMARY

This sewer system utility master plan (Plan) documents system trends and capacity, infrastructure and facility condition and performance, and provides a plan for near and long-term capital improvement and replacement needs. This executive summary provides a snapshot of the key findings from each section of the Plan. In total, the Plan is comprised of six sections detailing the collection, treatment, and effluent management components of the sewer system.

SECTION 1.0 – HISTORICAL, CURRENT, AND FUTURE FLOWS

Incline Village General Improvement District currently provides sewer service to 4,191 residential (i.e., single-family, and multi-family) and commercial customers within its service area. From 2018 to 2023 the annual average daily flow in millions of gallons per day ranges from 0.81 (2022) to 0.97 (2019) with a 6-year average of 0.90 as measured at the Water Resource Recovery Facility. A unique trend in the monthly average daily flow shows how the transient/seasonal population affects the daily collection system flows. As seen in Figure ES-1, the average daily flow increases in the winter months from December through February due to tourism and individuals utilizing vacation homes for the ski season while March and April see spikes related to inflow and infiltration of stormwater, groundwater, or snow melt runoff into the sewer system

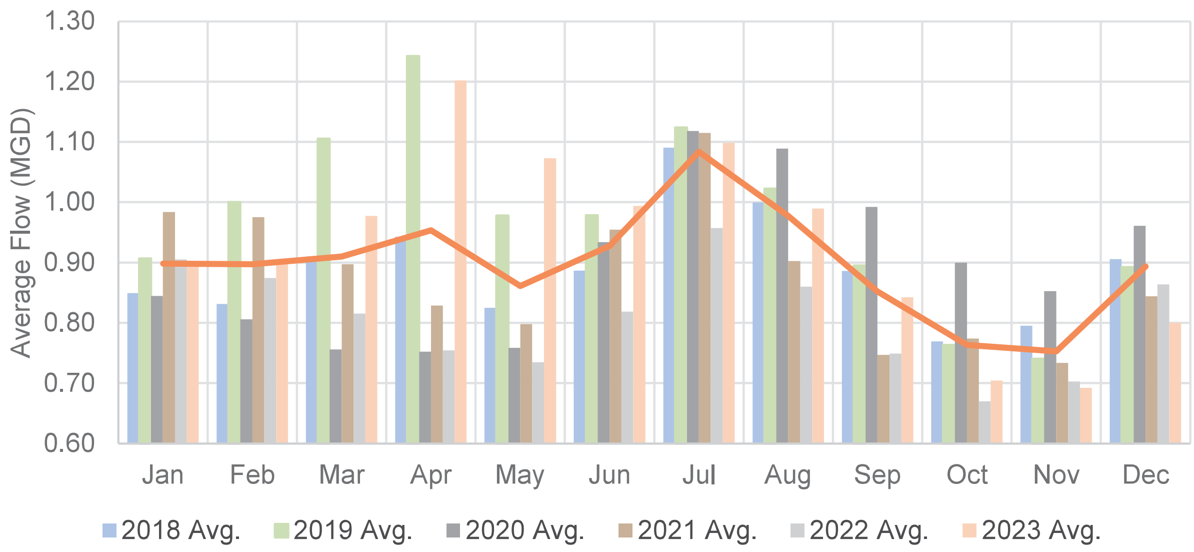

Figure ES-1: Average Daily Flow Summary, 2018 – 2023

Incline Village General Improvement District provided hourly historical flow totals at the Water Resource Recovery Facility from January 2022 through January 2023. A diurnal curve was computed for the system by averaging the hourly flowrate for each individual timestep (1 am, 2 am, etc.). Figure ES-2 presents the average day diurnal curve calculated for the system. During an average day, peak flows seen at the Water Resource Recovery Facility are approximately 1.5 times the average daily flow rate.

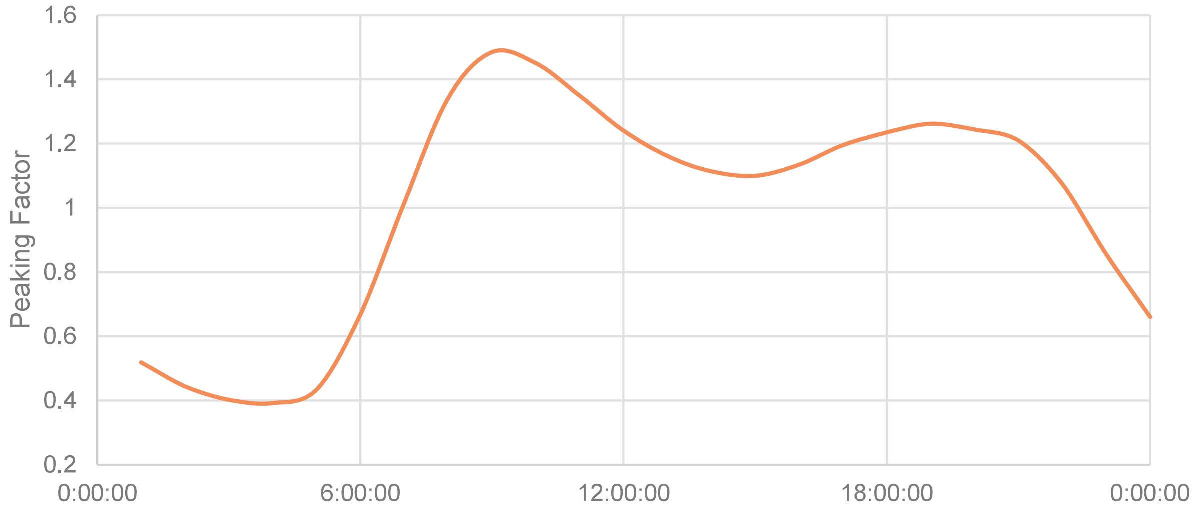

Figure ES-2: Average Day Diurnal Curve

To determine an overall peaking factor for the system, two different methodologies were used. Table ES-1 gives a summary of the peaking factors calculated for the system using the two different methods.

Table ES-1: Peaking Factor Summary

PF Description	Average Flow	Peak Flow	Peaking Factor
Daily Peaking Factor x Hourly Peaking Factor			
Daily Peaking Factor	0.90 MGD	1.62 MGD	1.8
Hourly Peaking Factor	0.034 MGH	0.05 MGH	1.5
System Peaking Factor	n/a	n/a	2.7
Peak Instantaneous Hourly Peaking Factor			
System Peaking Factor	0.034 MGH	0.10 MGH	3.1

The buildout projections presented in Section 1.4 indicate that wastewater flows within the sewer system may increase by approximately 6% based on the current average and peak daily flow rate. The Plan area is expected to see limited development of vacant parcels in the near to long-term future. It is expected that sewer flows will remain near their current values with small variations as Incline Village and Crystal Bay grows. Table ES-2 summarizes the existing sewer flows and potential buildout demands for the Plan area.

Table ES-2: Sewer Flow Summary

Flow Scenario	Average Daily Flow (MGD)	Peak Daily Flow (MGD)
Existing System	0.90	2.41
Additional Flow during Buildout	0.05	0.13
Total Buildout System	**0.95**	**2.54**

SECTION 2.0 – CONDITION ASSESSMENT AND RISK ANALYSIS

The sewer collection system is comprised of infrastructure ranging from 10 to 70 years old. The system consists of approximately 97 miles of sewer mains, 1,840 manholes, 19 sewer pump stations, and 11 miles of force main. Sewer mains range in size from 6 to 24-inches in diameter and consist of polyvinyl chloride , ductile iron, asbestos cement pipe, steel, vitrified clay pipe, high density polyethylene, or cured-in-place pipe. One key discovery made during the preparation of the master plan was that National Association of Sewer Service Companies Pipeline Assessment Certification Program condition assessment scoring data did not exist for the pipelines, manholes, or laterals so the project team developed a risk evaluation matrix based on previous risk assessment studies.

This Plan developed a matrix that consists of several categories and weighting factors unique to the Incline Village General Improvement District sewer collection system, land uses, infrastructure location, and operations. From these categories and weighting factors, a risk score for each pipe and manhole was determined. These scores are used to estimate relative risk throughout the system and act as an instrument to develop a capital improvement plan.

Sewer mains estimated to have a higher relative risk are concentrated along the western segment of Northwood Boulevard and Southwood Boulevard. Additionally, sections along Crystal Rocks Drive that runs parallel to Lakeshore Boulevard and a section of Jupiter Drive fall within the medium to high-risk range. The District has specified that these lines undergo cleaning in both the spring and fall seasons every year. It is important to note that the presence of significant structural defects in these pipes is currently unknown.

Typically, only manholes that have received condition assessments are selected for assessment. Since the condition of the manholes is unknown at this time, all manholes were analyzed based on their age, proximity to waterways, land use, and operator input. The District has identified a cluster of manholes located along a private road south of Lakeshore Boulevard as having issues with inflow and infiltration, resulting in the highest risk rating. Manholes falling within the medium-high risk category are situated close to waterways or storm drains.

An overall risk template approach for determining the consequence and likelihood risk scores for the District's sewer collection system has been developed with this Plan. As the District collects additional information, it will be able to incorporate and modify the parameter ranges of risk categories. As the data for the entire sewer collection system is incorporated into the risk assessment template, it will be possible for the District to score the consequences and likelihood of risks confidently and accurately.

Risk is only one of several parameters used when evaluating the sewer utility for prioritized reconstruction and/or rehabilitation. The results of this risk assessment should be used in conjunction with the operator input, rehabilitation technologies, project cost, and utility sewer planning objectives prior to initiating an asset replacement program. As the Incline Village General Improvement District collects condition assessment data in the future it will be able to incorporate and modify the parameter ranges of risk categories. As condition data for the entire sewer collection system is incorporated into the risk assessment template, it will be possible to score the consequences and likelihood of failure more confidently and accurately.

SECTION 3.0 – SYSTEM OVERVIEW AND CAPACITY ANALYSIS

The collection system conveys sewer through a network of gravity sewer mains and pump stations to a terminal collection point at the Water Resource Recovery Facility. Multiple sewersheds pump into one another, creating a step ladder of pumping and gravity flow conveying sewerage to the Water Resource Recovery Facility. Ultimately, there are three influent sources into the Water Resource Recovery Facility.

Due to the mountainous terrain in the sewer service area, the collection system utilizes a number of lift stations in order to convey sewer to the Water Resource Recovery Facility. Of the 19 lift stations in the system, SPS-1 is the largest and it collects sewer from 17 of the 20 system sewersheds. The lift stations are split between underground vaults and above ground structures and range from 25 gallons per minute to 1,000 gallons per minute. 13 lift stations have permanent access to an emergency power source.

The capacity of the gravity collection system was assessed against Washoe County Community Services Department – Gravity Sewer Collection Design Standards. Table ES-3 summarizes the maximum ratio of depth of flow to pipe diameter and the remaining capacity of the gravity sewer system, as reported by the hydraulic model.

Table ES-3: Existing Max d/D Summary

Sewershed ID	Max d/D	Minimum Remaining Capacity (EDUs)
WRRF	0.38	265
SPS-1	0.58	**-57**
SPS-2	0.31	257
SPS-4	0.02	1,515
SPS-5	0.23	219
SPS-6	n/a	n/a
SPS-7	0.28	470
SPS-8	0.41	**-16**
SPS-9	**1.00**	709
SPS-10	0.20	516
SPS-11	n/a	n/a
SPS-12	0.14	692
SPS-13	n/a	n/a
SPS-14A	0.04	232
SPS-14B	n/a	n/a
SPS-15	0.07	808
SPS-18	n/a	n/a
SPS-19	n/a	n/a

The capacity analysis for the lift stations and pressurized sewer looks at the three main components of each lift station: pumps, wet wells, and force mains. Table ES-4, Table ES-5, and Table ES-6 list the remaining capacity for each component respectively, with the capacity expressed in Equivalent Dwelling Units. Additionally, the lift stations equipped with backup power have been marked with an asterisk in Table ES-5.

Table ES-4: Existing Lift Station Pump Capacity Summary

SPS ID	Pump Operating Point (gpm)	Peak Flow (gpm)	Capacity Remaining (EDUs)
SPS-1	1,000	813.7	1,503
SPS-2	300	41.8	1,355
SPS-4	280	2.1	1,459
SPS-5	78	9.2	361
SPS-6	80	2.3	408
SPS-7	700	190.4	2,675
SPS-8	1,000	640.2	1,889
SPS-9	50	1.8	253
SPS-10	460	106.8	1,854
SPS-11	80	1.3	413
SPS-12	900	35.8 or 235.8	4,536 or 3,486
SPS-13	200	8.8	1,004
SPS-14A	55	4.9	263
SPS-14B	55	4.9 or 59.9	263 or **-26**
SPS-15	150	16.7	700
SPS-18	25	3.1	115
SPS-19	30	0.1	157

Table ES-5: Existing Lift Station Emergency Storage Capacity Summary

SPS ID	Required Emergency Storage (gal)	Emergency Storage Available (gal)	Capacity Remaining (EDUs)
SPS-1*	127,163	54,995	**-3,157**
SPS-2*	6,533	5,863	**-29**
SPS-4*	321	431	5
SPS-5*	1,444	2,508	47
SPS-6*	365	162	**-9**
SPS-7*	29,750	9,889	**-869**
SPS-8	100,041	2,084	**-4,285**
SPS-9*	273	1,454	52
SPS-10*	16,683	7,862	**-386**
SPS-11	209	536	14
SPS-12*	5,600	5,407	**-8**
SPS-13*	1,376	2,424	46
SPS-14A*	770	1,819	46
SPS-14B*	770	1,707	41
SPS-15*	2,604	7,101	197
SPS-18	478	212	**-12**
SPS-19	14	196	8

Table ES-6: Existing Force Main Capacity Summary

SPS ID	Force Main Velocity (ft/s)	Maximum Flow Rate (gpm)	Capacity Remaining (EDUs)
SPS-1: 10-inch	4.5	1,958	4,506
SPS-1: 14-inch	2.3	3,838	14,374
SPS-2	3.4	705	2,126
SPS-4	3.2	705	2,231
SPS-5	2.0	313	1,235
SPS-6	2.0	313	1,225
SPS-7	2.9	1,958	6,605
SPS-8	1.3	6,345	28,057
SPS-9	0.6	705	3,438
SPS-10	2.9	1,253	4,164
SPS-11	2.0	313	1,225
SPS-12	3.7	1,958	5,556
SPS-13	2.3	705	2,651
SPS-14A	1.4	313	1,356
SPS-14B	1.4	313	1,356
SPS-15	0.6	1,958	9,492
SPS-18	0.6	313	1,514
SPS-19	3.06	78	254
SR28 Combined Force Main	2.7	6,345	22,285
SR28 Combined Force Main	6.7	6,345	5,488
SR28/Lakeshore Blvd Combined Force Main	4.6	1,958	4,323
SR28/Lakeshore Blvd Combined Force Main	9.3	1,958	**-1,635**

SECTION 4.0 – WATER RESOURCE RECOVERY FACILITY (WRRF)

This condition assessment includes evaluations to identify the current condition, capacity, performance issues and safety status of the Water Resource Recovery Facility components along with the capability to meet the District's needs into the near and long-term future. The facility has undergone multiple expansions and modifications since construction of the original mechanical plant in 1962 developed by the Crystal Bay Development Company. The current treatment process train includes:

- Raw influent flow metering
- Fine screening
- Grit removal
- Secondary biological treatment
- Clarification and disinfection along with waste sludge digestion
- Mechanical sludge dewatering

The organic concentrations of the influent are representative of higher strength wastewater with low influence of groundwater infiltration or surface water inflow mixing with the domestic wastewater. Original design values for the plant biochemical oxygen demand and total suspended solids were approximately 240 mg/L and 264 mg/L, respectively, however the observed values are approximately double the facility design criteria. From 2020 through 2022 these values were exceeded every month of the year as seen in Figure ES-3 and Figure ES-4.

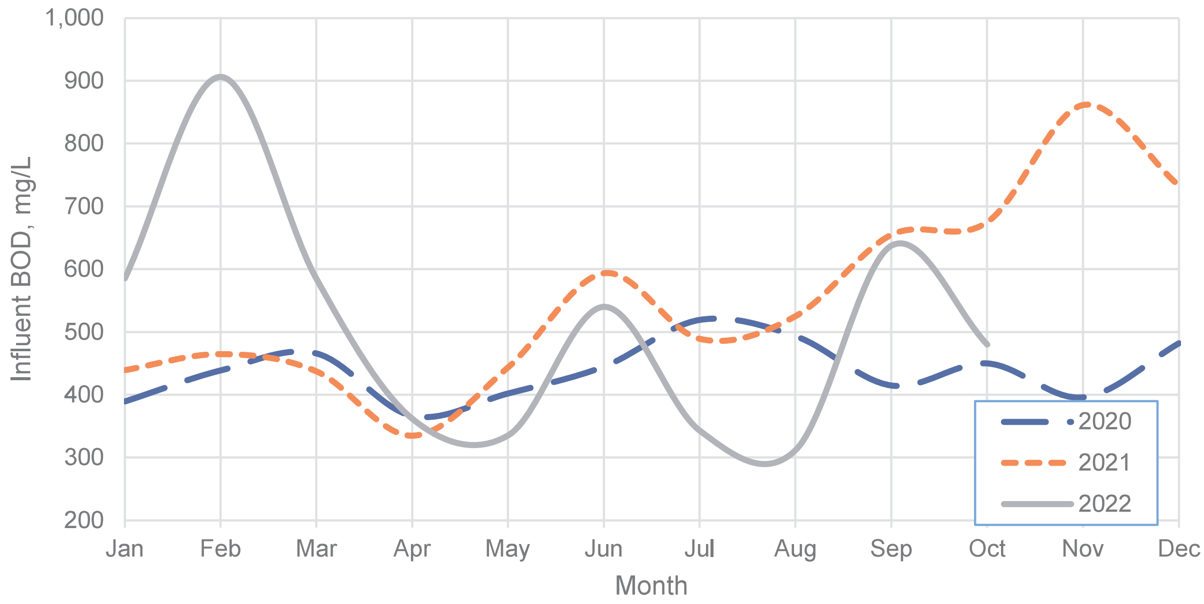

Figure ES-3: Ave. Monthly Influent BOD_5; (2020 – 2022)

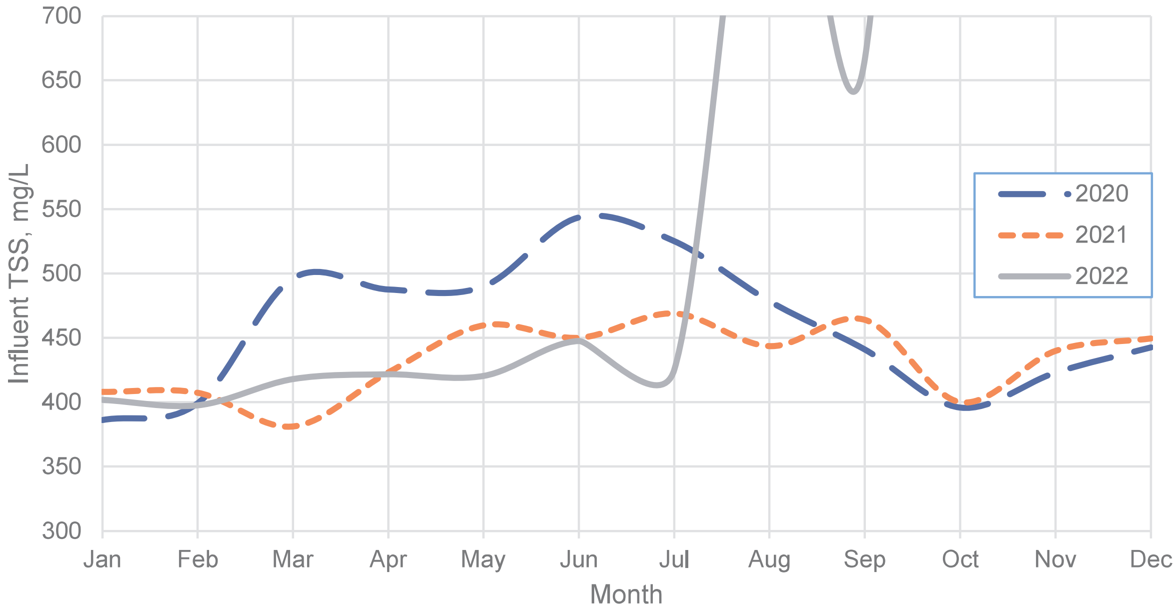

Figure ES-4: Ave. Monthly Influent TSS (2020 – 2022)

The higher concentrations are indicative of very low sewer collection system infiltration and overall conservative water use. Typically, these much higher influent loading values would present significant treatment challenges for a treatment facility if the influent flows were also approaching the design flow capacity. However, the recent influent flows are substantially less than expected with a hydraulic design average day of 3.0 million gallons per day versus 0.87 million gallons per day observed average daily flows. Therefore, though the organic concentration is substantially higher, the total daily organic loading in total pounds per day is still well below the assumed design loading. The Water Resource Recovery Facility has historically met all discharge permit limits at all three outfall locations.

A condition assessment of the Water Resource Recovery Facility was also conducted as part of the Plan. The Plan details the purpose and equipment found in each unit process at the facility and identifies any deficiencies and provides recommendations for additional study and/or improvement. Structural and electrical elements were also included in the condition assessment. Overall, eight capital improvement projects are recommended for implementation by fiscal year 2032 to further study or resolve existing deficiencies and increase facility resiliency.

The secondary treated effluent from the Incline Village Water Resource Recovery Facility is pumped out of the Lake Tahoe Basin via a 21-mile pipeline over the Sierra Nevada Mountains to the Carson Valley. The treated effluent discharges to the Wetlands Enhancement Facility near Hot Springs Mountain in northern Douglas County. Irrigation use with the effluent also occurs at the Schneider Ranch and the Clear Creek Tahoe Country Club. The Plan also evaluated the wetland facility and had no recommendations for improvements.

SECTION 5.0 – EFFLUENT EXPORT SYSTEM

The effluent export system begins at the Water Resource Recovery Facility and consists of a storage tank, pump station, surge tank, and approximately 23 miles of transmission main. Most recently, the Incline Village General Improvement District has made significant investments in the replacement of high-pressure sections of the transmission pipeline with an expected project completion at the end of the 2026 construction season. The Spooner Pump Station has a maximum design point of 2,800 gallons per minute at a total dynamic head of 1,500-feet. The pump station is equipped with a closed surge tank to mitigate transient events which is assumed to be near or in a state of failure. It is recommended that an engineering analysis be conducted in the near-term to identify a replacement device. The existing tank can be removed once the new pressure surge protection measures are in place.

