Lakevale Estates Pump Station Gravity System Modeling & Hydraulic Improvements

Preliminary Alternatives Identification Technical Memorandum

February 2022

Fairfax County Department of Public Works and Environmental Services

Task Order CH 2018-08

Table of Contents

List of Appendices

List of Tables

List of Figures

Abbreviations

AC	Asbestos Cement
ADF	Average Daily Flow
AFT Fathom	Applied Flow Technology Fathom
BBU	Basement Backup
BWF	Base Wastewater Flow
C	Hazen-Williams Roughness Coefficient
CCTV	Closed Circuit Television
CI	Cast Iron
CIPP	Cured in Place Pipe
CONC	Concrete
County	Fairfax County
CTC	Certificate to Construct
DPWES	Fairfax County Department of Public Works and Environmental Services
DS	Downstream
DWF	Dry Weather Flow
EPA	Environmental Protection Agency
EX	Existing
ft	Feet
GIS	Geographic Information Systems
gpd	Gallons per Day
gpm	Gallons per Minute
GWI	Ground Water Intrusion
HGL	Hydraulic Grade Line
ID	Internal Diameter
LF	Linear Feet
MGD	Million Gallons per Day
MH	Manhole
MOT	Maintenance of Traffic
MUTCD	Manual on Uniform Traffic Control Devices
MWCOG	Metropolitan Washington Council of Government
NPSHr	Net Positive Suction Head required
O&M	Operation and Maintenance
PER	Preliminary Engineering Report
PF	Peaking Factor
PFM	Public Facilities Manual
PS	Pump Station
psi	Pounds per Square Inch
RDII	Rainfall Derived Infiltration and Inflow
RPA	Resource Protection Area
S&L	Smith & Loveless
SCADA	Supervisory Control & Data Acquisition
SSOAP	Sanitary Sewer Overflow Analysis and Planning
TAZ	Transportation Analysis Zones
TDH	Total Dynamic Head
TM	Technical Memorandum
VDEQ	Virginia Department of Environmental Quality
VSCAT	Virginia Sewage Collection and Treatment
WDCD	Fairfax County Wastewater Design and Construction Division
WWF	Wet Weather Flow

Executive Summary

Jacobs was contracted by Fairfax County to evaluate the hydraulic performance of the existing Lakevale Estates Pump Station (PS) and downstream gravity sewer system to identify potential improvements needed to address long-term hydraulic capacity needs. This technical memorandum (TM) discusses the project's background and purpose, setting, compiled data and assumptions, project specific analyses completed to date, and evaluation of potential alternatives, in advance of development of a Preliminary Engineering Report (PER).

This TM encompasses two major parts: 1) pump station and gravity system hydraulic analysis and 2) improvements alternative analysis. Hydraulic analysis was performed to confirm existing conditions, review available flow data, model pump station and downstream gravity system and ultimately recommend a peak influent flow for the Lakevale Estates PS. The results of hydraulic calculations are summarized in the main body of the TM. Detailed analyses and calculations can be found in Appendices A and B of this TM.

The second part of this TM includes discussions about potential alternatives for downstream improvements. The purpose of this alternative analysis is to provide the County with information pertaining to advantages and disadvantages of three alternatives for the Lakevale Estates PS gravity system hydraulic improvements and to identify recommended alternatives for preliminary design. This TM includes a description of the proposed alternatives, findings and evaluation of alternative selection factors, and development of rough order-of-magnitude costs for each alternative. Based on the County's preference and feedback regarding the recommended alternatives, an analysis for the selected alternative will be conducted in the PER.

Pump Station and Gravity System Hydraulic Analysis

The Lakevale Estates PS is located at 10209 Kenbrooke Court in Vienna, Virginia, within the Lakevale Estates subdivision. This pump station collects flow from a mostly residential sewershed and discharges through a lined 8-inch Cast Iron force main that conveys flow through the Lakevale Estates Community Association's open space, past the Lakevale Community Center, and discharges into the existing 10-inch gravity sewer system on Newton Street as shown in Figure 1.

A hydraulic analysis was conducted to identify alternatives to improve the existing pump station and gravity system operation. Fairfax County provided Jacobs with flow monitoring, supervisory control & data acquisition (SCADA) and other relevant hydraulic data to perform this task. The key results of this analysis are summarized in Table 1.