SECTION 6.0 – CAPITAL IMPROVEMENT PROGRAM

In general, the sewer system is in good condition and has adequate capacity both now and into the future. The findings and recommendations of the master plan have been compiled into 15 improvement projects and/or annual maintenance/investigative programs which will yield the District a more robust and resilient sewer system. The 10-year capital improvement program can be found in Table ES-7 and Table ES-8. The 11 to 20-year program can be found in Table ES-9. The 10-year program totals $35.4 million while the 11 to 20-year program is currently estimated at $48.5 million.

It is recommended that this master plan be updated at least once every ten years so that the capital improvement program is representative of system needs.

Table ES-7: Year 1-5 Capital Improvement Program

Project	Type	FY25	FY26	FY27	FY28	FY29
New Effluent Storage Tank	WRRF	$7,172,900				
Aeration System Improvements	WRRF	$3,858,900				
Sewer System SCADA Master Plan	Study/Planning	$91,600				
Existing Sewer Video Scoring	Inspection	$424,500				
SPS-16 Surge Protection BDR	Study/Planning	$70,100				
Sewer Pump Station Condition Assessment and BDR	Study/Planning	$163,800				
Gravity Sewer Main Investigation	Study/Planning	$16,200				
Effluent Storage Tank Rehabilitation	WRRF		$1,217,000			
Headworks Improvements	WRRF		$49,000			
Headworks Second Screen BDR	Study/Planning		$33,600			
Gravity Sewer CCTV & Manhole Inspection Program	Inspection		$440,700			
Sewer Pump Station Rehabilitation Program	Repair/Replacement		$279,600			
Sewer System SCADA Upgrades	Repair/Replacement		$111,900			
SPS-16 Surge Protection Improvements	Repair/Replacement		$838,800			
Gravity Sewer CCTV & Manhole Inspection Program	Inspection			$457,400		
Sewer Pump Station Rehabilitation Program	Repair/Replacement			$290,300		
Sewer System SCADA Upgrades	Repair/Replacement			$116,100		
Headworks Second Screen Installation	WRRF			$694,300		
Aeration Basin Lining	WRRF			$2,579,000		
Gravity Sewer CCTV & Manhole Inspection Program	Inspection				$474,800	
Sewer Pump Station Rehabilitation Program	Repair/Replacement				$301,300	
Sewer System SCADA Upgrades	Repair/Replacement				$120,500	
Aerated Sludge Holding Tanks	WRRF				$347,700	
Gravity Sewer CCTV & Manhole Inspection Program	Inspection					$492,800
Sewer Pump Station Rehabilitation Program	Repair/Replacement					$312,700
Sludge Dewatering Improvements	WRRF					$1,649,200
Yearly Total		**$11,798,000**	**$2,970,600**	**$4,137,100**	**$1,244,300**	**$2,454,700**

Table ES-8: Year 6-10 Capital Improvement Program

Project	Type	FY30	FY31	FY32	FY33	FY34
Gravity Sewer CCTV & Manhole Inspection Program	Inspection	$511,600				
Sewer Pump Station Rehabilitation Program	Repair/Replacement	$324,600				
Secondary Clarifier Mechanism Replacement	WRRF	$2,447,400				
Onsite Hypochlorite System BDR	Study/Planning	$97,400				
Gravity Sewer CCTV & Manhole Inspection Program	Inspection		$531,000			
Sewer Pump Station Rehabilitation Program	Repair/Replacement		$337,000			
Onsite Hypochlorite System Installation	WRRF		$1,090,300			
Sewer Pump Station Rehabilitation Program	Repair/Replacement			$349,800		
Sewer System Condition Assessment	Study/Planning			$105,000		
Sewer Pump Station Rehabilitation Program	Repair/Replacement				$363,100	
Gravity System Replacement and Rehab Program	Repair/Replacement				$2,904,100	
Sewer Pump Station Rehabilitation Program	Repair/Replacement					$376,900
Sewer Master Plan Update	Study/Planning					$339,200
Gravity System Replacement and Rehab Program	Repair/Replacement					$3,014,500
Yearly Total		**$3,381,000**	**$1,958,300**	**$454,800**	**$3,267,200**	**$3,730,600**

Table ES-9: Year 11-20 Capital Improvement Program

Project	Type	FY35	FY36	FY37	FY38	FY39	FY40	FY41	FY42	FY43	FY44
Gravity Sewer CCTV & Manhole Inspection Program	Inspection	$554,800									
Sewer Pump Station Rehabilitation Program	Repair/Replacement	$391,200									
Gravity System Replacement and Rehab Program	Repair/Replacement	$3,129,000									
Gravity Sewer CCTV & Manhole Inspection Program	Inspection		$575,900								
Sewer Pump Station Rehabilitation Program	Repair/Replacement		$406,000								
Gravity System Replacement and Rehab Program	Repair/Replacement		$3,247,900								
Gravity Sewer CCTV & Manhole Inspection Program	Inspection			$597,800							
Sewer Pump Station Rehabilitation Program	Repair/Replacement			$421,500							
Gravity System Replacement and Rehab Program	Repair/Replacement			$3,371,300							
Gravity Sewer CCTV & Manhole Inspection Program	Inspection				$620,500						
Sewer Pump Station Rehabilitation Program	Repair/Replacement				$437,500						
Gravity System Replacement and Rehab Program	Repair/Replacement				$3,499,400						
Gravity Sewer CCTV & Manhole Inspection Program	Inspection					$644,000					
Sewer Pump Station Rehabilitation Program	Repair/Replacement					$454,100					
Gravity System Replacement and Rehab Program	Repair/Replacement					$3,632,400					
Gravity Sewer CCTV & Manhole Inspection Program	Inspection						$668,500				
Sewer Pump Station Rehabilitation Program	Repair/Replacement						$471,300				
Gravity System Replacement and Rehab Program	Repair/Replacement						$3,770,400				
Gravity Sewer CCTV & Manhole Inspection Program	Inspection							$693,900			
Sewer Pump Station Rehabilitation Program	Repair/Replacement							$489,300			
Gravity System Replacement and Rehab Program	Repair/Replacement							$3,913,700			
Gravity Sewer CCTV & Manhole Inspection Program	Inspection								$720,300		
Sewer Pump Station Rehabilitation Program	Repair/Replacement								$507,800		
Gravity System Replacement and Rehab Program	Repair/Replacement								$4,062,400		
Gravity Sewer CCTV & Manhole Inspection Program	Inspection									$747,600	
Sewer Pump Station Rehabilitation Program	Repair/Replacement									$527,100	
Gravity System Replacement and Rehab Program	Repair/Replacement									$4,216,800	
Gravity Sewer CCTV & Manhole Inspection Program	Inspection										$776,100
Sewer Pump Station Rehabilitation Program	Repair/Replacement										$547,200
Gravity System Replacement and Rehab Program	Repair/Replacement										$4,377,000
Yearly Total		**$4,075,000**	**$4,229,800**	**$4,390,600**	**$4,557,400**	**$4,730,500**	**$4,910,200**	**$5,096,900**	**$5,290,500**	**$5,491,500**	**$5,700,300**

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1.0 HISTORICAL, CURRENT, AND FUTURE FLOWS

1.1 Customer Profile

Incline Village General Improvement District (IVGID) operates and maintains the sanitary sewer system serving the Incline Village and Crystal Bay areas. The following sections present the analysis of the wastewater flows and assumptions which will form the basis of the Sewer Master Plan (Plan).

IVGID currently provides service to 4,191 customers within its service area. However, many of the sewer customers are made up of several buildings or units. A breakdown of current sewer customers and their number of units by land use can be found in Table 1. As shown, the system is made up of single-family and multi-family residential customers (making up over 94% of all customers and 97% of all units), and commercial customers making up the rest.

Table 1: Sewer Customer Land Use Summary

Land Use Type	Customer Counts	Units
Commercial	232	232
Single-Family Residential	3,701	3,701
Multi-Family Residential	258	4,083

While the majority of utility customers are residential, the area is not expecting large permanent population growth or development. As identified within the 2021 Washoe County Tahoe Area Plan, adopted by the County and the Tahoe Regional Planning Agency (TRPA), the vast majority of the vacant lots within IVGID are owned by public agencies and will be preserved from development. Therefore, it was assumed that the privately owned vacant parcels would be developed in the buildout scenario whereas the publicly owned vacant parcels would be left undeveloped. A breakdown of vacant lot counts by ownership type can be found in Table 2. Of the 1,239 vacant lots within the IVGID service area, 1,012 are publicly owned. The remaining 227 lots were considered for the buildout demand scenario. This would represent a customer increase of only 5.3% from the existing customer counts.

Table 2: Vacant Land Use Summary

Land Use Type	Publicly Owned	Privately Owned
Vacant, other, or unknown	2	2
Vacant, under development	0	3
Vacant, single family	1,001	185
Vacant, multi-residential	2	1
Vacant, commercial	8	36

1.2 System Flows

The collection system conveys wastewater flow through a network of gravity sewer mains and pump stations to a terminal collection point at the WRRF. As wastewater flows throughout the Plan area are not metered, daily historical flow totals at the inlet to the WRRF from January 2018 through December 2023 were used to determine system flow characteristics. Table 3 provides the average daily flow (ADF) in millions of gallons per day (MGD) for each month from 2018 to 2023.

Table 3: WRRF Average Daily Flow Summary 2018 – 2023 (MGD)

Month	2018	2019	2020	2021	2022	2023	Average
January	0.85	0.91	0.84	0.98	0.91	0.90	0.90
February	0.83	1.00	0.81	0.98	0.87	0.90	0.90
March	0.91	1.11	0.76	0.90	0.82	0.98	0.91
April	0.94	1.24	0.75	0.83	0.75	1.20	0.95
May	0.82	0.98	0.76	0.80	0.73	1.07	0.86
June	0.89	0.98	0.93	0.95	0.82	0.99	0.93
July	1.09	1.12	1.12	1.11	0.96	1.10	1.08
August	1.00	1.02	1.09	0.90	0.86	0.99	0.98
September	0.89	0.90	0.99	0.75	0.75	0.84	0.85
October	0.77	0.76	0.90	0.77	0.67	0.70	0.76
November	0.80	0.74	0.85	0.73	0.70	0.69	0.75
December	0.91	0.89	0.96	0.84	0.86	0.80	0.88
Yearly Average	**0.89**	**0.97**	**0.90**	**0.88**	**0.81**	**0.93**	**0.90**

A unique trend in the monthly ADF shows how the transient/seasonal population affects the daily collection system flows. As seen in Figure 1, the ADF increases in the winter months from December through February due to tourism and individuals using vacation homes for the ski season. March and April see spikes in sewer flows when tourism would be expected to dip with the end of ski season. This spike may be the result of inflow and infiltration (I&I) of stormwater, groundwater, or snow melt runoff into the sewer system. As weather improves, sewer flows also increase due to more people visiting the Tahoe Basin for recreation. As a result, July sees peak sewer flows in the system.

Of particular note when comparing the month-to-month flow data over the previous six years, is the higher-than-average sewer flows in 2020 and 2021 during the summer months. This increase in flow was primarily due to the COVID-19 pandemic, and the record low vacancy rates in the Tahoe basin during that time period.

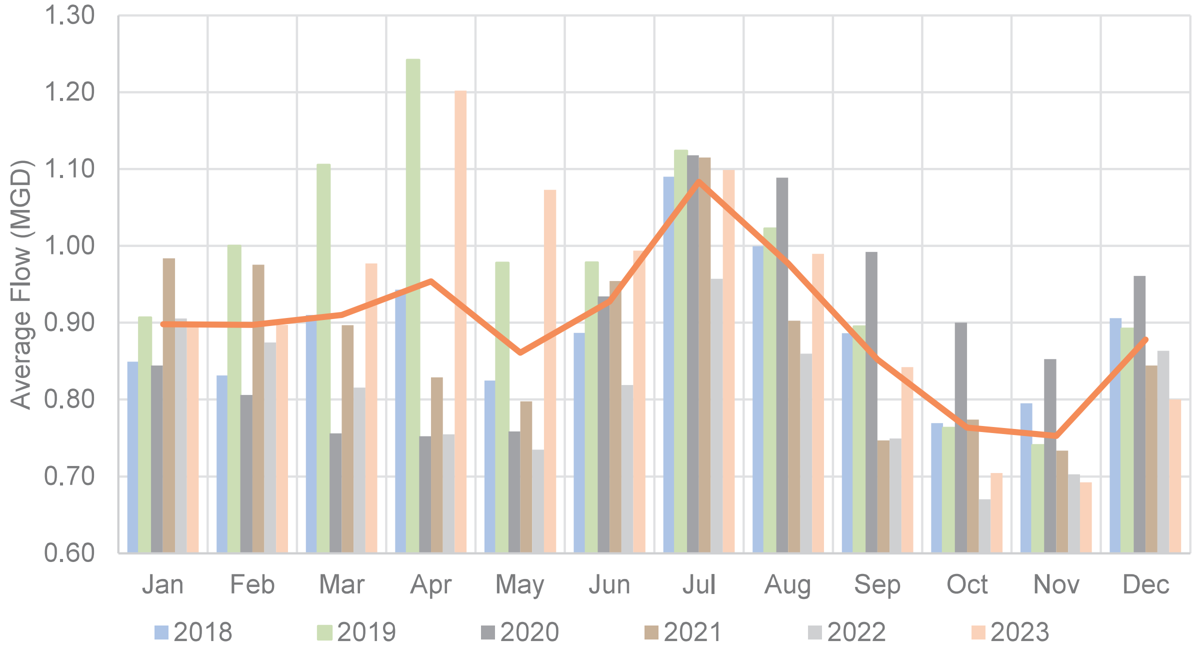

Figure 1: Average Daily Flow Summary, 2018 – 2023

There are three components wastewater flow within a sewer system that help define flows within a system: base sewer flows, inflow and infiltration, and sewer flow generated per connection. Base sewer flow, hereinafter referred to as average dry weather flow (ADWF), is the amount of sewer produced by the District and is not influenced by outside sources of water. Inflow and infiltration (I&I), as discussed earlier is the intrusion of outside water sources into the sanitary sewer system. Sewer generation per customer can be expressed in multiple ways, however the most common is through the development of an equivalent dwelling unit (EDU), that defines the average amount of sewer flow generated per day by a residential sewer connection.

To determine the ADWF of the system, the average metered water usage from October 2019 to November 2023 of all sewer customers was analyzed and compared to the average monthly sewer flows seen at the WRRF. This comparison was done to determine the months where water usage and sewer flows are most in line. The comparison is shown in Figure 2. The comparison showed that the winter months (November through March) had equal water usage and sewer flows. It was assumed that no outside sources, I&I, irrigation, etc., had an impact on sewer flows during these months.

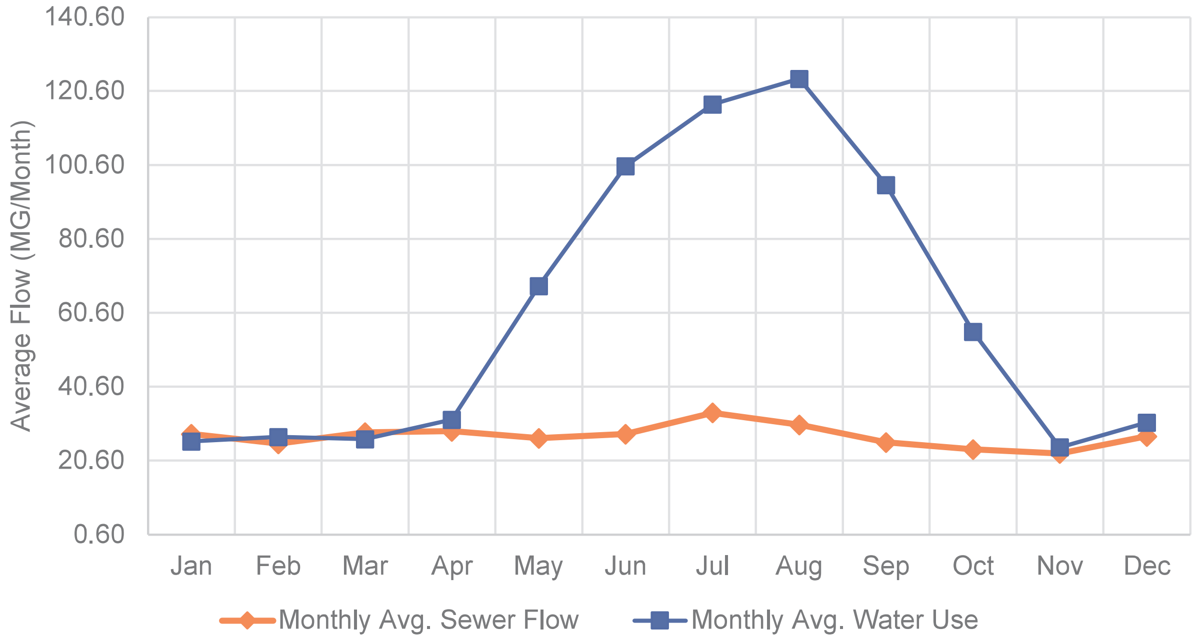

Figure 2: Monthly Average Sewer Flow and Water Usage, 2018 – 2023

Winter sewer flow data for five years (e.g., winter 2019 begins in November 2018 and ends March 2019) was used to calculate an ADWF, in MGD. Table 4 gives the calculated values. As seen, the ADWF has been fairly variable over the last five years. While there was an expected rise in 2021 due to the pandemic, it still did not exceed the ADWF in 2019.

Table 4: ADWF Summary, Winter 2019 – Winter 2023

Year	Total Flow (MG)	Average Daily Flow (MGD)
2019	142.32	0.94
2020	122.86	0.81
2021	140.99	0.93
2022	126.01	0.83
2023	131.14	0.87
Average	**132.66**	**0.88**

Inflow into a sewer system is typically surface stormwater due to precipitation or snow melt runoff entering into pipes and manholes through exposed openings (e.g., holes in manhole lids). Infiltration is through groundwater seeping through underground collection assets via cracks in pipes and manholes. Due to a lack of sewer flow data, outside of the WRRF influent data that was provided, I&I was not estimated as part of this Plan. However, the spike in average sewer flows in March and April, followed by the drop in flows in May is an indication that I&I may be an issue in the system. Additionally, as show in Figure 2, sewer flows have exceeded water usage in January and February on average the last five years. Water usage can only be lower than sewer flows if outside water is entering into the collection system. Future flow monitoring efforts are recommended to identify areas of the sewer system experiencing I&I and to determine the impact of the I&I on the system.

In order to accurately calculate the system EDU, residential and commercial sewer flows need to be separated. However, as sewer customers are not individually monitored, winter water usage for sewer customers was used based on the corresponding water usage and sewer flows shown in Figure 2. The total water usage of the District's sewer customers was summed and averaged out to a daily flow rate. This was then divided by the total number of units within each land use type. Multi-family and single-family residential customers were added together to calculate the EDU using only residential customers. The results of the analysis are shown in Table 5. The average residential generation was calculated to be 92.80 gpd/unit. This was then multiplied by a 10% safety factor for a final EDU value of 102.08 gpd/unit. This EDU value will be used to define system capacity in later sections of the Plan. Additionally, the commercial wastewater generation rate was multiplied by the same safety factor for a value of 775.80 gpd/unit.

Table 5: Wastewater Generation Rates by Customer Type, Winter 2020 – Winter 2023

Year	Commercial (gpd/unit)	Residential (gpd/unit)
2020	726.98	89.80
2021	652.97	98.26
2022	761.06	89.02
2023	680.08	94.10
Average	**705.27**	**92.80**

1.3 System Diurnal Curve

As the sewer system in the Plan area is not metered, IVGID provided hourly historical flow totals at the inlet to the WRRF from January 2022 through January 2023. A diurnal curve was computed for the system by averaging the hourly flowrate for each individual timestep (1 am, 2 am, etc.). Figure 3 presents the average day diurnal curve calculated for the system. During an average day, peak flows seen at the WRRF are approximately 1.5 times the average daily flow rate.

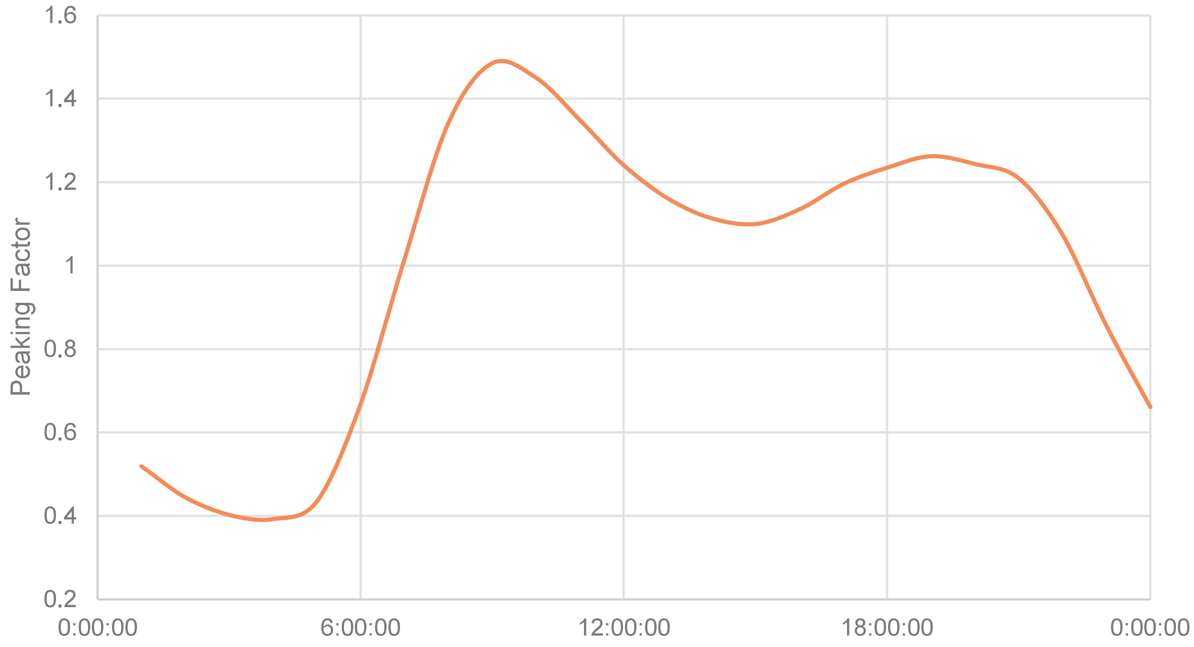

Figure 3: Average Day Diurnal Curve

To determine an overall peaking factor for the system, two different methodologies were used. The first compares the max daily flow from 2018 to 2023 to the average daily flow over the same period to obtain the daily flow peaking factor. This was then multiplied by the hourly peak of 1.5 in the average day diurnal curve. The second compares the absolute peak instantaneous hourly flow, in millions of gallons per hour (MGH), from January 2022 to January 2023, and compared to the average hourly flow rate over the same time period. Table 6 gives a summary of the peaking factors calculated for the system using the two different methods.

Table 6: Peaking Factor Summary

PF Description	Average Flow	Peak Flow	Peaking Factor
Daily Peaking Factor x Hourly Peaking Factor			
Daily Peaking Factor	0.90 MGD	1.62 MGD	1.8
Hourly Peaking Factor	0.034 MGH	0.05 MGH	1.5
System Peaking Factor	n/a	n/a	2.7
Peak Instantaneous Hourly Peaking Factor			
System Peaking Factor	0.034 MGH	0.10 MGH	3.1

Peaking factors from surrounding sewer districts were compared to the calculated peaking factors of 2.7 and 3.1. Table 7 contains the peaking factors referenced from sewer master plans for surrounding sewer districts.

Table 7: Peaking Factor Comparison

Utility Name	Date of Study	Peaking Factor
North Tahoe Public Utility District	2016	2.0
	1991	2.0-2.5
Olympic Valley Public Service District	2020	2.6
South Tahoe Public Utility District	2009	3.5
Truckee Sanitary District	2019	2.64-5.21

As peaking factors of 2.7 and 3.1 fall within the range of most of the surrounding districts, either one is assumed to be reasonable. However, of the utilities listed, two are actually within the Tahoe Basin, and the average of those two is 2.75. As this is in line with the first methodology results, an overall system peaking factor of 2.7 will be used for this Plan and to project future flows for the Plan area.

1.4 Future Flows

The buildout condition for IVGID was created by assuming that every privately owned vacant parcel would be developed at buildout. A future buildout flow was calculated by multiplying the vacant land use customer counts by either the system EDU of 102.08 gpd/unit or the commercial wastewater generation rate of 775.80 gpd/unit calculated in Section 1.2. Table 8 shows the expected average and peak flows to be added to the system assuming the District is fully built out.

Table 8: Average and Peak Buildout Sewer Flow Rates

Land Use Type	Unit Count	Average Buildout Flow Rate (gpd)	Peak Buildout Flow Rate (gpd)
Vacant, other, or unknown*	2	1,552	4,170
Vacant, under development*	3	306	823
Vacant, single family	185	18,884	50,753
Vacant, multi-residential	4	408	1,097
Vacant, commercial	36	27,929	75,061
Commercial Total:		29,480	79,232
Residential Total:		19,599	52,673
System Total:		**49,079**	**131,905**

1.5 Sewer Flow Summary

The buildout projections presented in Section 1.4 indicate that wastewater flows within the IVGID system may increase by approximately 6% based on the current average and peak daily flow rate. The Plan area is expected to see limited development of vacant parcels in the near to long-term future. It is expected that sewer flows will remain near their current values with small variations as IVGID grows. Table 9 summarizes the existing sewer flows and potential buildout demands for the Plan area.

Table 9: Sewer Flow Summary

Flow Scenario	Average Daily Flow (MGD)	Peak Daily Flow (MGD)
Existing System	0.90	2.41
Additional Flow during Buildout	0.05	0.13
Total Buildout System	**0.95**	**2.54**

2.0 RISK ANALYSIS

2.1 System Background

The Plan is a comprehensive evaluation of the District's entire sewer collection system with infrastructure ranging from 10 to 70 years old. The system serves residential and commercial customers located northeast of Lake Tahoe. The area primarily consists of residential customers ranging from large custom home sites to condominiums as well as commercial customers ranging from restaurants to hotels. The system consists of approximately 360,749 linear feet of 6-inch sewer main, 101,491 linear feet of 8-inch sewer main, 19,482 linear feet of 10-inch sewer main, 8,910 linear feet of 12-inch sewer main, 1,026 linear feet of 14-inch sewer main, 17,034 linear feet of 15-inch sewer main, 298 linear feet of 16-inch sewer main, 948 linear feet of 18-inch sewer main, 287 linear feet of 24-inch sewer main, and 1,840 manholes. A summary of pipe sizes is detailed in Figure 4.

Materials used to construct the system varies based on construction practice and material availability at the time of installation. Figure 5 shows the various pipe materials the existing system is comprised of. The older mains primarily consist of asbestos cement pipe (ACP), vitrified clay pipe (VCP). Construction within the last 40 years frequently used polyvinyl chloride (PVC). Ductile iron is used in several locations, commonly near waterways. Other materials with minimal use of C900 PVC, cast iron, cured-in-place pipe (CIPP), cement lined steel, and high-density polyethylene (HDPE).

Due to the regional significance of a utility located near Lake Tahoe tributaries, minimizing sewage collection infiltration and spill risk potential is a major consideration of system operation and potential system rehabilitation and replacement.

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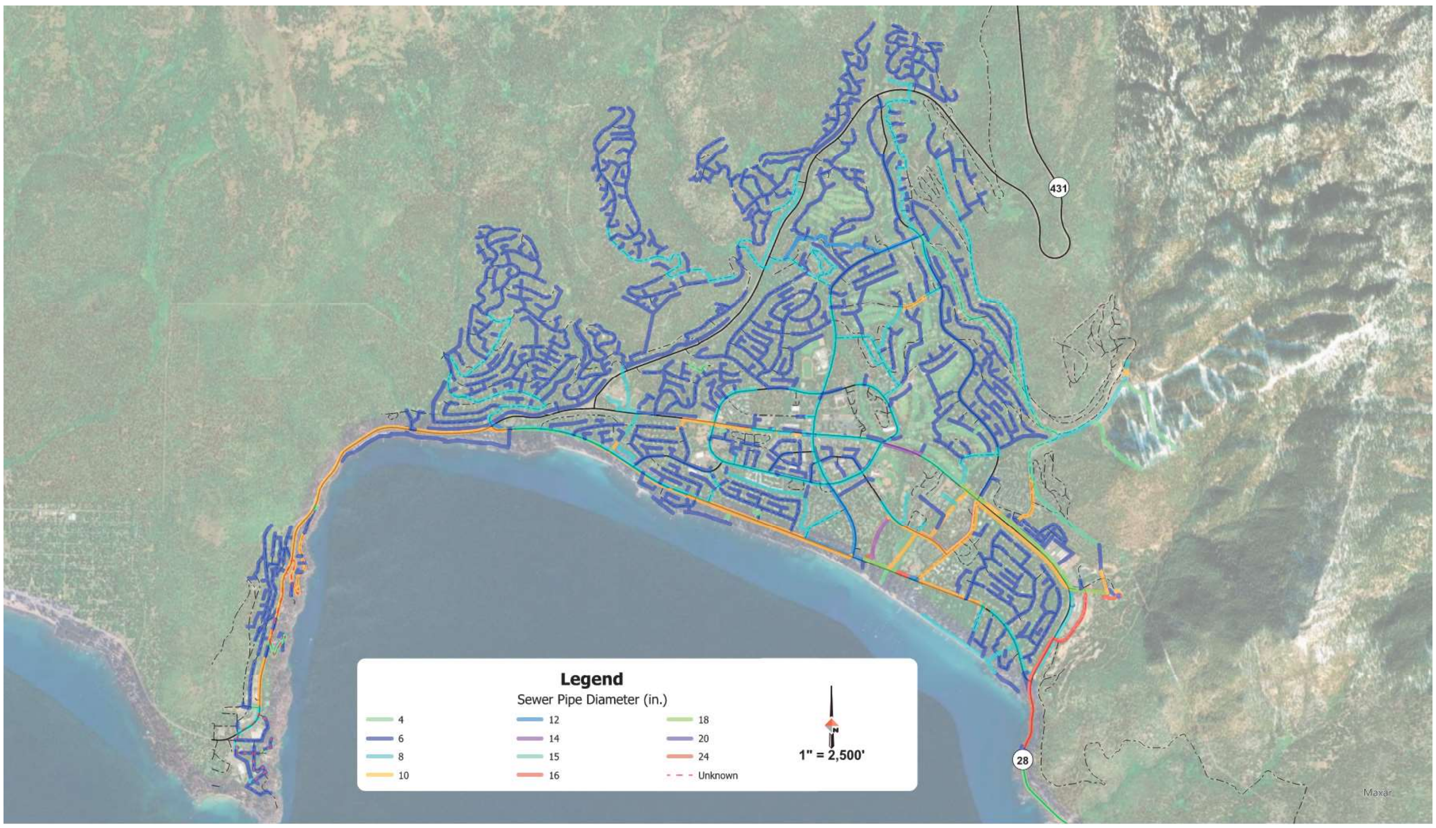

Figure 4: Sewer System Pipe Size Summary

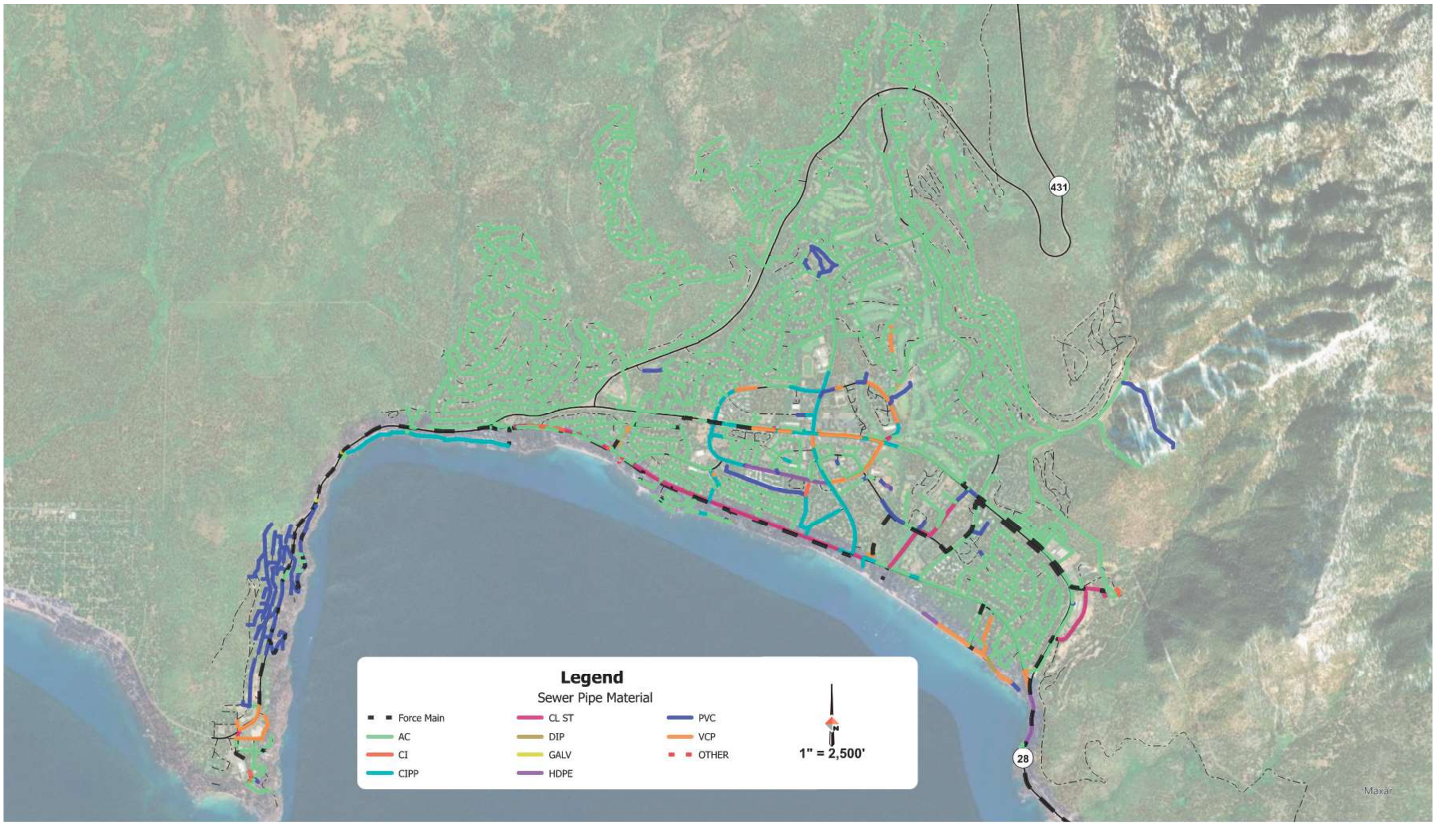

Figure 5: Sewer System Pipe Material Summary

2.2 Risk Assessment

The project team developed a risk evaluation matrix based on previous risk assessment studies. The developed matrix consists of several categories and weighting factors unique to the District's sewer collection system, land uses, infrastructure location, and operations. From these categories and weighting factors, a risk score for each pipe or manhole was determined. These scores are used to evaluate relative risk throughout the system and act as an instrument to develop a Capital Improvement Program (CIP).

2.2.1 Data Collection and Organization

The District provided the project team with GIS data to perform a desktop review of the sewer collection system. The GIS contains all of the District's pertinent sewer utility information that the project team used for the risk assessment.

2.2.2 Risk Categories

"Risk is a concept that relates to the expectation of a negative impact generated by some action or inaction. Commonly, risk is used synonymously with the likelihood (or probability) of a negative impact occurring. Sometimes risk is used to describe severity of the consequence of a potential failure. However, it is the combination of both of these factors, likelihood and consequence, that contributes to risk." (Implementing Asset Management: A Practical Guide, NACWA, AMWA, WEF, and CH2MHILL 2007).

Risk categories can be determined differently depending on what the District deems to be critical. The District's risk assessment uses the consequence and likelihood of failure components. Consequence relates to the resulting impact from failure, while likelihood relates to the potential for failure. Each of these categories includes parameters that contain a corresponding weighting factor. A separate matrix for pipes and manholes (MH) has been developed and provided below.

Table 10: Pipe Risk Matrix

Consequence			Likelihood			
Category*	Weight Factor*		Category*	Weight Factor*		
Waterway Proximity	0.50		Condition	0.40		
Pipe Size & Use	0.30	X	Operation & Maintenance	0.30	=	PIPE RISK
Land Use	0.20		Age	0.10		
			Slope	0.10		
			Material	0.10		
*Determined by DOWL						

Table 11: Manhole Risk Matrix

Consequence			Likelihood			
Category*	Weight Factor*	X	Category*	Weight Factor*	=	MH RISK
Waterway Proximity	0.70		Condition	0.50		
Land Use	0.30		Operation & Maintenance	0.30		
			Age	0.20		
*Determined by DOWL MH = manhole						

2.2.3 Consequence of Failure Categories

The categories of waterway proximity, pipe size and use, and land use have been established for the assessment and scoring of the risk of consequence for the District's sewer mains and manholes. Consequence risk refers to the potential adverse outcomes or impacts associated with the deterioration or failure of a pipe within a system. Each category is discussed in detail below.

2.2.3.1 Waterway Proximity

The waterway proximity category observed for consequence considers the impact on water quality resulting from potential failure. This evaluation considers the proximity of features to water bodies such as Lake Tahoe, Incline Creek, smaller tributaries leading to Third Creek, areas with high groundwater, and proximity to storm drain inlets. For the sewer system, a sewer pipe or manholes that are closer to waterways pose a greater risk to water quality than pipes located farther away. If effluent reaches a waterway, containment efforts and associated cleanup costs will be necessary.

The parameters, corresponding risk score, and weight factor for main pipes and manholes are detailed in Table 12.

Table 12: Waterway Proximity Consequence

Parameter	Risk Score	Weight Factor
100' or less from waterway	10	0.50 Pipe 0.70 MH
100' – 500' from waterway	7	
500' – 1000' from waterway	4	
1000' or more from waterway	1	

2.2.3.2 Pipe Size and Use

Pipe size and use are key components when analyzing risk as the size directly corresponds to multiple factors such as flow capacities, infrastructure cost, population served, etc. Larger pipes are a greater risk than smaller pipes because the larger pipes are responsible for carrying greater quantities of water or wastewater. For the sewer system, the larger pipes indicate that they are located downstream of the rest of the collection system. A blockage in the large pipe or manhole could result in problems upstream or a wall fracture could release a larger quantity of wastewater into the surrounding soils.

Use is also considered with the variables of gravity or pressure. Gravity sewers are more susceptible to leaks and root growth while pressurized mains are usually fused and lower chance of leakage into the system. For this reason, gravity mains are considered a higher consequence.

The parameters, corresponding risk score, and weight factor for sewer mains are detailed in Table 13.

Table 13: Sewer Pipe Size and Use Consequence

Parameter	Risk Score	Weight Factor
Collection≥10	10	0.30 Pipe
FM≥10	8	
Collection<10	6	
FM<10	4	
Collection<8	2	
FM<8	1	

2.2.3.3 Land Use

Land use identifies critical facility services and the consequence of service loss. Land use assumes that commercial users have considerably higher use than residential users; therefore, commercial land use has a greater risk score. Land use scoring is applied to the pipe that is upstream from the facility of interest and does not apply downstream of the facility. The viability of commercial businesses relies significantly on a robust sewer system, making it critical for them to have dependable infrastructure. A failure in the collection system would have a more severe impact on commercial customers compared to residential customers.

The parameters, corresponding risk score, and weight factor for main pipes and manholes are detailed in Table 14.

Table 14: Land Use Consequence

Parameter	Risk Score	Weight Factor
Commercial	10	0.20 Pipe 0.30 MH
Residential	4	
Other	6	

2.2.4 Likelihood of Failure Categories

The likelihood of failure categories of condition, operation and maintenance, pipe slope, age, and material have been established for the District’s sewer mains (S) and sewer manholes (SM). The likelihood of failure represents the probability of a failure occurring due to existing conditions. Each category is discussed in detail below.

2.2.4.1 Condition

The most important likelihood category is the condition of the asset. The condition of a pipe can be determined in two ways: visually when exposed, or by internal video inspection. Internal video inspection is the most common method for determining the condition of a pipe. The National Association of Sewer Service Companies (NAASCO) has established pipe scoring protocols that are becoming mandated by various agencies to obtain a consistent scoring method. The District has adopted the NAASCO PACP scoring of its sewer pipes and possesses PACP, MACP, and LACP scores for sewer pipes throughout the project area. This information is summarized in Section 2.1. The total PACP scores for sewer mains will be used to assign risk scores for this category. The risk scores and weight factors are detailed in Table 15. The total MACP scores for sewer manholes will be used to assign risk scores for this category. The risk scores and weight factors are detailed in Table 16.

Table 15: Main Condition Likelihood

Parameter	Risk Score	Weight Factor
>15	10	0.50 Pipe
11-15	8	
6-10	6	
1-5	4	
0	0	

Table 16: Manhole Condition Likelihood

Parameter	Risk Score	Weight Factor
≥8	10	0.50 MH
6-<8	8	
4-<6	6	
2-<4	4	
0-<2	0	

2.2.4.2 Operation and Maintenance

The level of operation and maintenance can serve as an indicator of other factors, including pipe condition and slope. However, these factors do not provide an estimate of maintenance frequency or the severity of operational consequences. The Operation and Maintenance category ensures that problematic features are considered in the risk assessment and given a high priority in the ranking. It is assumed that features requiring frequent maintenance due to structural or operational issues, such as frequent material deposition and clogging, pose a significant risk to the system and therefore merit a higher score. A Moderate O&M score indicates the need for annual maintenance, while a High O&M score suggests maintenance required multiple times a year.

The parameters, corresponding risk score, and weight factor for sewer mains, and sewer manholes are detailed in Table 17 and Table 18 respectively.

Table 17: Main Operation and Maintenance Likelihood

Parameter	Risk Score	Weight Factor
High O&M	10	0.30 Pipe
Moderate O&M	5	
Infrequent O&M	0	

Table 18: Manhole Operation and Maintenance Likelihood

Parameter	Risk Score	Weight Factor
High O&M	10	0.30 MH
Moderate O&M	5	
Infrequent O&M	0	

2.2.4.3 Slope

This category explores the slope of a sewer main and relates it to the likelihood of slope being a potential risk for backups in the sewer system.

The slope category will include the following parameters:

- Flat slopes or pipe segments with bellies: These pipe segments have the likelihood of solids deposition within the pipe. Pipes with bellies also have deposition in addition to stagnant areas.
- Mild slopes: Pipes with slopes of 1% to 2%, which are mild with respect to the system. Mains with mild slopes are typically free of severe bellies but may have mild deposition within the pipe.
- Typical to steep slopes: Pipes with slopes greater than 2% have typically had minor to no issues for the District and are considered low risk.

The parameters, corresponding risk score, and weight factor are detailed in Table 19.

Table 19: Sewer Main Slope Likelihood

Parameter	Risk Score	Weight Factor
0%-1%; Flat; Belly	10	0.1
1%-2%	7	
>2%	1	

2.2.4.4 Material

The District's sewer collection system consists of pipes made from different materials. Over the years, there have been variations in the preferred installation of standard pipe materials. These different materials have different lifespans, points of failure, and levels of reliability. Each pipe material is constructed with varying lengths, resulting in more or fewer joints. Joints are commonly identified as a point of failure in the sewer collection system. As a result, shorter pipes are considered to be at a higher risk because they require more joints, and pipes made of different materials are constructed with different lengths, resulting in additional joints. Lengths based on pipe material include the following:

- Clay – 4-foot lengths
- ACP/VCP – 6-foot lengths
- Ductile Iron (DI) – 18-foot lengths
- PVC/C900 – 20-foot lengths
- Steel – 21-foot lengths
- HDPE – Continuous
- Cured In Place Pipe (CIPP) - Varies

The parameters, corresponding risk score, and weight factors for pipes are detailed in Table 20

Table 20: Sewer Main Material Likelihood

Parameter	Risk Score	Weight Factor
Clay	10	0.10
ACP	8	
CIPP	6	
DI/Steel/CI	4	
PVC/C900/HDPE	2	

2.2.4.5 Age

The age of pipelines is an important factor when analyzing the risk of deterioration over time increases. As the age of a pipe increases, no matter the material, the risk of failure due to the composition or weakening of the pipe increases. The increased age can result in structural failures and O&M failures. The manufacturer's claimed life of features is displayed below.

- Clay – 75 years
- ACP – 75 years
- Ductile Iron – 80-100 years
- PVC – 80-100 years
- HDPE – 50-100 years

Older manholes have a typical life expectancy of 40-50 years and may last decades longer with proper maintenance and lining.

The parameters and corresponding risk score and weight factors for pipes are detailed in Table 21.

Table 21: Pipe Age Likelihood

Parameter	Risk Score	Weight Factor
0 < Age ≤ 20	1	0.10 Pipe 0.20 MH
21 < Age ≤ 40	4	
41 < Age ≤ 60	7	
61 < Age ≤ 80	10	
81 < Age ≤ 100	10	

2.2.5 Risk Scoring

DOWL assessed the data supplied by the District, assigning a risk score ranging from 1-10 (1=lowest and 10=highest), which was assigned to each sewer pipe and manhole within the study area. Each category also received a corresponding weighting factor to demonstrate their importance. At this time, the consequence categories are added together, and the likelihood categories are added together. Finally, the total consequence and likelihood scores are multiplied together to yield a cumulative risk score for a pipe or manhole. The risk score has a potential range from 0 – 100, with 0 being no risk and 100 being the highest risk.