Table 1 – System Capacity and Project Performance Analysis Results

Parameters	Values
Estimated existing downstream gravity system capacity	580 gpm
10 ¾ -inch impeller pumping capacity (single pump)	484 gpm
Average daily flow rate	226 gpm
Future average daily flow rate *(5% growth)*	237 gpm
Peak design flow rate *(226 gpm x 2.5 PF)*	565 gpm
Future peak design flow rate *(237 gpm x 2.5 PF)*	593 gpm

Three sources of data were analyzed to determine the average daily flow: SCADA data at the pump station wet well, downstream flow meter data, and the provided Infoworks ICM model calibrated for dry weather flow. Based on the flow analysis, the existing average daily flow rate was determined to be 226 gpm. Jacobs also analyzed the future growth projections developed by the Metropolitan Washington Council of Government (MWCOG). The projected growth was determined to be 5% over the next 24 years. Therefore, applying a 5% growth factor based on the MWCOG population projections and typical pump station peaking factor of 2.5 produces a peak flow of 593 gpm.

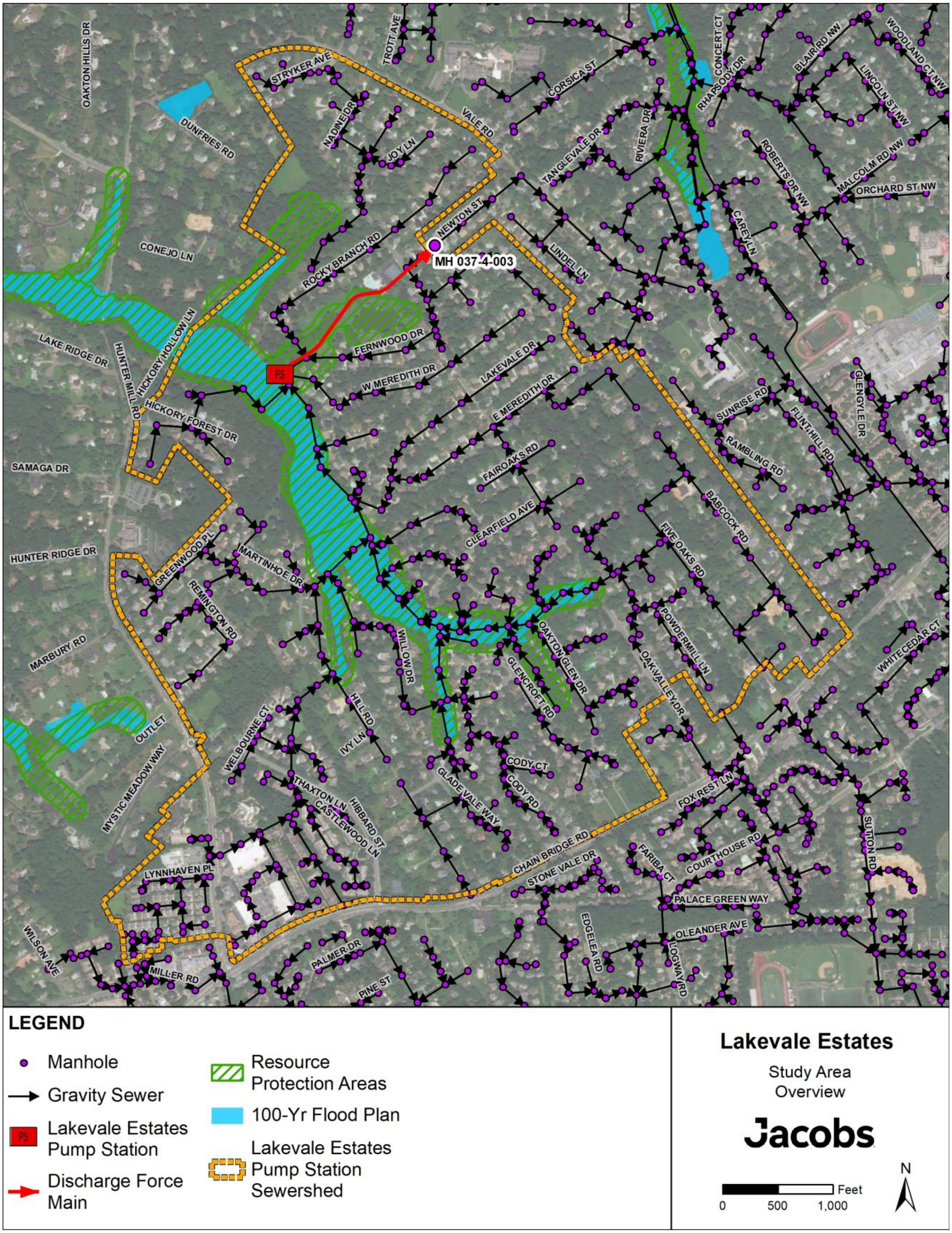


Figure 1 - Lakevale Estates Pump Station Gravity System – Overall Site Plan

Alternative Analysis

The project team evaluated and developed alternatives for the pump station and the downstream receiving sewer system. The project team considered and screened many alternatives and identified the best course of actions in each of the areas. Three alternatives (DS-1 through DS-3) were evaluated. The intent for the three downstream alternatives is to improve hydraulic capacity and provide relief for the downstream gravity system. Cost ranges were established for low, most likely, and high ranges based on uncertainty in quantities, geotechnical conditions, utility constraints, and unit prices. Other alternatives considered, but not found to be technically feasible or reasonable, along with the reasons they were eliminated from further analysis, are included Table 18 of this TM.