Overall main pipe and manhole risk scoring results are presented in Figure 6 and Figure 7. Risk score tables for each feature are provided in Appendix A.

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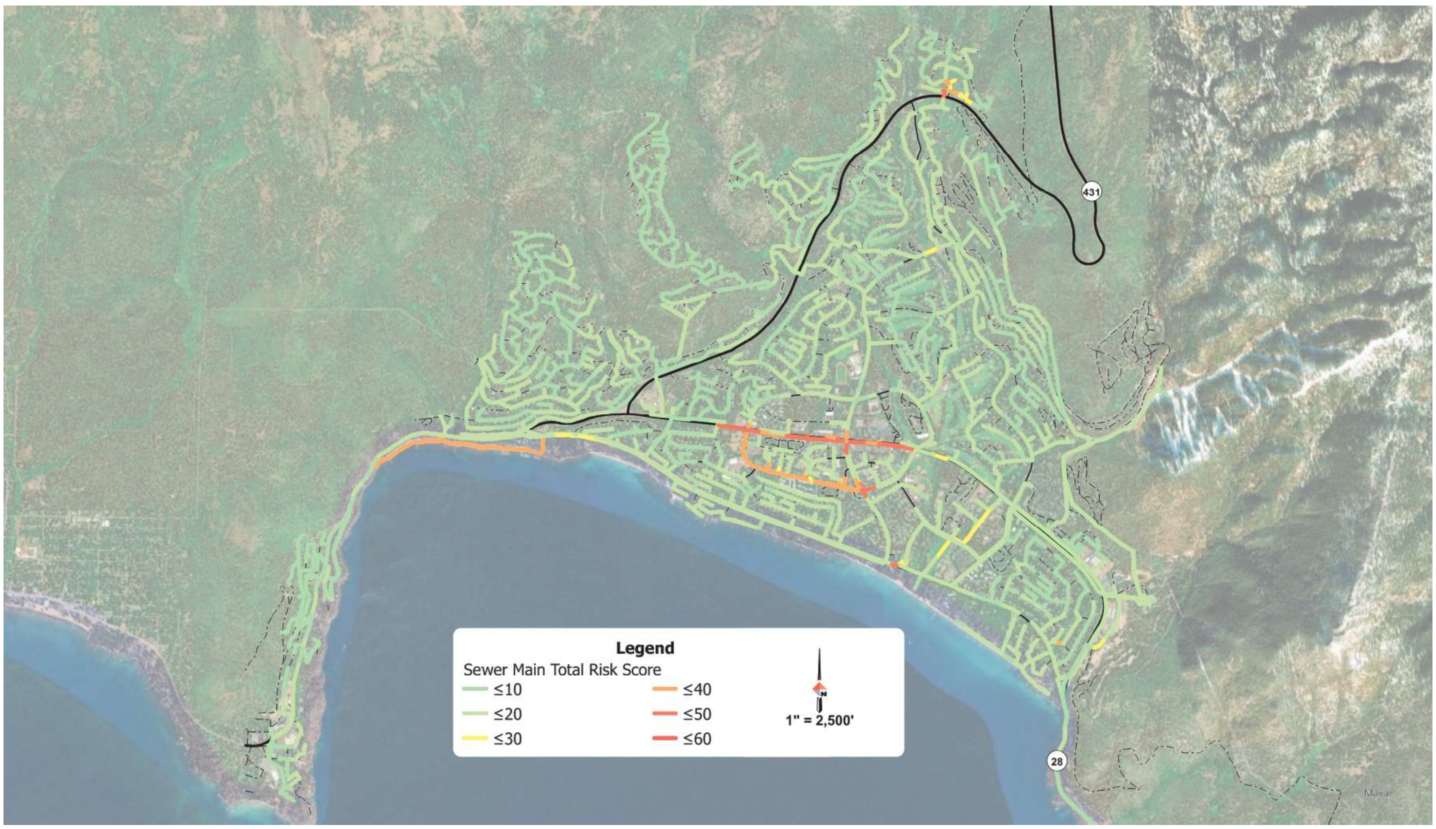

Figure 6: Pipe Rating Overall Map

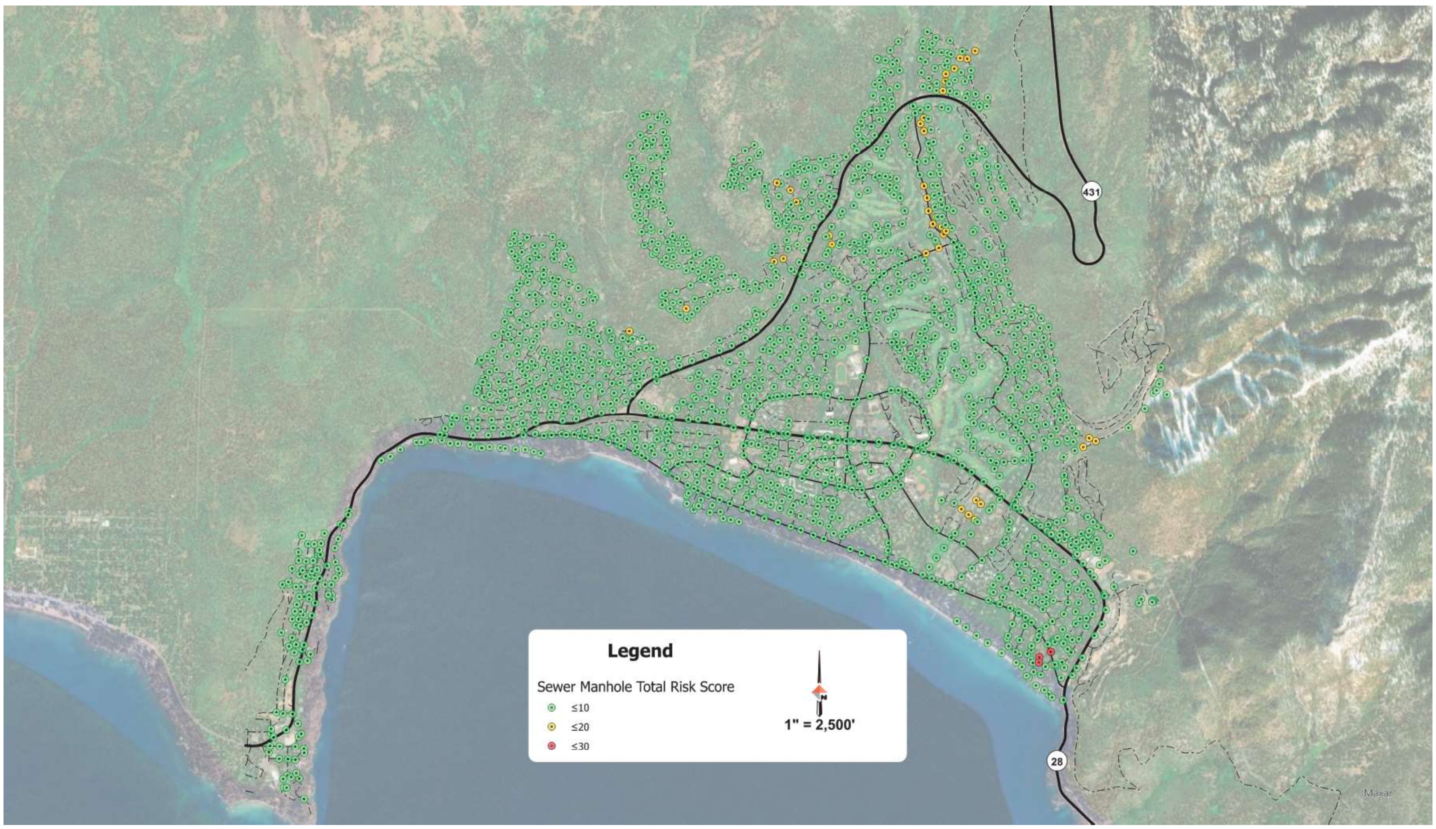

Figure 7: Manhole Rating Overall Map

2.2.6 Risk Results and Conclusions

An overall risk template approach for determining the consequence and likelihood risk scores for the District's sewer collection system has been developed with this document. As the District collects additional information, it will be able to incorporate and modify the parameter ranges of risk categories. As the data for the entire sewer collection system is incorporated into the risk assessment template, it will be possible for the District to score the consequences and likelihood of risks confidently and accurately.

This assessment provides insight into the high-risk areas within the sewer collection system, which will be further enhanced by NASSCO scoring information. This data, along with other tools, can serve as a parameter for prioritizing rehabilitation and/or replacement efforts in the sewer collection system. The District will use additional tools such as condition assessment, available rehabilitation technologies, cost analysis, and prioritized planning to support the capital planning process for the sewer system.

Sewer mains posing a higher relative risk are concentrated along the western segment of Northwood Boulevard and Southwood Boulevard. Additionally, sections along Crystal Rocks Drive that runs parallel to Lakeshore Boulevard and a section of Jupiter Drive fall within the medium to high-risk range. The District has specified that these lines undergo cleaning in both the spring and fall seasons every year. Many of these pipes serve critical facilities, are larger interceptor pipes, and are located in or near waterways posing a higher risk to the system. These categories are assigned higher weight factors, resulting in an increased risk rating. It is important to note that the presence of significant structural defects in these pipes is currently unknown.

Typically, only manholes that have received condition assessments are selected for assessment. Since the condition of the manholes is unknown at this time, all manholes were analyzed based on their age, proximity to waterways, land use, and operator input. The District has identified a cluster of manholes located along a private road south of Lakeshore Boulevard as having issues with inflow and infiltration, resulting in the highest risk rating. Manholes falling within the medium-high risk category are situated close to waterways or storm drains.

It is evident that the highest risk areas exhibit a combination of factors, including a history of frequent maintenance needs, proximity to waterways, and service to a commercial user. However, it is important to note that the identification of medium to higher risk sections does not indicate the need for immediate repairs. The next step involves conducting a structural analysis of the infrastructure, followed by developing a CIP which will be guided by the risk assessment. This analysis will help determine the priority for repair and replacement efforts.

2.3 Recommendations

Although the analysis has provided an overview of the potential risk of failure in the system, the absence of specific NASSCO scoring data presents a challenge when it comes to identifying the system's vulnerable areas. The ongoing condition assessment requires NASSCO scoring information to ensure its completion. By obtaining NASSCO's pipe information, the analysis can address critical factors such as corrosion, leakage, and structural stability. NASSCO has established a national standard to provide the sewer industry with the ability to accurately assess their infrastructure. It has three categories for infrastructure assessment and condition ranking:

- Pipeline Assessment and Certification Program (PACP).
- Lateral Assessment and Certification Program (LACP); and
- Manhole Assessment and Certification Program (MACP).

The first objective of the PACP scoring is to document structural deficiencies and construction features as they will have the highest potential of long-term influences on pipe integrity and pipe management. The structural family of events describes the various types of events where the pipe has been damaged or otherwise defective. Another objective of the PACP scoring is to identify operation and maintenance (O&M) deficiencies. The O&M family of events describes the various types of foreign objects that are found in sewers that may interfere with the operations of the system.

The LACP program is a continuation of the PACP program in that lateral pipes are no different than mainline pipes, except for size and configuration. The differences include access and fittings such as wyes, bends, and clean-outs.

MACP is based on standard PACP defect coding. Manhole component defects use PACP defect codes for the chimney, cone, wall, bench, and channel only. Additional MACP specific coding is used for manhole components such as cover/frame and adjustment rings, materials of construction, etc.

This data is critical to provide valuable insights into the current state of the District's pipes and structures, enabling a comprehensive evaluation of their performance and identification of specific projects to address any deficiencies. It is important to note that the risk assessment is just one part of the process for prioritizing reconstruction and rehabilitation efforts in the sewer utility. When the results of the condition assessment are obtained, it will be used in conjunction with the risk assessment performed in this analysis to further identify the area’s most vulnerable in the system.

3.0 SYSTEM OVERVIEW AND CAPACITY ANALYSIS

3.1 System Overview

3.1.1 Hydraulic Profile and Sewersheds

The IVGID sewer collection system is broken up into 19 sewersheds and comprised of 19 lift stations, 1,840 manholes, and approximately 97 miles of gravity main and 11 miles of force main, not including the effluent pipeline. Table 22 gives a summary of the number of manholes and the outfall for each sewershed.

Table 22: System Sewershed Summary

Sewershed	Manhole Count	Outfall
00	574	WRRF
01	763	SPS-1
02	34	SPS-2
04	3	SPS-4
05	16	SPS-5
06	8	SPS-6
07	322	SPS-7
08	268	SPS-8
09	3	SPS-9
10	151	SPS-10
11	3	SPS-11
12	20	SPS-12
13	22	SPS-13
14	7	SPS-14A
15	22	SPS-15
17	0	SPS-17
18	1	SPS-18
19	1	SPS-19
20	1	SPS-20

The collection system conveys sewer through a network of gravity sewer mains and pump stations to a terminal collection point at the WRRF. Multiple sewersheds pump into one another, creating a step ladder of pumping and gravity flow conveying sewerage to the WRRF. There are three influent sources into the WRRF. The first is a section of the system that is able to gravity flow into the facility, the second is SPS-1 (the largest lift station), and the third SPS-8. Of the 19 sewersheds, 17 ultimately flow into SPS-1. Of the remaining 2 sewersheds, one is gravity flows into the WRRF and the other flows into SPS-8. A map of the collection system and its sewersheds can be found in Figure 8. A hydraulic profile of the pressurized portion of the collection system can be found in Figure 9. Table 23 gives a summary of each sewershed flow path to the WRRF. The District also has the option to manually re-route flows throughout the system. Table 24 gives a summary of the manual operation of each lift station that has a path different than what is already shown in Table 23.

Table 23: Sewershed Flow – Normal Operation

Sewershed	Outfall	SPS Path to WRRF – Normal Operation
00	WRRF	Gravity Flow
01	SPS-1	1
02	SPS-2	2-7- 8 (can overflow to 1)
04	SPS-4	4-1
05	SPS-5	5-1
06	SPS-6	6-10-1
07	SPS-7	7- 8 (can overflow to 1)
08	SPS-8	8 (can overflow to 1)
09	SPS-9	9-5-1
10	SPS-10	10-1
11	SPS-11	11-13-12-1
12	SPS-12	12-1
13	SPS-13	13-12-1
14	SPS-14A	14A-14B-1
15	SPS-15	15-1
17	SPS-17	17-1
18	SPS-18	18-1
19	SPS-19	19-1
20	SPS-20	20-17-1

Table 24: Sewershed Flow – Manual Operation

Sewershed	Outfall	SPS Path to WRRF – Manual Operation
00	WRRF	Gravity Flow
01	SPS-1	1
02	SPS-2	2-7- 8-can overflow to 1
04	SPS-4	4-1
05	SPS-5	5-1
06	SPS-6	6-10-1
07	SPS-7	7- 8- can overflow to 1
08	SPS-8	8
09	SPS-9	9-5-1
10	SPS-10	10-1
11	SPS-11	11-13-12-7-8-WWRF or overflow to 1
12	SPS-12	12-7-8-WWRF or overflow to 1
13	SPS-13	13-12-7-8-WWRF or overflow to 1
14	SPS-14A	14A-14B-7-8-WWRF or overflow to 1
15	SPS-15	15-7-8-WWRF or overflow to 1
17	SPS-17	17-1
18	SPS-18	18-1
19	SPS-19	19-1
20	SPS-20	20-17-1

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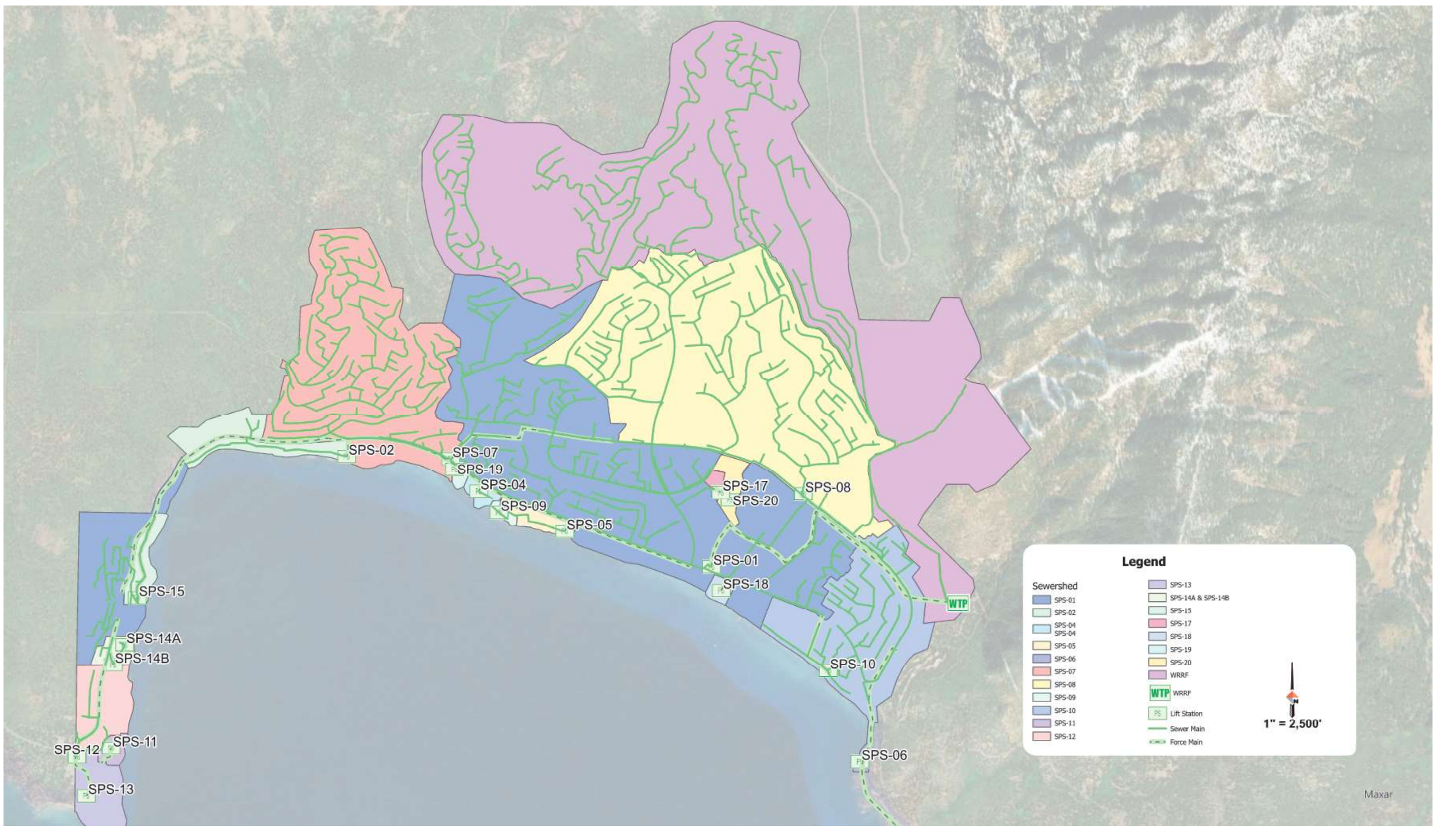

Figure 8: Sewer System Map

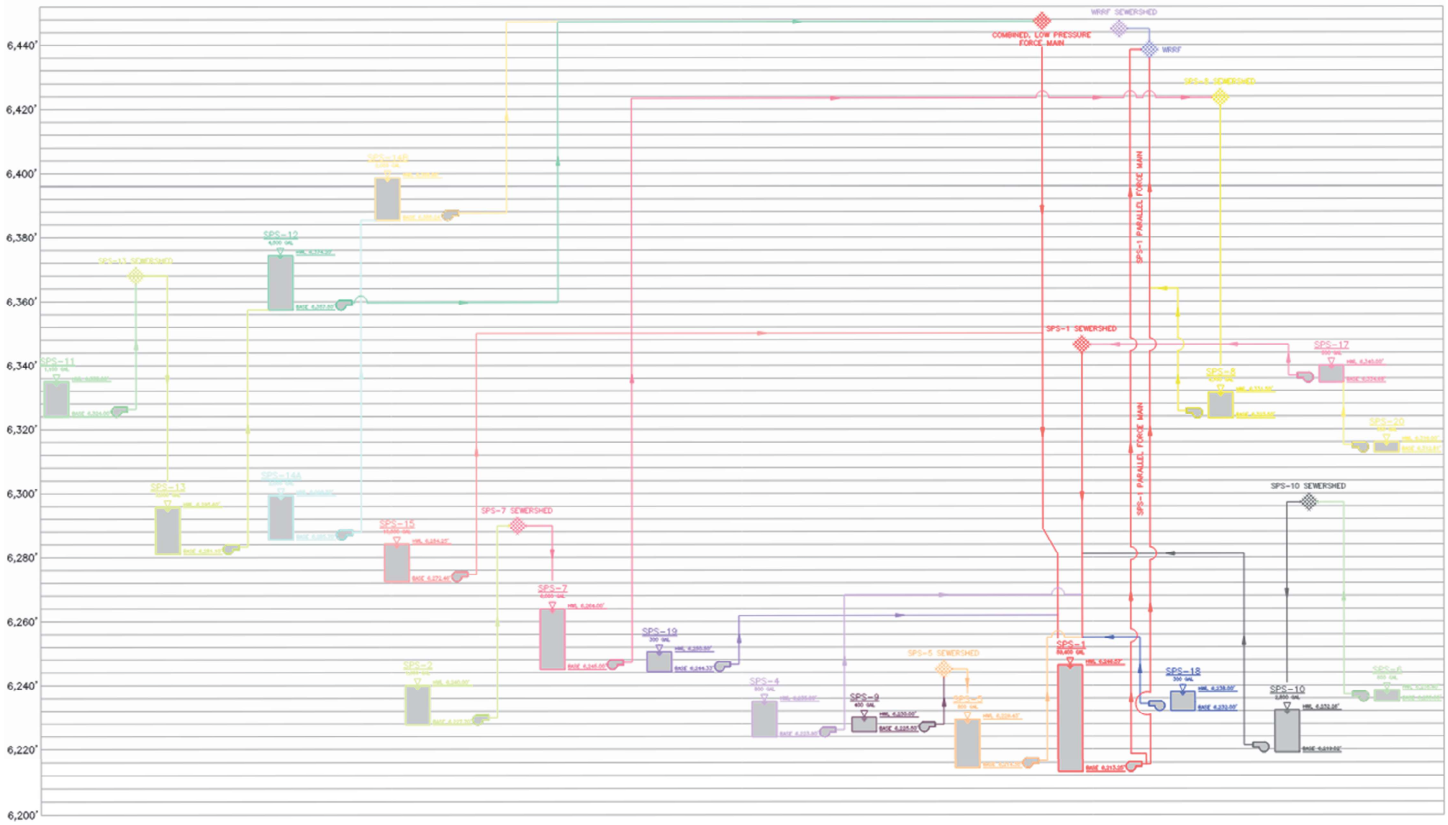

Figure 9: System Hydraulic Grade Line Profile

3.1.2 Lift Stations

Due to the mountainous terrain in the IVGID service area, the collection system utilizes a number of lift stations in order to convey sewer safely to the WRRF. Of the 19 lift stations in the IVGID system, SPS-1 is the largest and it collects sewer from 17 of the system sewersheds as shown in Figure 9 and Table 23.

The lift stations are split between underground vaults and stand-alone structures. Six of the lift stations have standalone structures. Two lift stations have automatic bypasses constructed into the wet wells, SPS-8, and SPS-12. The SPS-8 overflow moves excess sewer via gravity to SPS-1 where it will then be pumped to the WRRF. The SPS-12 overflow is a key overflow in the system and is consistently monitored. Located in Crystal Bay near the state boundary with California, the SPS-12 overflow will convey flow to the North Tahoe Public Utilities District (NTPUD) in California. Due to the regulatory concerns of moving sewer to another state, this lift station is consistently monitored. A summary of each lift stations wet well capacities and pump information can be found in Table 25 and Table 26 respectively.

Table 25: Lift Station Wet Well Summary

SPS ID	Wet Well Diameter (ft)	Wet Well Capacity (gal)
SPS-1	Two Rectangular Cells (13.75 ft x 12.5 ft)	59,400
SPS-2	5	1,600
SPS-4	3	800
SPS-5	3	800
SPS-6	5	500
SPS-7	8	6,000
SPS-8	Rectangular (12 ft x 6 ft)	4,700
SPS-9	4	400
SPS-10	6	2,800
SPS-11	4	1,100
SPS-12	8	4,500
SPS-13	6.08	3,000
SPS-14A	5	2,000
SPS-14B	5	2,000
SPS-15	Rectangular (7 ft x 17 ft)	10,500
SPS-17	Inaccessible	500
SPS-18	3	300
SPS-19	2.83	300
SPS-20	Inaccessible	Inaccessible

Table 26: Lift Station Pump Summary

SPS ID	# Of Pumps	Pump Flow (gpm)	Pump TDH (ft)	Pump Type
SPS-1	3	1,000	400	Wet pit/dry pit two centrifugal in series
SPS-2	2	300	72	Wet pit/dry pit centrifugal
SPS-4	2	280	41	Wet pit/dry pit centrifugal
SPS-5	2	78	42	Wet pit/dry pit air-eject
SPS-6	2	80	74	Wet pit/dry pit vacuum-lift centrifugal
SPS-7	2	700	205	Wet pit/dry pit centrifugal
SPS-8	3	1,000	178	Wet pit/dry pit centrifugal
SPS-9	2	50	20	Wet pit/dry pit air-eject
SPS-10	2	460	90	Wet pit/dry pit vacuum-lift centrifugal
SPS-11	2	80	44	Wet pit/dry pit vacuum-lift centrifugal
SPS-12	2	900	144	Wet pit/dry pit centrifugal
SPS-13	2	200	128	Wet pit/dry pit centrifugal
SPS-14A	2	55	105	Wet pit submersible grinder
SPS-14B	2	55	105	Wet pit submersible grinder
SPS-15	2	150	104	Wet pit/dry pit centrifugal
SPS-17	2	40	30	Wet pit submersible grinder
SPS-18	2	25	30	Wet pit submersible grinder
SPS-19	2	30	15	Wet pit submersible grinder
SPS-20	2	Unknown	Unknown	Unknown

Of the 19 lift stations in the system, 13 have access to an emergency power source. Of the 6 lift stations that do not have access to emergency power, only SPS-8 has an overflow. 5 lift stations have direct access to emergency power via onsite generators. 5 have emergency power access via connections to offsite generators. 3 lift stations are able to use portable generators owned by IVGID and able to be transported and connected by staff. A summary of the lift station emergency power sources is found in Table 27.

Table 27: Lift Station Emergency Power Summary

SPS ID	Emergency Power Access	Emergency Power Type
SPS-1	Yes	Onsite Generator
SPS-2	Yes	Onsite Generator
SPS-4	Yes	Offsite Generator (Burnt Cedar WDP)
SPS-5	Yes	Portable Generator
SPS-6	Yes	Portable Generator w/ Cannon Plug
SPS-7	Yes	Onsite Generators
SPS-8	No	n/a
SPS-9	Yes	Portable Generator
SPS-10	Yes	Onsite Generators
SPS-11	No	n/a
SPS-12	Yes	Offsite Generator (WPS-2)
SPS-13	Yes	Offsite Generator (WPS-2)
SPS-14A	Yes	Offsite Generator (SPS-15)
SPS-14B	Yes	Offsite Generator (SPS-15)
SPS-15	Yes	Onsite Generators
SPS-17	No	n/a
SPS-18	No	n/a
SPS-19	No	n/a
SPS-20	No	n/a

Two lift stations, SPS-17 and SPS-20 were removed from further capacity calculations and analysis. Elevation and capacity information at these lift stations did not exist and it was noted that the wet wells were locked shut and inaccessible. SPS-17 and SPS-20 both serve a few bathrooms and surrounding facilities near Incline Park and were identified as non-crucial pieces within the Plan nor the collection system. After discussions with IVGID staff, it was agreed that removing these lift stations from the model and not including them in capacity assessments was the best path forward.

3.1.3 Collection Mains and Manholes

The IVGID sewer collection system is made up of over 97 miles of gravity main and 11 miles of force main, excluding the export pipeline, with a large range of pipe diameters, materials, and ages. The collection system also includes 1,840 manholes. Table 28, Table 29, Table 30, Table 31, and Table 32 give summaries of the distribution main diameters, materials, and age respectively. Table 33 summarizes the depth profiles of all manholes in the system.

Table 28: Gravity Main Diameter Summary

Pipe Diameter (in)	Length (ft)
6	360,749
8	101,491
10	19,482
12	8,910
14	1,026
15	17,034
16	298
18	948
24	287

Table 29: Force Main Diameter Summary

Pipe Diameter (in)	Length (ft)
2	160
4	4,155
6	2,227
8	1,837
10	38,269
14	4,768
18	4,989

Table 30: Gravity Main Material Summary

Pipe Material	Length (ft)
Asbestos Cement, AC	423,541
Cast Iron, CI	1,685
Cured in Place Pipe, CIPP	23,390
Cement Lined Steel, CL ST	11,128
Ductile Iron Pipe, DIP	348
High Density Polyethylene, HDPE	2,605
Polyvinyl Chloride, PVC	31,598
Vitrified Clay Pipe, VCP	15,930

Table 31: Force Main Material Summary

Pipe Material	Length (ft)
Asbestos Cement, AC	34,020
Cast Iron, CI	15
High Density Polyethylene, HDPE	1,666
Polyvinyl Chloride, PVC	9,780
Steel	10,764

Table 32: Pipe Age Summary

Pipe Age	Length (ft)
0-10 years	10,556
11-20 years	9,432
21-30 years	0
31-40 years	6
41-50 years	7,454
51-60 years	19,158
61-70 years	2,440
Unknown	517,424

Table 33: Manhole Depth Summary

Depth (ft.)	# Of Manholes
0-5	219
5-10	1,242
10-15	130
15-20	14
20-30	1
>30	1
Unknown	233
Total	**1,840**

Throughout the IVGID collection system, there are believed to be seven different sewer overflow locations. These overflows are utilized during periods of high flow and allow sewer to overflow into a different part of the collection system. Using GIS data coupled with a manhole measure down survey, performed by DOWL, DOWL was able to confirm that five of these locations are sewer overflows and two are not. A brief description of these locations is included below.

Near the westernmost intersection of Lakeshore Boulevard and Highway 28, an overflow pipe branches off of the low pressurized force main. The low pressurized force main normally conveys sewer flow from Crystal Bay to SPS-1 but a valve can be opened that allows the flow to discharge north to the SPS-7 sewershed. Figure 10 is an overview of the overflow location.

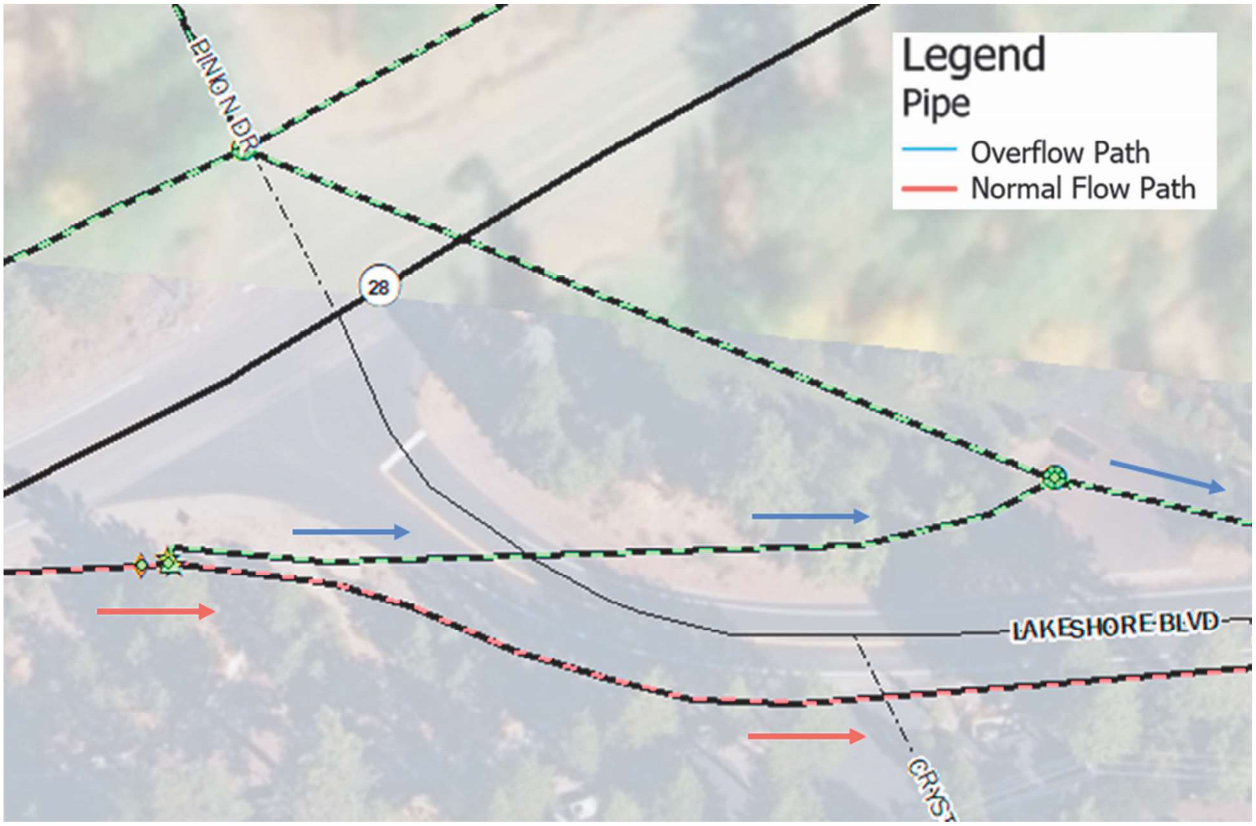

Figure 10: Lakeshore Boulevard & Highway 28 Sewer Overflow

Near the intersection of College Drive and Mount Rose Highway, sewer flow is normally conveyed to the northeast towards Titlist Drive, through the WRRF sewershed. An overflow pipe located approximately 0.14' above the normal flow path, continues down College Drive and can convey the flow to the SPS-8 sewershed. Figure 11 is an overview of the overflow location.

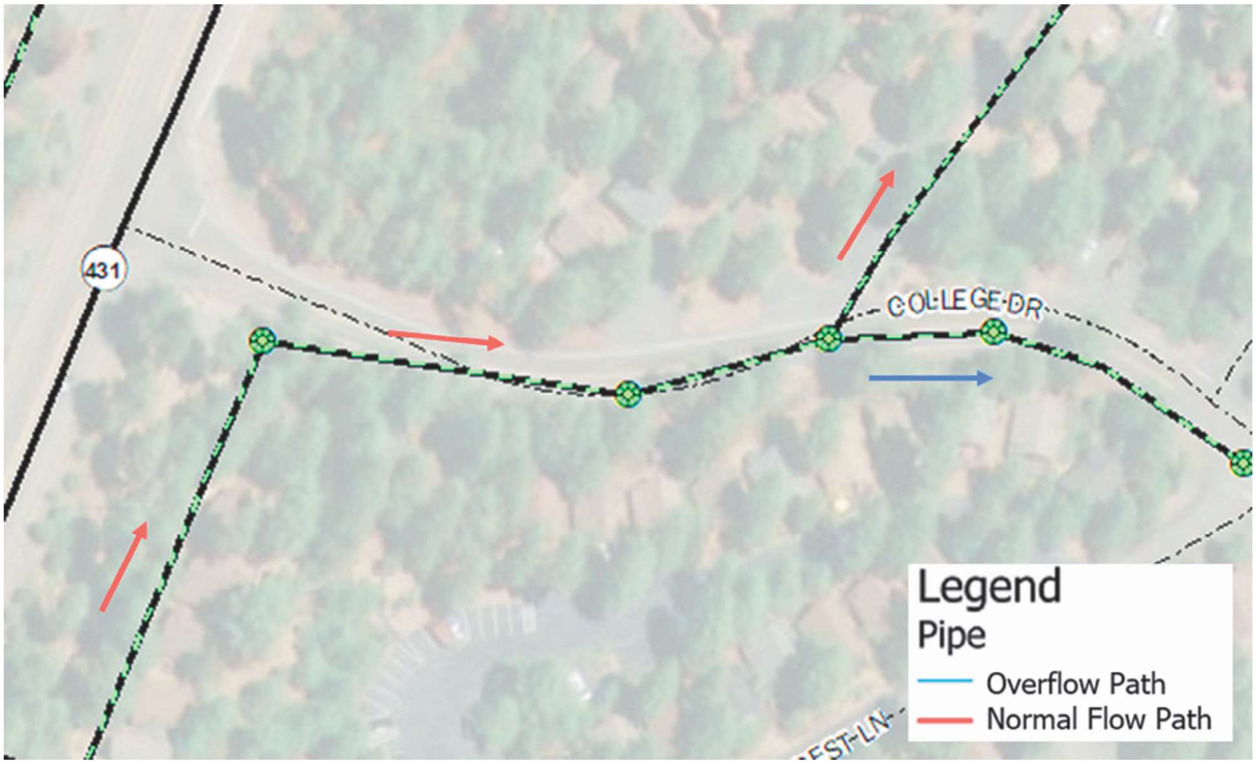

Figure 11: College Drive & Mount Rose Highway Sewer Overflow

At the intersection of Village Boulevard and Harold Drive, sewer flow is normally conveyed to the south along Village Boulevard, through the SPS-8 sewershed. An overflow pipe continues down Harold Drive and can convey the flow to a separate portion of the SPS-8 sewershed. Currently, IVGID staff have a manual plug installed within the pipe along Harold Drive and the overflow line is not utilized. Figure 12 is an overview of the overflow location.

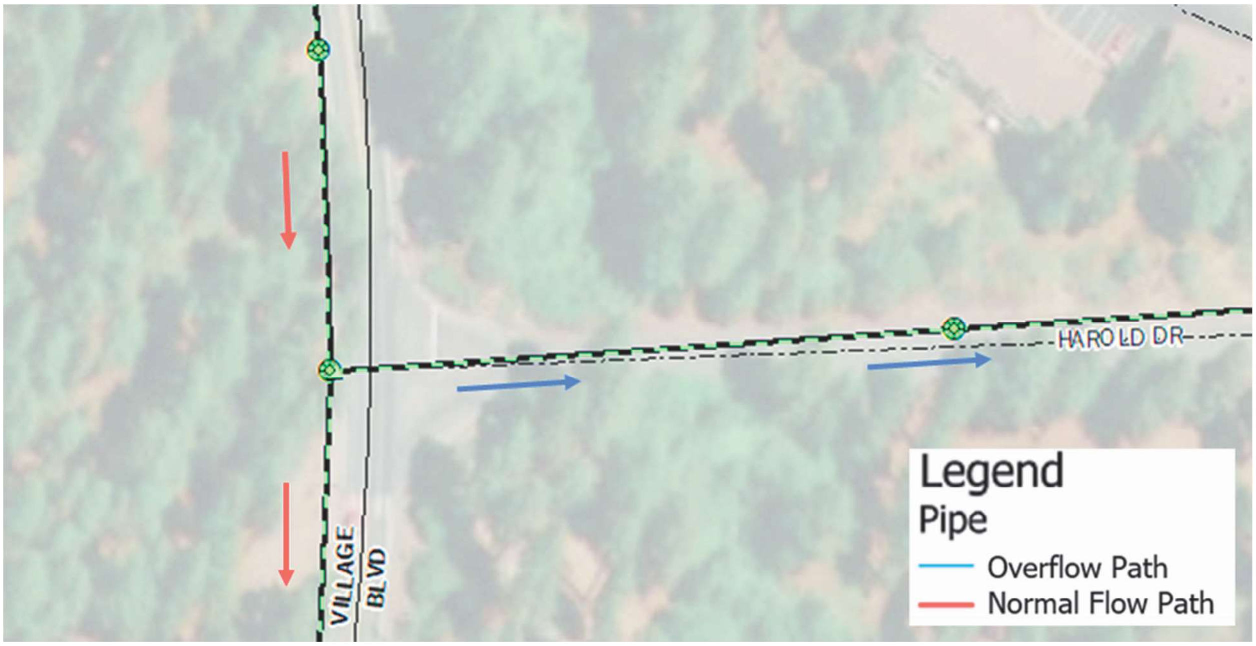

Figure 12: Village Boulevard & Harold Drive Sewer Overflow

At the intersection of Village Boulevard and Highway 28, sewer flow is normally conveyed to the east along Highway 28, through the SPS-8 sewershed. An overflow pipe located approximately 0.16’ above the normal flow path, continues down Village Boulevard and can convey the flow to the SPS-1 sewershed. Figure 13 is an overview of the overflow location.

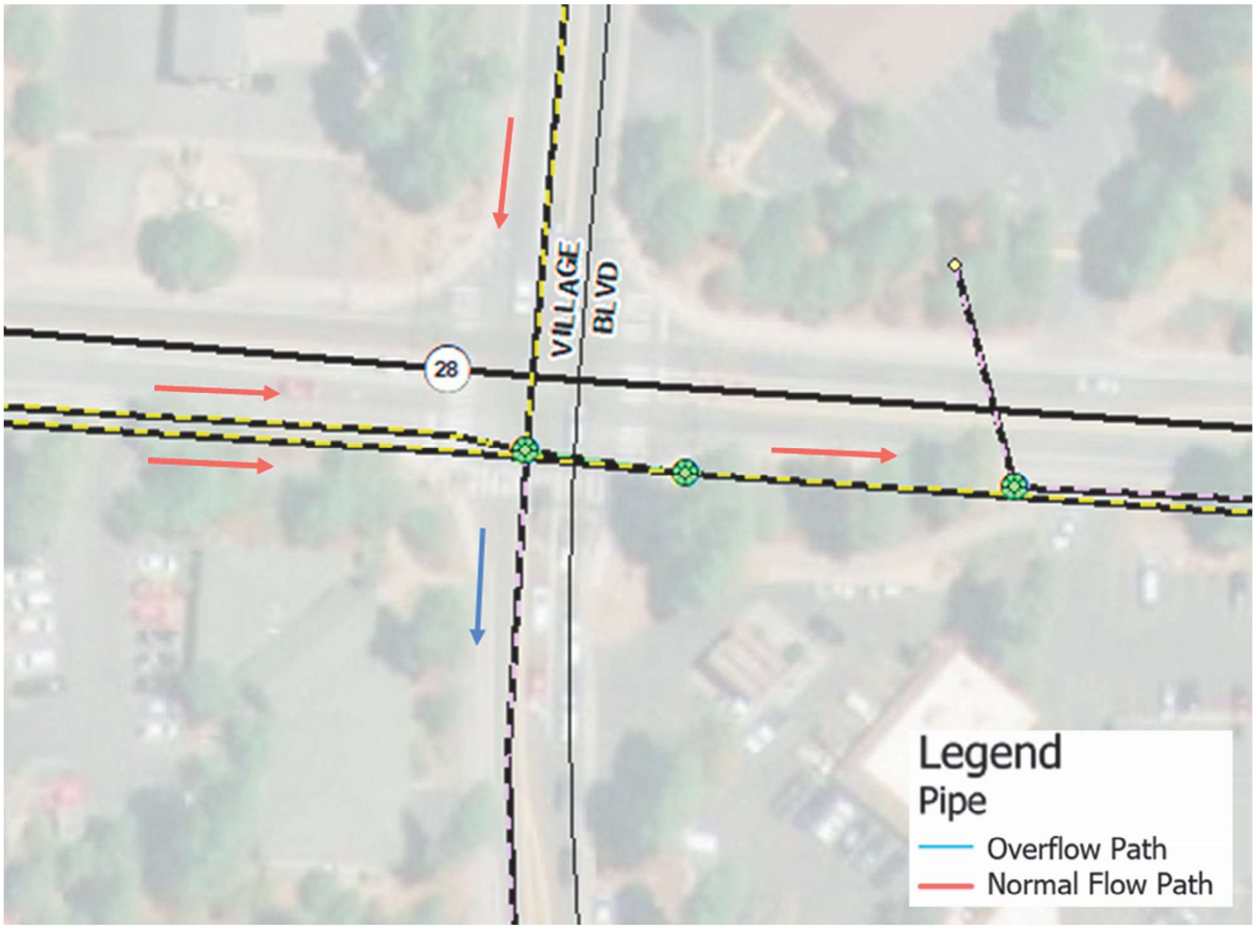

Figure 13: Village Boulevard & Highway 28 Sewer Overflow

At the intersection of Northwood Boulevard and Highway 28, it was determined that the sewer overflow pipe does not exist. It was originally assumed that an overflow pipe continued down Southwood Boulevard and could convey the flow to the SPS-1 sewershed. However, as observed during the manhole measure down survey, there was no pipe that continued to the southwest. Figure 14 is an overview of this location.

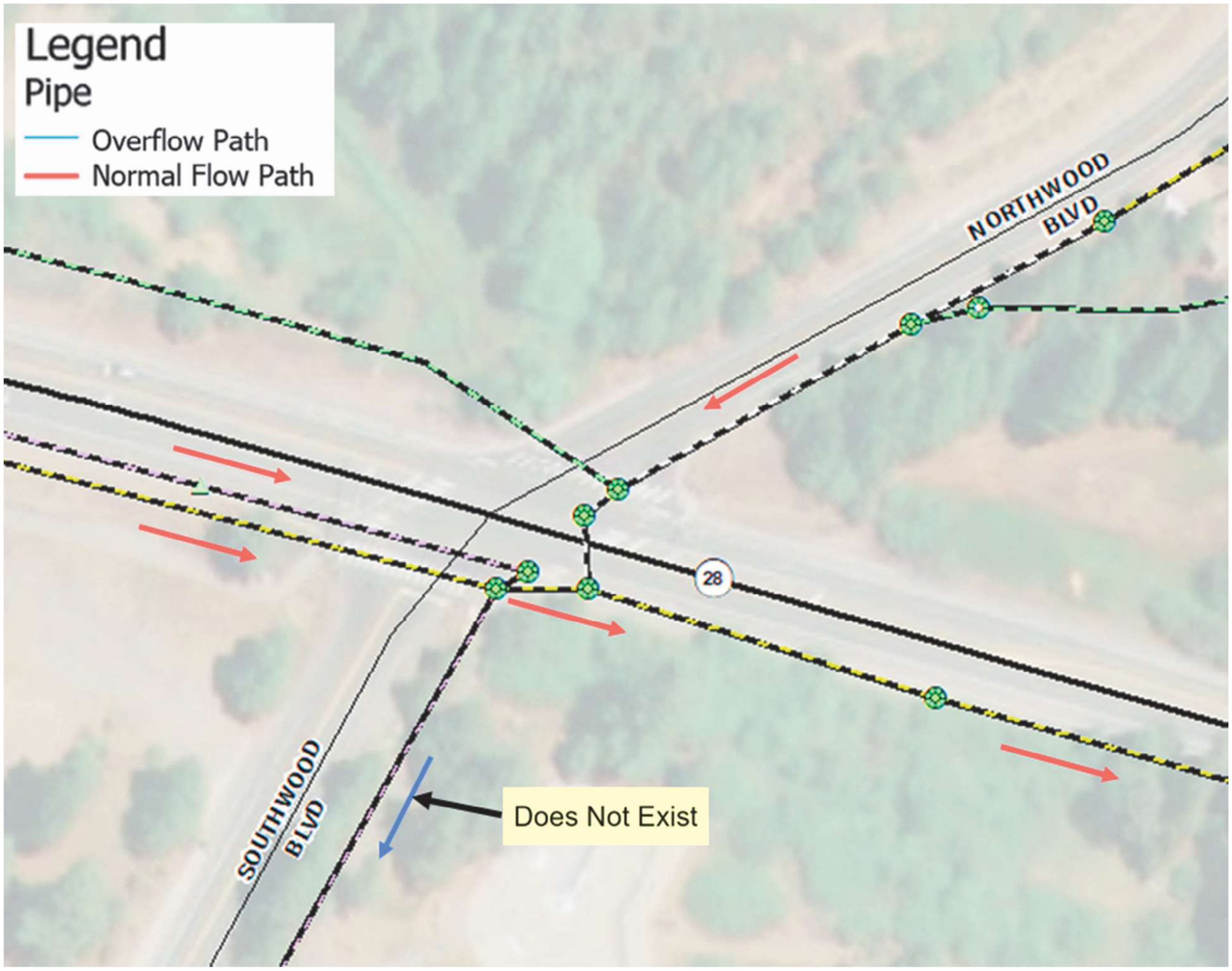

Figure 14: Northwood Boulevard & Highway 28 Sewer Overflow

At the intersection of Country Club Drive and Highway 28, sewer flow is normally conveyed to the northwest along Highway 28, through the SPS-8 sewershed. Two overflow pipes were assumed to exist in this area, one that follows Country Club Drive and one that cuts through private property, to the west of Country Club Drive. During the manhole measure down survey, it was determined that the pipe that continues along Country Club Drive does not exist but the pipe to the west does. The western overflow pipe is located approximately 0.23' above the normal flow path and can convey flow to the SPS-1 sewershed. Figure 15 is an overview of the overflow location.

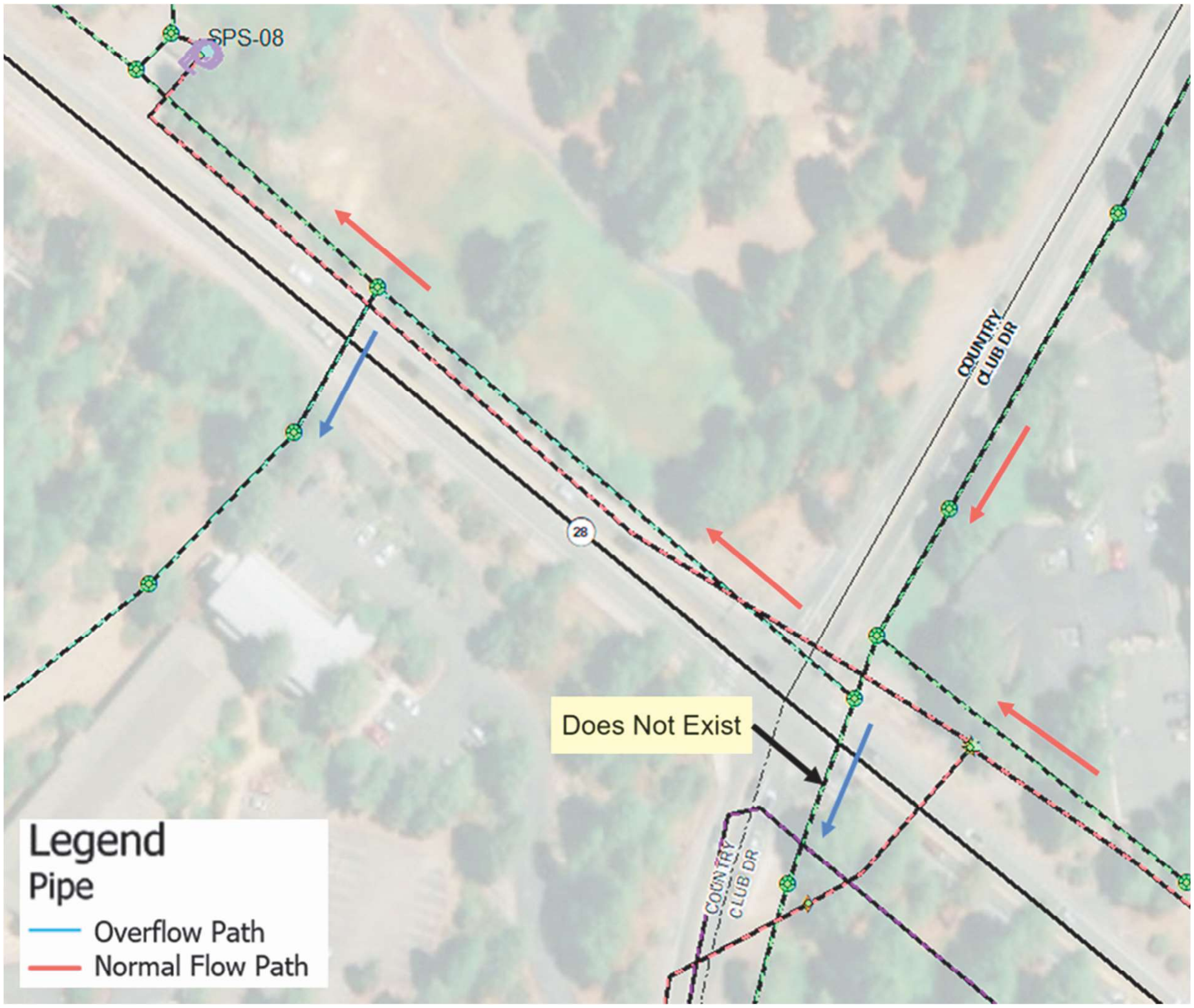

Figure 15: Country Club Drive & Highway 28 Sewer Overflow

3.1.4 Water Resource Recovery Facility (WRRF)

The terminal point for all sewer in the system is the WRRF. The facility treatment process train currently includes raw influent flow metering, fine screening, grit removal, secondary biological treatment, clarification, and disinfection along with waste sludge digestion, and mechanical sludge dewatering. A more comprehensive description of the WRRF history, treatment process, and condition assessment of the plant can be found in Section 4.0.

3.1.5 Effluent Export System

Effluent from the WRRF is stored in the 500,000-gallon effluent storage tank before it is exported out of the Lake Tahoe Basin to the Wetlands Enhancement Facility south of Carson City and other end users. The effluent storage tank provides operational storage and suction head for the Spooner Effluent Pump Station (SPS-16). The tank provides 8 to 12 hours of effluent storage depending on the prevailing influent flow rate. The Spooner Pump Station moves the effluent to the wetlands facility, as well as two different irrigation sites, Clear Creek at Tahoe, and Schneider Ranch. Further description of the effluent export system, its operations and an assessment of its condition can be found in Section 4.6.

3.2 System Flows

Existing and future system sewer flows, peaking factors, and system diurnal were developed using WRRF influent flow data provided by IVGID. A description of this analysis ca be found in Section 1.0. For the purposes of this Plan and the following capacity assessment, the system is assumed to have the existing and peak flows summarized in Table 34.

Table 34: Sewer Flow Summary

Flow Scenario	Average Flow (MGD)	Peak Flow (MGD)
Existing	0.86	1.29
Buildout	0.90	1.41

3.3 System Capacity

Capacity of the gravity collection system was assessed against the following criteria as identified within the Washoe County Community Services Department (CSD) – Gravity Sewer Collection Design Standards:

- The ratio of maximum sewer flow in an individual pipe to the pipe diameter (d/D) cannot exceed 0.80 per criteria 2.1.02.4.
- Pipe velocities must be a minimum of 2.5 feet per second (ft/s) when flowing half full per criteria 2.1.02.3.
- Minimum allowable pipe slope is identified as the slope at which the flow velocity is at least 2.5 ft/s when flowing half full per criteria 2.1.02.3.
- No manholes in the system may surcharge
- A manhole is considered surcharged if at any point the sewer flow line overtops the contributing pipes

It should be noted that capacity for the gravity sewer system in several sewersheds was not a part of the capacity analysis. These areas were either missing elevation data and not included in the hydraulic model of the system after consultation with District staff or were small lift stations that had no known upstream gravity mains. A summary of these sewersheds and the purpose of their exclusion are presented in Table 35.

Table 35: Sewersheds Excluded from Gravity Main Capacity Analysis

Sewershed ID	Reason for Exclusion
SPS-6	Insufficient elevation data
SPS-11	Insufficient elevation data
SPS-13	Insufficient elevation data
SPS-14B	No upstream sewer mains
SPS-18	No upstream sewer mains
SPS-19	No upstream sewer mains

Capacity for the lift stations and pressurized was analyzed for the pumps, wet wells, and force mains at each lift station. The capacity for each component will be expressed in EDUs for an even comparison.

The hydraulic capacity of the lift stations was determined using the following criteria from the Nevada Division of Environmental Protection (NDEP) Technical Sheet WTS-14:

- Minimum of 10 minutes between successive starts per hour
- Minimum of two independent pumps able to convey peak flows independently
- Emergency storage adequate to meet 3.5 times the average hourly flow for 2 hours
- Identified through industry standards, force main velocities between 3 and 8 ft/s should be maintained.

As shown in this report, the District is not expected to have substantial growth in a buildout condition. As such, many of the system capacity issues identified are not exacerbated by the buildout condition. Sections 3.3.1 and 3.3.2 below present the system capacity findings for the existing and buildout conditions respectively. Any deficiencies identified in these sections are discussed more thoroughly, along with proposed solutions, in Section 3.4 to avoid repetition.

3.3.1 Existing System Capacity

3.3.1.1 Collection System

Table 36 summarizes the maximum d/D and the remaining capacity of the gravity sewer system, as reported by the hydraulic model. Sewersheds that do not meet the capacity standards have been highlighted. Additionally, Figure 16 shows the max d/D throughout the District and Figure 17 shows the remaining capacity in the collection system.

Table 36: Existing Max d/D Summary

Sewershed ID	Max d/D	Minimum Remaining Capacity (EDUs)
WWTP	0.38	265
SPS-1	0.58	**-57**
SPS-2	0.31	257
SPS-4	0.02	1,515
SPS-5	0.23	219
SPS-6	n/a	n/a
SPS-7	0.28	470
SPS-8	0.41	**-16**
SPS-9	**1.00**	709
SPS-10	0.20	516
SPS-11	n/a	n/a
SPS-12	0.14	692
SPS-13	n/a	n/a
SPS-14A	0.04	232
SPS-14B	n/a	n/a
SPS-15	0.07	808
SPS-18	n/a	n/a
SPS-19	n/a	n/a

It should be noted that while SPS-1, SPS-8, and SPS-9 show capacity deficiencies, the max d/D and capacity remaining show conflicting information. This is primarily due to the two capacity calculation methods can conflict under certain circumstances such as pipes with low slope (in the case of SPS-1 and SPS-8), or sewer backing up due to wet well geometry (in the case of SPS-9). Section 3.4.1 goes into further detail about these areas and makes recommendations to resolve this.

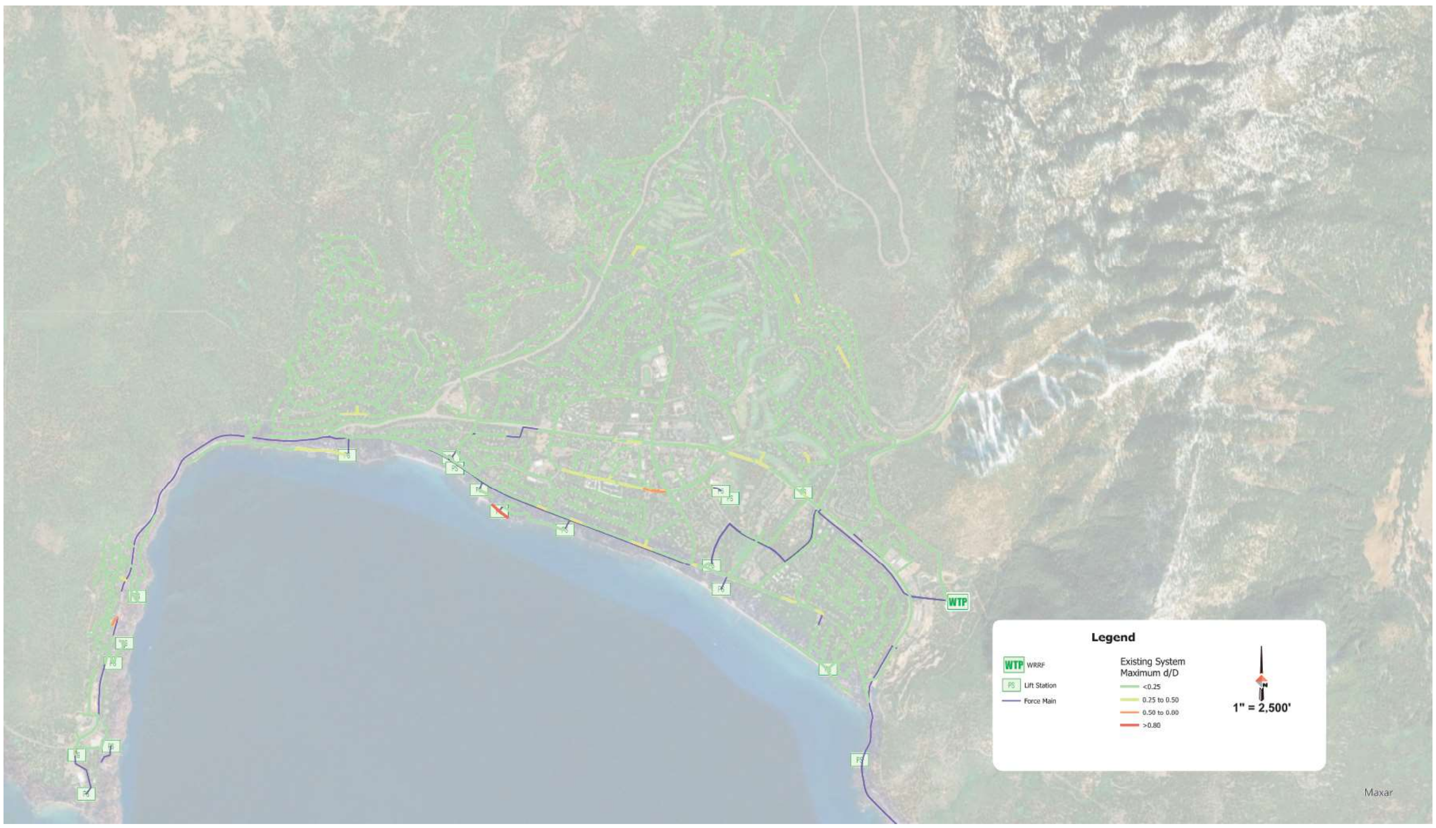

Figure 16: Existing System Max d/D

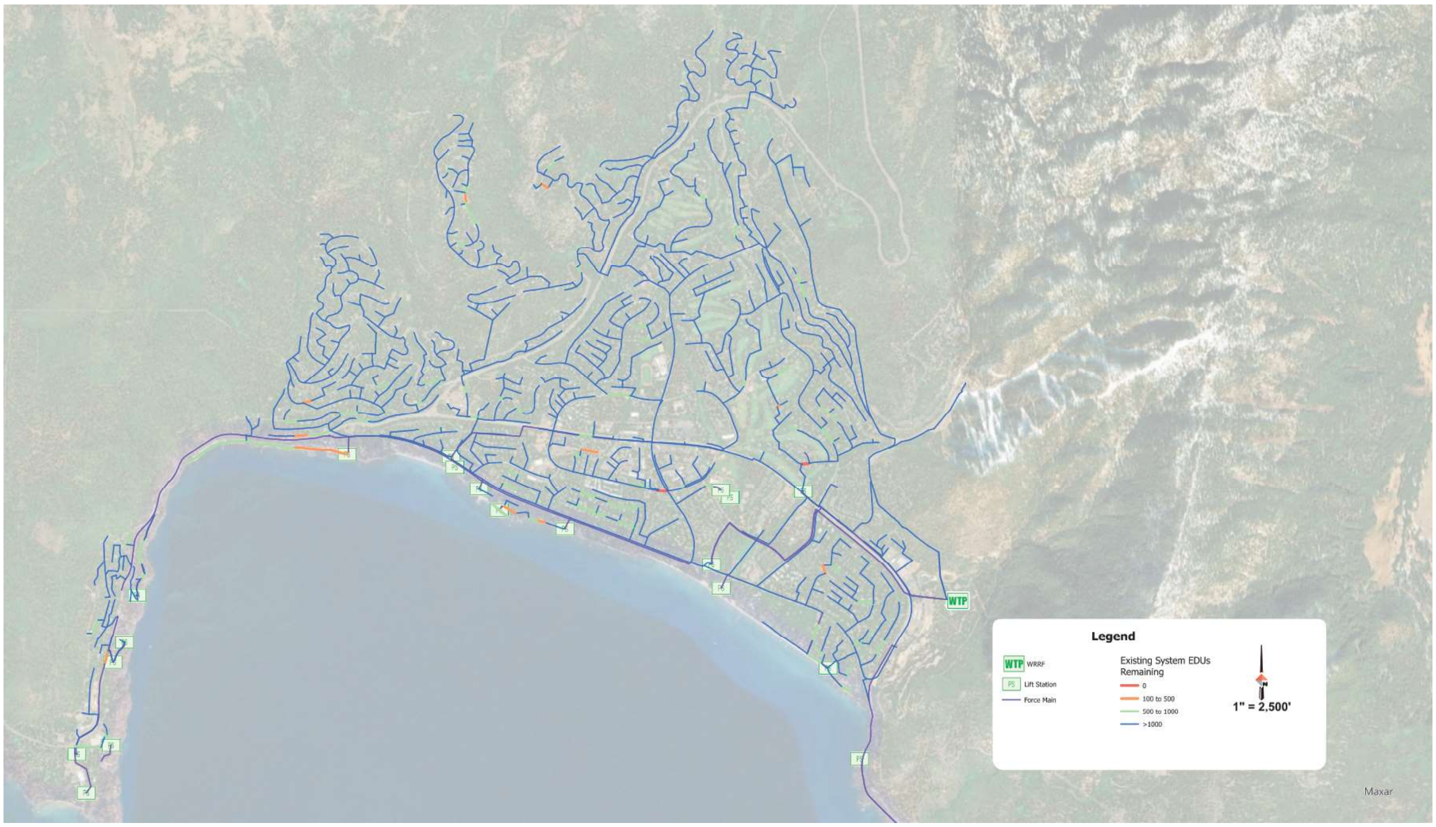

Figure 17: Existing System EDUs Remaining

Table 37 summarizes the maximum velocity observed in the gravity sewer system, as reported by the hydraulic model. Areas that do not meet the minimum velocity requirement have been highlighted. Additionally, Figure 18, shows the maximum velocity throughout the District.

Table 37: Existing Velocity Summary

Sewershed ID	Max Velocity Range (fps)
WWTP	**0.00** to 4.80
SPS-1	**0.00** to 6.04
SPS-2	**0.00 to 1.77**
SPS-4	**0.00**
SPS-5	**0.36 to 1.74**
SPS-6	n/a
SPS-7	**0.00** to 5.11
SPS-8	**0.00** to 8.80
SPS-9	**0.00 to 0.01**
SPS-10	**0.07** to 3.15
SPS-11	n/a
SPS-12	**0.00 to 1.82**
SPS-13	n/a
SPS-14A	**0.00 to 2.23**
SPS-14B	n/a
SPS-15	**0.00 to 1.50**
SPS-18	n/a
SPS-19	n/a

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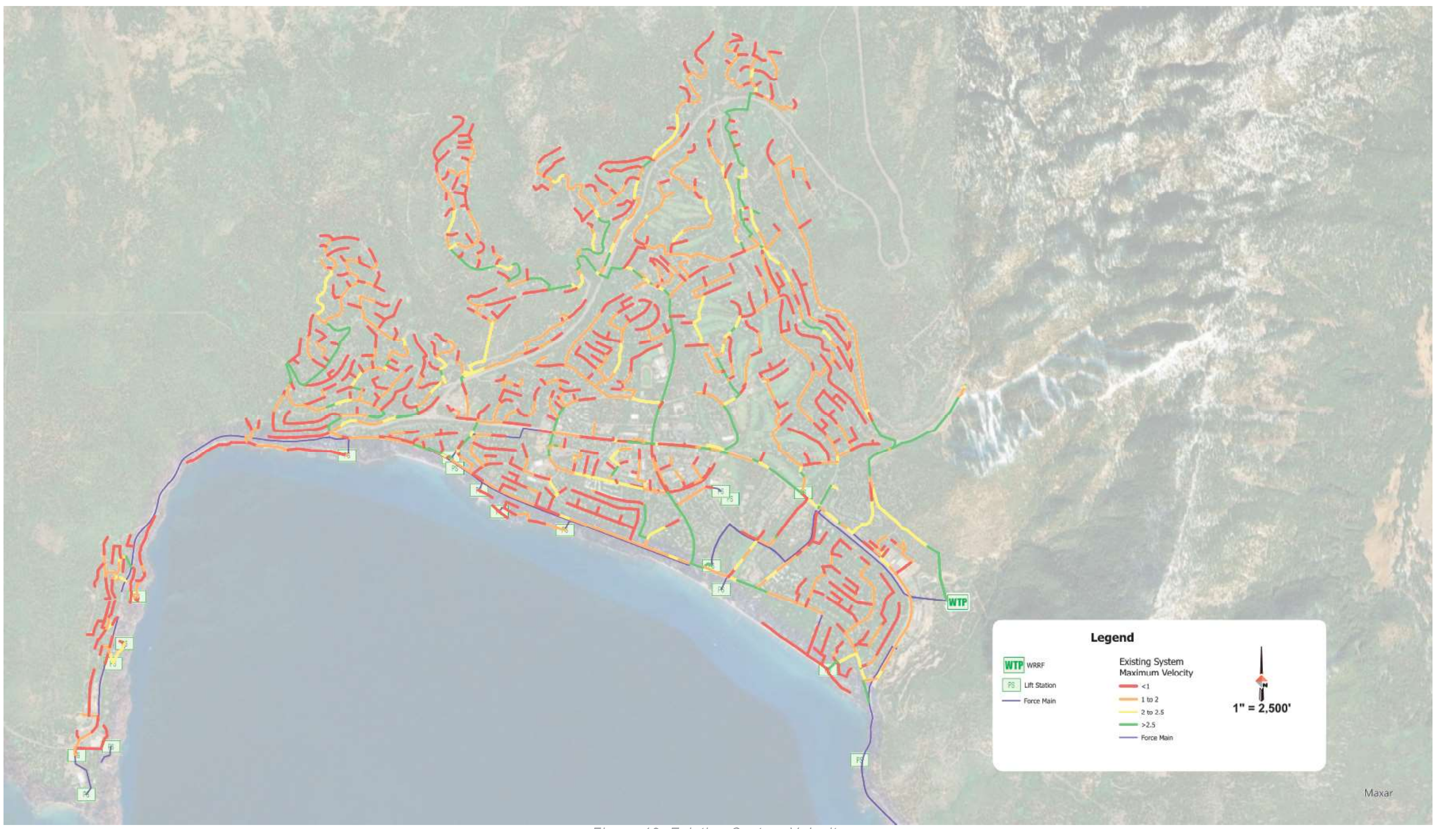

Figure 18: Existing System Velocity

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Table 38 summarizes any manholes within the gravity sewer system that surcharge, as reported by the hydraulic model. Areas with surcharging manholes have been highlighted.

Table 38: Existing Surcharge Summary

Sewershed ID	No. of Surcharged MHs
WWTP	0
SPS-1	0
SPS-2	0
SPS-4	0
SPS-5	0
SPS-6	n/a
SPS-7	0
SPS-8	0
SPS-9	**3**
SPS-10	0
SPS-11	n/a
SPS-12	0
SPS-13	n/a
SPS-14A	0
SPS-14B	n/a
SPS-15	0
SPS-18	n/a
SPS-19	n/a

3.3.1.2 Lift Stations

For each lift station, a pump cycle time figure was created. This graph shows the total cycle time of the lift station pumps based on a selected average flow into the lift station. The graph is created using the wet well geometry, pump flow capacity, and the on/off set points for the pump. The graph also allows a determination of the theoretical shortest pump cycle time at the lift station. Table 39 gives a summary of the pump cycle time at each lift station based on its current average flow, and the theoretical minimum pump cycle time. Lift stations with an average pump cycle times less than 10 minutes have been highlighted. The lift station capacity calcs and pump cycle time figures can be found in Appendix B and Appendix C, respectively.

Table 39: Existing Lift Station Pump Cycle Time Summary

SPS ID	Average Pump Cycle Time (min)	Minimum Pump Cycle Time (min)
SPS-1	63	50
SPS-2	12	2
SPS-4	18	1
SPS-5	33	5
SPS-6	123	7
SPS-7	**9**	3
SPS-8	n/a	n/a
SPS-9	176	10
SPS-10	**5**	2
SPS-11	132	6
SPS-12	58	4
SPS-13	111	7
SPS-14A	138	16
SPS-14B	138	16
SPS-15	163	26
SPS-18	50	8
SPS-19	156	6

Of the pump stations analyzed, SPS-8 is a unique case as the lead pump is on 24 hours a day. The pump utilizes a VFD that will match the incoming flow up to the pump capacity. If at any point the wet well level reaches its high point, the lag pump will then engage to help handle the high flows. As such, there is no average or minimum pump cycle time for this station.

Lift station pump capacity was determined by comparing the estimated peak flow into the lift station to the calculated pump operating point, with the difference being the remaining capacity. Table 40 gives a summary of the pump capacity at each lift station.

Table 40: Existing Lift Station Pump Capacity Summary

SPS ID	Pump Operating Point (gpm)	Peak Flow (gpm)	Capacity Remaining (EDUs)
SPS-1	1,000	813.7	1,503
SPS-2	300	41.8	1,355
SPS-4	280	2.1	1,459
SPS-5	78	9.2	361
SPS-6	80	2.3	408
SPS-7	700	190.4	2,675
SPS-8	1,000	640.2	1,889
SPS-9	50	1.8	253
SPS-10	460	106.8	1,854
SPS-11	80	1.3	413
SPS-12[1]	900	35.8 or 235.8	4,536 or 3,486
SPS-13	200	8.8	1,004
SPS-14A	55	4.9	263
SPS-14B[2]	55	4.9 or 59.9	263 or **-26**
SPS-15	150	16.7	700
SPS-18	25	3.1	115
SPS-19	30	0.1	157

[1] SPS-13 discharges directly into SPS-12. This peak flow accounts for a 200 gpm pumped flow from SPS-13. If SPS-13 is not pumping, SPS-12 sees a peak flow of 35.8 gpm.

[2] SPS-14A discharges directly into SPS-14B. This peak flow accounts for a 55 gpm pumped flow from SPS-14A. If SPS-14A is not pumping, SPS-14B sees a peak flow of 4.9 gpm.

Emergency storage available at each lift station was calculated per WTS-14. The total storage at each lift station is calculated by adding two separate volumes:

1. The total volume the wet well can store before spilling
2. The volume of sewer than can be stored in the collection system up to the elevation before a spill occurs at the lift station

The total emergency storage required was calculated by using the model average flow and multiply it by a peaking factor of 3.5 for a 2-hour duration per WTS-14. These two numbers were then compared, with the difference being the remaining wet well capacity. Table 41 gives a summary of the capacity remaining in each lift station wet well.

Table 41: Existing Lift Station Emergency Storage Capacity Summary

SPS ID	Required Emergency Storage (gal)	Emergency Storage Available (gal)	Capacity Remaining (EDUs)
SPS-1	127,163	54,995	**-3,157**
SPS-2	6,533	5,863	**-29**
SPS-4	321	431	5
SPS-5	1,444	2,508	47
SPS-6	365	162[3]	**-9**
SPS-7	29,750	9,889	**-869**
SPS-8	100,041	2,084	**-4,285**
SPS-9	273	1,454	52
SPS-10	16,683	7,862	**-386**
SPS-11	209	536[4]	14
SPS-12	5,600	5,407	**-8**
SPS-13	1,376	2,424[4]	46
SPS-14A	770	1,819	46
SPS-14B	770	1,707	41
SPS-15	2,604	7,101	197
SPS-18	478	212[5]	**-12**
SPS-19	14	196[5]	8

[3] *Value displayed is the emergency storage within the wet well only. Due to missing elevation information within the upstream manholes and after discussion with IVGID staff, it was determined that collecting elevation data at these locations would not be the best use of our time/budget. The gravity sewer was not included within the model.*

[4] *Value displayed is the emergency storage within the wet well only. Due to missing elevation information within Crystal Bay, the storage within the collection system is unknown. The gravity sewer was not included within the model.*

[5] *Collection system emergency storage = 0. There are no upstream sewer mains.*

Force main capacity was calculated by comparing the existing pump flow rates and the corresponding pipe velocity to the max flow rate that the force main can achieve at a pipe velocity of 8 ft/s. The difference between the two is the capacity remaining. Table 42 gives a summary of the capacity remaining for each lift station force main.

Table 42: Existing Force Main Capacity Summary

SPS ID	Force Main Velocity (ft/s)	Maximum Flow Rate (gpm)	Capacity Remaining (EDUs)
SPS-1: 10-inch	4.5	1,958	4,506
SPS-1: 14-inch	2.3	3,838	14,374
SPS-2	3.4	705	2,126
SPS-4	3.2	705	2,231
SPS-5	2.0	313	1,235
SPS-6	2.0	313	1,225
SPS-7	2.9	1,958	6,605
SPS-8	1.3	6,345	28,057
SPS-9	0.6	705	3,438
SPS-10	2.9	1,253	4,164
SPS-11	2.0	313	1,225
SPS-12	3.7	1,958	5,556
SPS-13	2.3	705	2,651
SPS-14A	1.4	313	1,356
SPS-14B	1.4	313	1,356
SPS-15	0.6	1,958	9,492
SPS-18	0.6	313	1,514
SPS-19	3.06	78	254
SR28 Combined[6]	2.7	6,345	22,285
SR28 Combined[7]	6.7	6,345	5,488
SR28/Lakeshore Blvd Combined[8]	4.6	1,958	4,323
SR28/Lakeshore Blvd Combined[9]	9.3	1,958	**-1,635**

[6] *Analyzed with one pump on at each SPS (SPS-1 and SPS-8)*
[7] *Analyzed with all pumps on at each SPS (3 pumps in series, 6 total at SPS-1 and 2 at SPS-8)*
[8] *Analyzed with one pump on at each SPS (SPS-12, SPS-14B, SPS-15, and SPS-19)*
[9] *Analyzed with all pumps on at each SPS (2 at SPS-12, SPS-14B, SPS-15, and SPS-19)*

3.3.2 Buildout System Capacity

3.3.2.1 Collection System

Table 47 summarizes the maximum d/D and the remaining capacity of the gravity sewer system, as reported by the hydraulic model. Areas that do not meet the capacity standards have been highlighted. Additionally, Figure *19* shows the max d/D throughout the District and Figure 20 shows the remaining capacity in the collection system.

Table 43: Buildout Max d/D Summary

Sewershed ID	Max d/D	Minimum Remaining Capacity (EDUs)
WWTP	0.38	256
SPS-1	0.59	**-89**
SPS-2	0.31	257
SPS-4	0.02	1,515
SPS-5	0.23	219
SPS-6	n/a	n/a
SPS-7	0.28	470
SPS-8	0.42	**-18**
SPS-9	**1.00**	702
SPS-10	0.21	516
SPS-11	n/a	n/a
SPS-12	0.15	692
SPS-13	n/a	n/a
SPS-14A	0.04	232
SPS-14B	n/a	n/a
SPS-15	0.08	806
SPS-18	n/a	n/a
SPS-19	n/a	n/a

It should be noted that while SPS-1, SPS-8, and SPS-9 show capacity deficiencies, the max d/D and capacity remaining show conflicting information. This is primarily due to the two capacity calculation methods can conflict under certain circumstances such as pipes with low slope (in the case of SPS-1 and SPS-8), or sewer backing up due to wet well geometry (in the case of SPS-9). Section 3.4.1 goes into further detail about these areas and makes recommendations to resolve this.

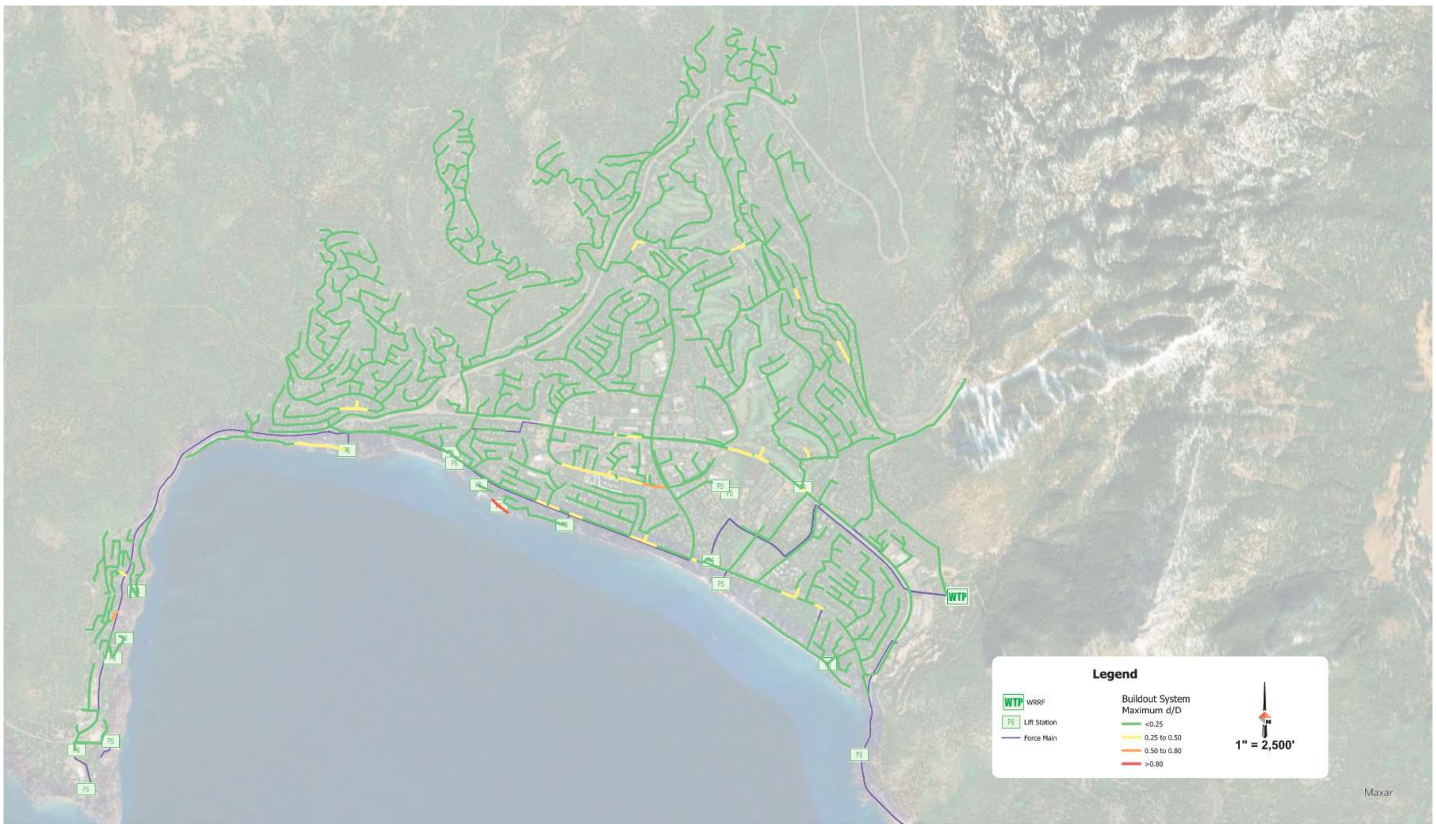

Figure 19: Buildout System Max d/D

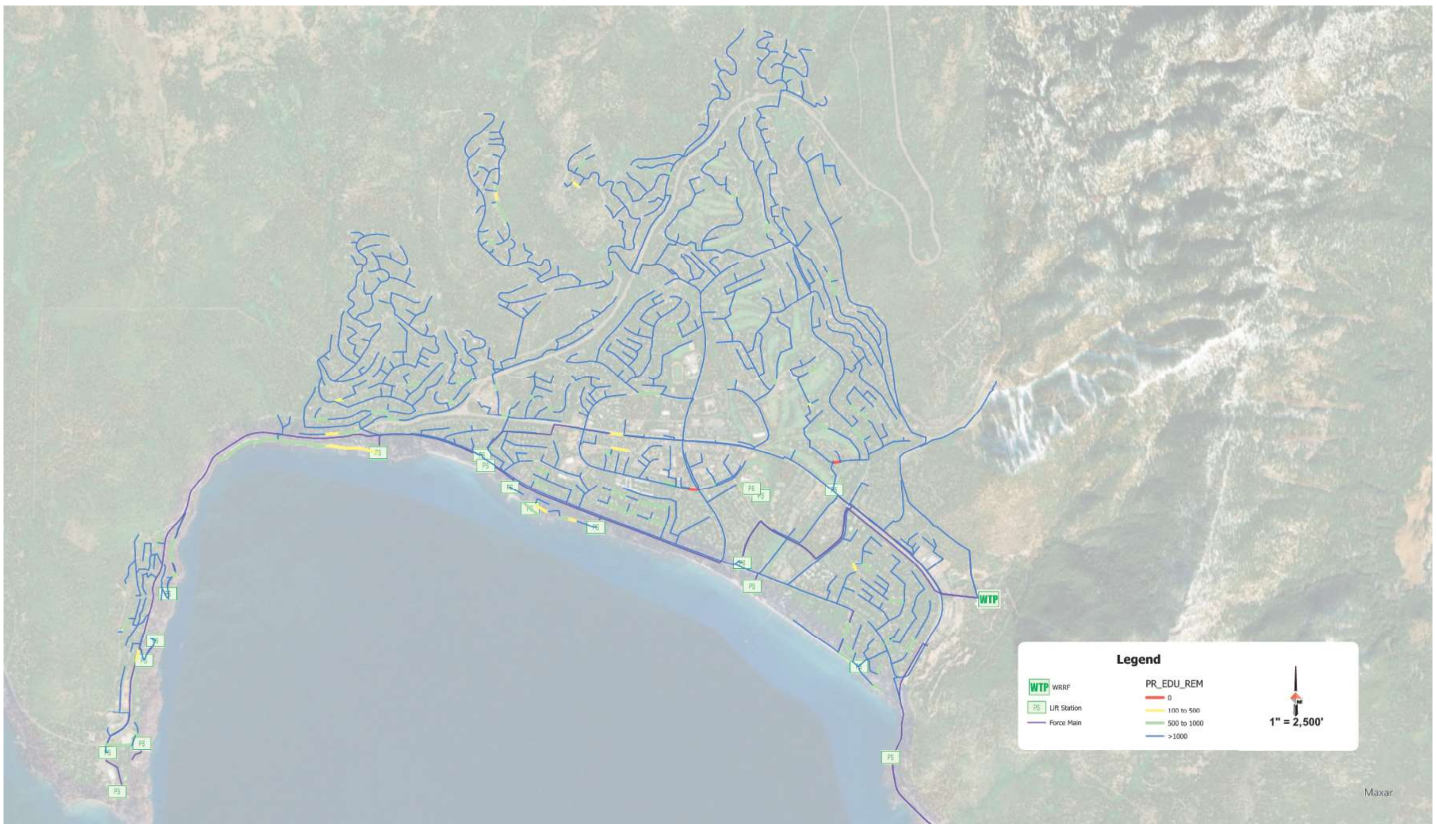

Figure 20: Buildout System EDUs Remaining

Table 44 summarizes the maximum velocity observed in the gravity sewer system, as reported by the hydraulic model. Areas that do not meet the minimum velocity requirement have been highlighted. Additionally, Figure 21, shows the maximum velocity throughout the District.

Table 44: Buildout Velocity Summary

Sewershed ID	Max Velocity Range (fps)
WWTP	**0.00** to 5.11
SPS-1	**0.00** to 6.12
SPS-2	**0.00 to 1.77**
SPS-4	**0.00**
SPS-5	**0.47 to 1.74**
SPS-6	n/a
SPS-7	**0.00** to 5.15
SPS-8	**0.00** to 8.91
SPS-9	**0.00 to 0.02**
SPS-10	**0.07** to 3.19
SPS-11	n/a
SPS-12	**0.00 to 1.99**
SPS-13	n/a
SPS-14A	**0.00 to 2.23**
SPS-14B	n/a
SPS-15	**0.00 to 1.99**
SPS-18	n/a
SPS-19	n/a

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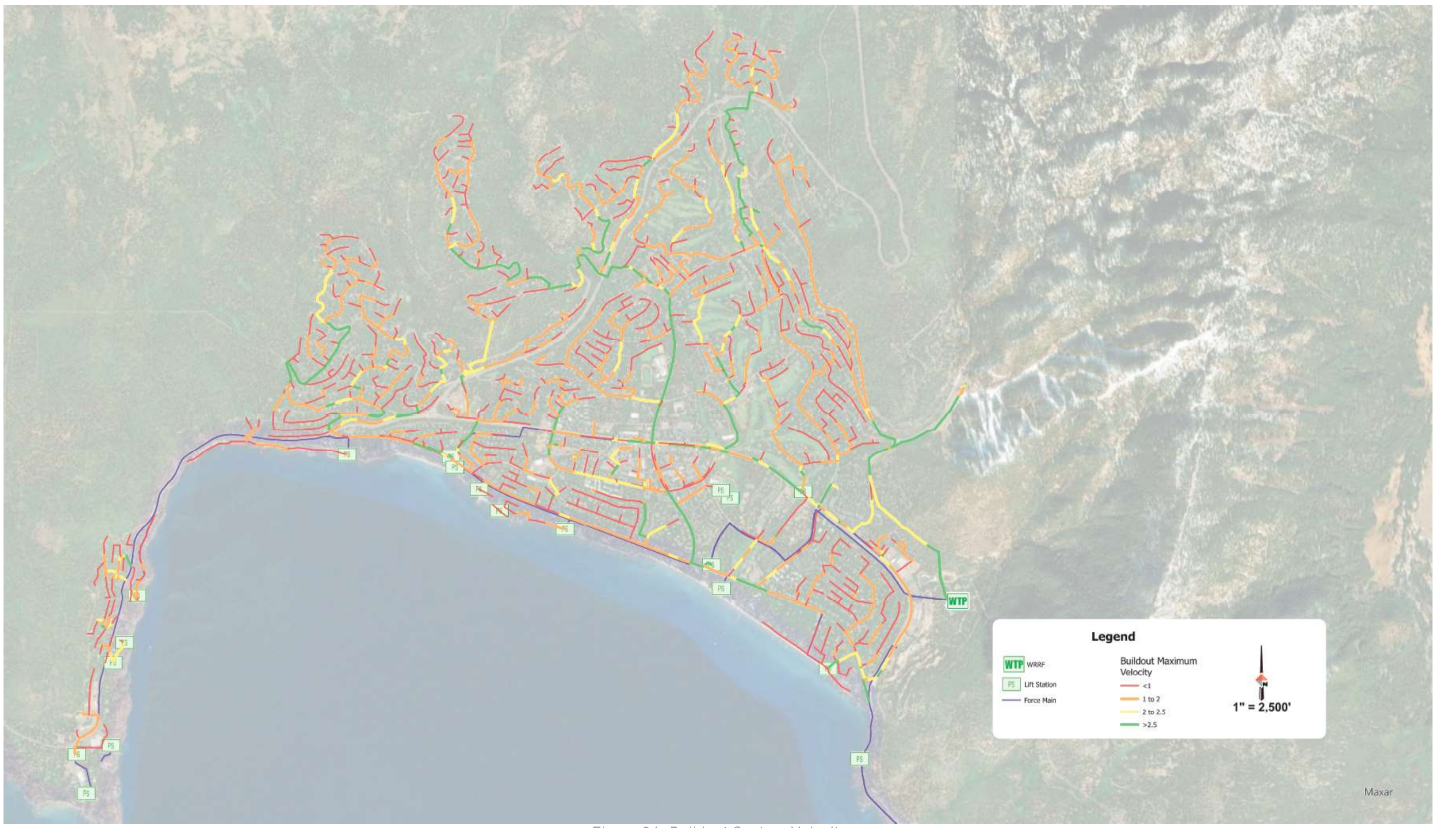

Figure 21: Buildout System Velocity

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Table 45 summarizes any manholes within the gravity sewer system that surcharge, as reported by the hydraulic model. Sewersheds with surcharging manholes have been highlighted.

Table 45: Buildout Surcharge Summary

Sewershed ID	No. of Surcharged MHs
WWTP	0
SPS-1	0
SPS-2	0
SPS-4	0
SPS-5	0
SPS-6	n/a
SPS-7	0
SPS-8	0
SPS-9	**3**
SPS-10	0
SPS-11	n/a
SPS-12	0
SPS-13	n/a
SPS-14A	0
SPS-14B	n/a
SPS-15	0
SPS-18	n/a
SPS-19	n/a

3.3.2.2 Lift Stations

For each lift station, a pump cycle time figure was created. This graph shows the total cycle time of the lift station pumps based on the average flow into the lift station. The graph also allows a determination of the theoretical shortest pump cycle time at the lift station. Table 46 gives a summary of the average buildout pump cycle time at each lift station. The lift station capacity calcs and pump cycle time figures can be found in Appendix B and Appendix C respectively.

Of the pump stations analyzed, SPS-8 is a unique case as the lead pump is on 24 hours a day. The pump utilizes a VFD that will match the incoming flow up to the pump capacity. If at any point the wet well level reaches its high point, the lag pump will then engage to help handle the high flows. As such, there is no average or minimum pump cycle time for this station.

Table 46: Buildout Lift Station Pump Cycle Time Summary

SPS ID	Average Pump Cycle Time (min)
SPS-1	62
SPS-2	12
SPS-4	18
SPS-5	33
SPS-6	123
SPS-7	**9**
SPS-8	n/a
SPS-9	176
SPS-10	**4**
SPS-11	132
SPS-12	52
SPS-13	90
SPS-14A	115
SPS-14B	123
SPS-15	147
SPS-18	50
SPS-19	156

Of the pump stations analyzed, SPS-8 is a unique case as the lead pump is on 24 hours a day. The pump utilizes a VFD that will match the incoming flow up to the pump capacity. If at any point the wet well level reaches its high point, the lag pump will then engage to help handle the high flows. As such, there is no average or minimum pump cycle time for this station.

Lift station pump capacity was determined by comparing the estimated peak flow into the lift station to the calculated pump operating point, with the difference being the remaining capacity. Table 47 gives a summary of the pump capacity at each lift station.

Table 47: Buildout Lift Station Pump Capacity Summary

SPS ID	Pump Operating Point (gpm)	Peak Flow (gpm)	Capacity Remaining (EDUs)
SPS-1	1,000	828.7	1,424
SPS-2	300	41.8	1,355
SPS-4	280	2.1	1,459
SPS-5	78	9.3	360
SPS-6	80	2.5	407
SPS-7	700	196.0	2,646
SPS-8	1,000	662.6	1,771
SPS-9	50	1.8	253
SPS-10	460	121.7	1,776
SPS-11	80	1.3	413
SPS-12[10]	900	42.7 or 242.7	4,500 or 3,450
SPS-13	200	11.8	988
SPS-14A	55	5.6	259
SPS-14B[11]	55	5.3 or 60.3	261 or **-28**
SPS-15	150	19.2	686
SPS-18	25	3.1	115
SPS-19	30	0.1	157

[10] SPS-13 discharges directly into SPS-12. This peak flow accounts for a 200 gpm pumped flow from SPS-13. If SPS-13 is not pumping, SPS-12 sees a peak flow of 42.7 gpm.

[11] SPS-14A discharges directly into SPS-14B. This peak flow accounts for a 55 gpm pumped flow from SPS-14A. If SPS-14A is not pumping, SPS-14B sees a peak flow of 5.3 gpm.

Emergency storage available at each lift station was calculated by adding two separate volumes:

1. The total volume the wet well can store before spilling
2. The volume of sewer than can be stored in the collection system up to the elevation before a spill occurs at the lift station

The total emergency storage required was calculated by using the model average flow and multiply it by a peaking factor of 3.5 for a 2-hour duration per WTS-14. These two numbers were then compared, with the difference being the remaining wet well capacity. Table 48 gives a summary of the capacity remaining in each lift station wet well.

Table 48: Buildout Lift Station Emergency Storage Capacity Summary

SPS ID	Required Emergency Storage (gal)	Emergency Storage Available (gal)	Capacity Remaining (EDUs)
SPS-1	129,500	54,995	**-3,259**
SPS-2	6,533	5,863	**-29**
SPS-4	326	431	5
SPS-5	1,458	2,508	46
SPS-6	393	162[12]	**-10**
SPS-7	30,625	9,889	**-907**
SPS-8	103,541	2,084	**-4,438**
SPS-9	273	1,454	52
SPS-10	19,016	7,862	**-488**
SPS-11	209	536[13]	14
SPS-12	6,679	5,407	**-56**
SPS-13	1,846	2,42413[13]	25
SPS-14A	875	1,819	41
SPS-14B	834	1,707	38
SPS-15	3,004	7,101	179
SPS-18	478	212[14]	**-12**
SPS-19	14	196[14]	8

[12] Value displayed is the emergency storage within the wet well only. Due to missing elevation information within the upstream manholes and after discussion with IVGID staff, it was determined that collecting elevation data at these locations would not be the best use of our time/budget. The gravity sewer was not included within the model.

[13] Value displayed is the emergency storage within the wet well only. Due to missing elevation information within Crystal Bay, the storage within the collection system is unknown. The gravity sewer was not included within the model.

[14] Collection system emergency storage = 0. There are no upstream sewer mains.

Force main capacity was calculated by comparing the existing pump flow rates and the corresponding pipe velocity to the max flow rate that the force main can achieve at a pipe velocity of 8 ft/s. The difference between the two is the capacity remaining. Table 49 gives a summary of the capacity remaining for each lift station force main.

Table 49: Buildout Force Main Capacity Summary

SPS ID	Force Main Velocity (ft/s)	Maximum Flow Rate (gpm)	Capacity Remaining (EDUs)
SPS-1: 10-inch	4.5	1,958	4,506
SPS-1: 14-inch	2.3	3,838	14,374
SPS-2	3.4	705	2,126
SPS-4	3.2	705	2,231
SPS-5	2.0	313	1,235
SPS-6	2.0	313	1,225
SPS-7	2.9	1,958	6,605
SPS-8	1.3	6,345	28,057
SPS-9	0.6	705	3,438
SPS-10	2.9	1,253	4,164
SPS-11	2.0	313	1,225
SPS-12	3.7	1,958	5,556
SPS-13	2.3	705	2,651
SPS-14A	1.4	313	1,356
SPS-14B	1.4	313	1,356
SPS-15	0.6	1,958	9,492
SPS-18	0.6	313	1,514
SPS-19	3.06	78	254
SR28 Combined[15]	2.7	6,345	22,285
SR28 Combined[16]	6.7	6,345	5,488
SR28/Lakeshore Blvd Combined[17]	4.6	1,958	4,323
SR28/Lakeshore Blvd Combined[18]	9.3	1,958	**-1,635**

[15] *Analyzed with one pump on at each SPS (SPS-1 and SPS-8)*
[16] *Analyzed with all pumps on at each SPS (3 pumps in series, 6 total at SPS-1 and 2 at SPS-8)*
[17] *Analyzed with one pump on at each SPS (SPS-12, SPS-14B, SPS-15, and SPS-19)*
[18] *Analyzed with all pumps on at each SPS (2 at SPS-12, SPS-14B, SPS-15, and SPS-19)*

3.4 System Deficiencies and Operational Challenges

Operational staff regularly perform scheduled maintenance on the sewer collection system including hydroflushing and camera portions of the system annually. Figure 22 is an overview map of the manhole structures and pipes identified as needing rehabilitation or spot repairs as identified from CCTV footage. Figure 23 shows the seasonal/monthly hydroflushing schedule for the collection system as well as areas of the system that have been hydroflushed in the past.

In addition to areas identified by video, operations staff report that the system has experienced high levels of grease. Grease levels are such that SPS-1 requires monthly grease cleaning, as well as utilizing the split wet well configuration in order to trap grease and not convey it to the WRRF.

The system also has many smaller collection lines that are located not in the street, but in small easements adjacent to residences. Many of these easements have become overgrown leading to root penetration in manholes and spilling.

As explained in Section 2.0, the sewer system is missing the PACP scoring required to complete a full condition assessment of the system. However, once future efforts to video and score the mains and manholes of the gravity system and a condition assessment is completed, it is recommended that the District begin a sewer main and manhole rehabilitation and replacement program.

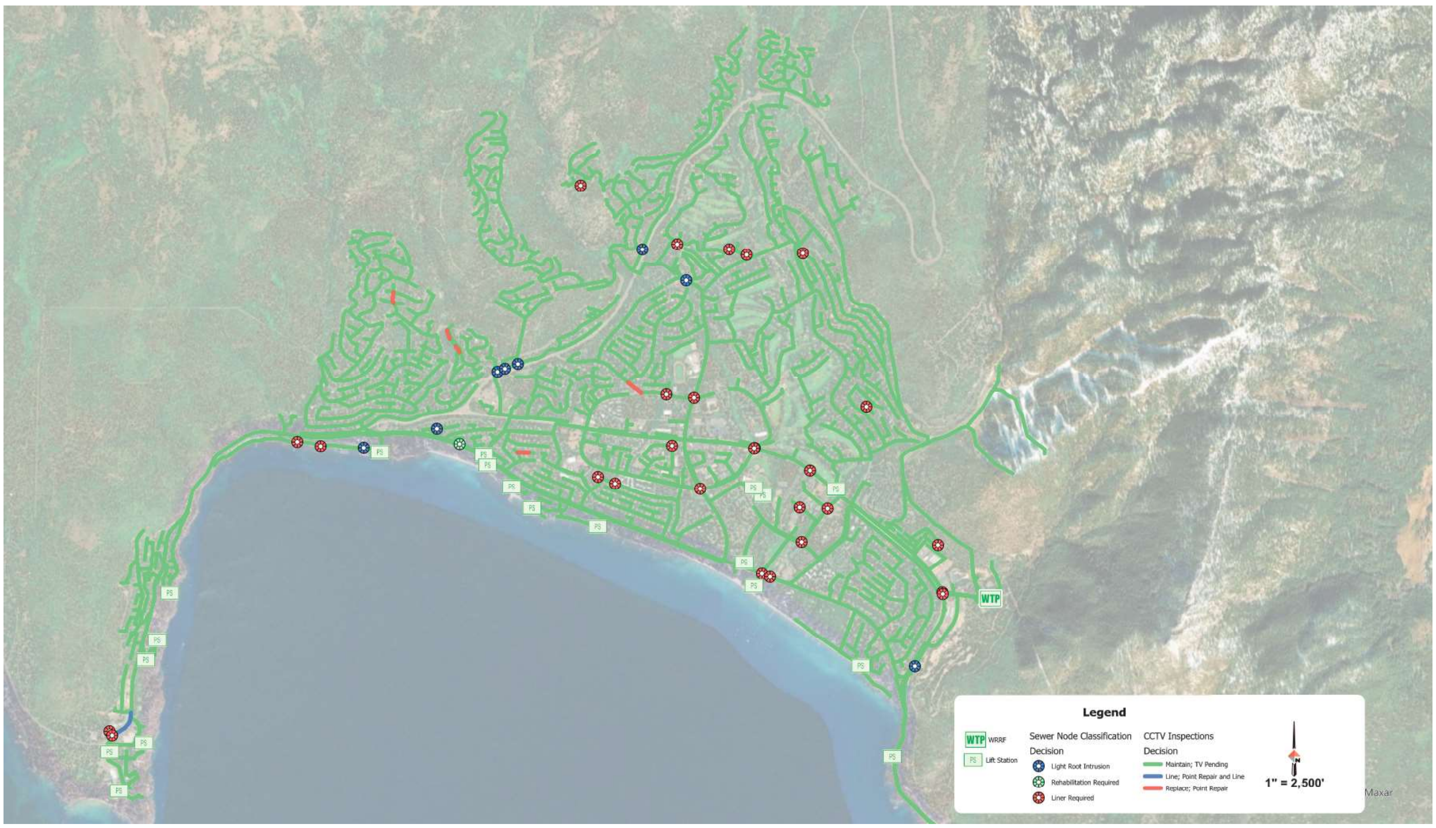

Figure 22: Collection System CCTV Decision Map

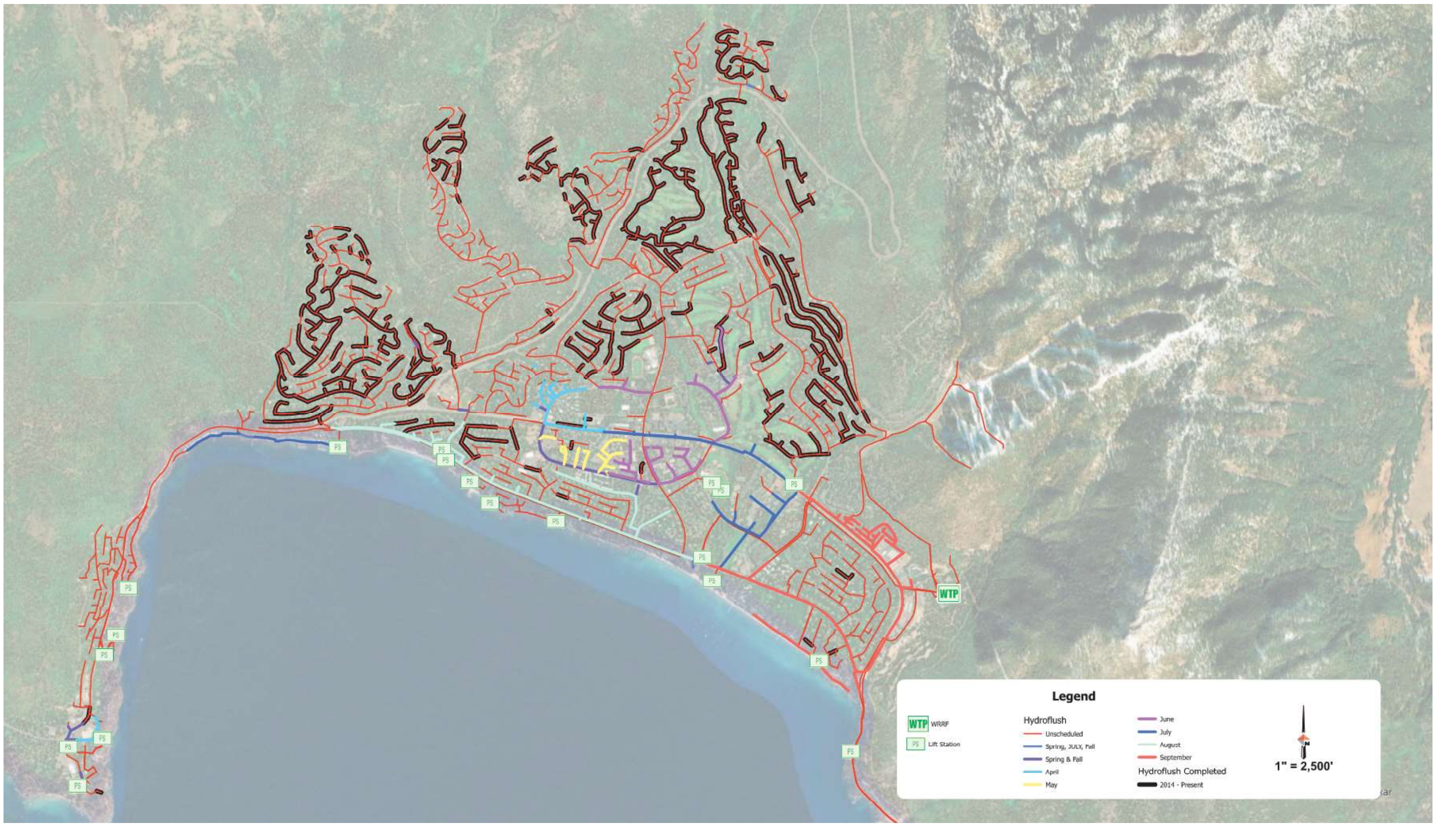

Figure 23: Pipe Hydroflushing Schedules

3.4.1 Gravity Sewer Capacity

As shown in Section 3.3, several sewersheds have pipes that exceed the Washoe County CSD d/D standard of 0.80. Likewise, there are multiple pipes that have zero capacity remaining and manholes that surcharge. Table 50 gives a summary of the number of manholes and pipes within each sewershed that exceed their capacity. An explanation for each of the manhole and pipe deficiencies is provided in the sections below.

Table 50: Sewersheds with Capacity Deficiencies

Sewershed ID	No. of Max d/D Deficiencies	No. of Capacity Deficiencies	No. of Surcharged Manholes
SPS-1	0	1	0
SPS-8	0	1	0
SPS-9	3	0	3

3.4.1.1 SPS-1 Sewershed

The hydraulic model reported that one 8-inch pipe segment exceeded its capacity within the SPS-1 sewershed. This pipe is located to the east of the Southwood Boulevard and Village Boulevard intersection, between manhole 001-1150 and 001-1180. Although this pipe has a maximum d/D of 0.54 and does not exceed a d/D of 0.80, it is in exceedance of its capacity due to the flat nature of its slope, 0.045%. Within the model, the hydraulic grade line (HGL) at the inlet is higher than the HGL at the outlet, 6,332.170 feet versus 6,331.685 feet. The model averages these two sewage depths (0.362 feet) and compares it back to the pipe diameter of 0.67 feet to calculate the d/D value of 0.54. However, the d/D at the inlet of the pipe is 0.82 and violates the criteria described in Section 3.3.

The pipe invert elevations at this location were calculated by taking the measure down depth, provided in the District's sewer GIS, and subtracting it from a ground surface elevation raster. This capacity deficiency could be the result of incorrect elevations within the hydraulic model. This area requires further field investigation to determine what the slope of this pipe is.

3.4.1.2 SPS-8 Sewershed

The hydraulic model reported that one 8-inch pipe segment exceeded its capacity within the SPS-8 sewershed. This pipe is located to the west of the Sand Iron Drive and 4th Green Drive intersection, between manhole 008-1375 and 008-1377. Similar to what was described for the SPS-1 capacity deficiency, the capacity issues at the pipe in SPS-8 are due to the flat nature of its slope, 0.00099%. The pipe invert elevations at this location were calculated by taking the measure down depth, provided in the District's sewer GIS, and subtracting it from a ground surface elevation raster. This capacity deficiency could be the result of incorrect elevations within the hydraulic model. This area requires further field investigation to determine what the slope of this pipe is.

3.4.1.3 SPS-9 Sewershed

The hydraulic model reported that three 6-inch pipe segments exceeded their capacity, and three manholes were considered surcharged. These pipes and manholes are located to the south of Shoreline Circle with corresponding manhole IDs of 009-1592A, 009-1592, and 009-1593. These manholes and pipes directly contribute and connect to SPS-9. The District provided information that stated that the gravity sewer enters the wet well at the invert of the structure (6,225.50 feet). Elevation information was not available for the upstream manholes nor pipes and this section of the model was built assuming that they were constructed with a slope of 0.4% (10 State Standards minimum slope).

When pump one is in use at SPS-9, the pump startup depth is 3.28 feet and shutoff depth is 1.92 feet. Similarly, when pump two is in use, the pump startup depth is 3.70 feet and shutoff depth is 2.28 feet. Given these operating depths and the minimal slopes used to construct the upstream system, the sewage depths in SPS-9 cause backups into the upstream system, fully inundating the pipes and partially inundating the manholes. This capacity and surcharging deficiency could be the result of incorrect elevations within the hydraulic model. This area requires further field investigation to determine what the slope and elevations of the surrounding infrastructure are.

3.4.2 Gravity Sewer Velocity

As shown in Section 3.3, several sewersheds have pipes that do not have a velocity greater than or equal to 2.5 ft/s. Typically, additional flow as a system grows through development will allow low velocity areas to reach the 2.5 ft/s barrier. However, the District is expected to see a limited addition of EDUs in the near to long term future and the sewer flows will remain close to their current values. As Figure 21 displays the results from the buildout model scenario, future development cannot be relied upon to increase flow velocities. It is recommended that the District continue to inspect and flush gravity mains as necessary to clear any sediment or debris built up within the main.

The Washoe County CSD standard states that pipe velocities must maintain a minimum of 2.5 ft/s when flowing half full. Further investigation was completed to try to identify areas of the collection system that do not have adequate slope. A pipe with inadequate slope is herein defined as a pipe that has a flow velocity of less than 2.5 ft/s when flowing half full. Table 51 gives a summary of the number of pipes within each sewershed that have inadequate slope. Figure 24 is a map showing the location of these pipes throughout the collection system. Pipes with inadequate slope, identified below, could be the result of incorrect elevations within the hydraulic model. These areas require further field investigation to determine if the slopes within the hydraulic model are accurate.

Table 51: Summary of Inadequate Pipe Slope by Sewershed

Sewershed ID	No. of Pipes with Inadequate Slope
WWTP	35
SPS-1	60
SPS-2	23
SPS-4	0
SPS-5	3
SPS-6[19]	n/a
SPS-7	25
SPS-8	29
SPS-9	3
SPS-10	22
SPS-11[20]	n/a
SPS-12	4
SPS-13[20]	n/a
SPS-14A	1
SPS-14B[21]	n/a
SPS-15	3
SPS-18	n/a
SPS-19[21]	n/a

[19] Due to missing elevation information within the upstream manholes and after discussion with IVGID staff, it was determined that collecting elevation data at these locations would not be the best use of our time/budget. The gravity sewer was not included within the model.

[20] Due to missing elevation information within Crystal Bay, the gravity sewer was not included within the model.

[21] There are no upstream sewer mains.

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