Section 4 of this TM presents the project alternatives, describes the development of the project alternatives, evaluates the alternatives for consistency with stated project objectives, summarizes and compares the findings, in order to make recommendations on the feasible alternative.

The evaluation to select a preferred alignment was based on the selection categories listed below. The project team placed each of the categories into a matrix to evaluate a preferred project alternative.

- Community Impact
- Constructability
- Environmental Impact
- Operations & Maintenance
- Cost

Using the selected categories and key factors applicable within each category, the project team conducted an analysis of the feasible alternatives. The analyses are presented in Section 5. Once the analyses were completed, each alternative was given a quantitative numerical score, ranging from 4 points for more favorable characteristics to 1 point for relatively negative/poor characteristics. In addition, rough order-of-magnitude cost estimates were developed for each alternative and are included in Appendix C.

The following paragraphs provide a brief description of the highest ranked alternative. Detailed descriptions for each alternative are provided in Section 4 of this TM, so that reviewers may evaluate their comparative merits. Once the apparent best alternative has been established, work related to the PER and other actual implementation activities will commence.

Alternative DS -3

Based on the aforementioned criteria, alternative DS-3 is the most favored alternative for downstream gravity sewer system improvements. This alternative includes provisions for a new sanitary sewer force main in parallel to the existing gravity sewer system in Newton Street. Pump station downstream hydraulic capacity will be upgraded by extending the force main to a point where the gravity system is no longer constrained.

Preliminary evaluations show that the existing pump model equipped with the largest (12-inch) impeller can discharge up to 593 gpm to a distance between manhole 038-3-011 and 038-3-010, approximately 2,600 linear feet from its current discharge location (manhole 037-4-003). Implementation of this alternative would allow for concurrent operation of two pumps during peak influent flow events if one pump cannot accommodate influent flows. The rough order-of-magnitude cost estimate for extending the force main is approximately $3.5 million. A detailed cost breakdown of these alternatives is included in Appendix C.

1. Introduction

1.1 Purpose

The purpose of this Task Order is to prepare a Preliminary Engineering Report (PER) for the hydraulic improvements to the Lakevale Estates PS and the existing gravity sewer system downstream of the Lakevale Estates PS to identify the improvements needed to address sewer surcharging downstream and upstream of the existing pump station. This memorandum documents the preliminary evaluation of the system and evaluation of potential alternatives. The results will be discussed with the County to select a preferred alternative, which will then be further documented in the PER.

1.2 Background

The Lakevale Estates Pump Station is located at 10209 Kenbrooke Court in Vienna, Virginia (grid map number 037-4-004), within the Lakevale Estates subdivision, in a low area adjacent to a tributary to Rocky Branch. The pump station discharges through a lined 8-inch force main that conveys flow through the Lakevale Estates Community Association's open space, past the Lakevale Community Center, and discharges into the 10-inch gravity sewer system on Newton Street. The gravity sewer system runs along Newton Street, turns southeast on Vale Service Road and then northeast between houses before dropping into a stream valley along a tributary to Piney Branch. Approximately 0.9 miles downstream of the force main the 10-inch sewer transitions to a 12-inch sewer and finally discharges into the downstream trunk sewer. It is understood that the downstream trunk sewer is included in a capital improvement project, and therefore, this is the downstream end of the project area. Figure 1 provides a map that shows the pump station and sewers in the project area.

The watershed consists of a residential subdivision in addition to a small strip mall (Oakton Shopping Center) and miscellaneous community building. The residential lots generally vary from 0.25-1 acres in size. The Oakton Shopping Center and adjacent parking areas are approximately 16 acres in size and nearly 90% of commercial tenants operate retail facilities, including a grocery store, pharmacy, and furniture store. There are also several eat-in restaurants. The remaining 10% of tenants occupy office space. Approximately 39 parcels (76 acres) in the study area are within the 100-year floodplain.

The following table summarizes major constructions and upgrades at the Lakevale Estates Pump Station:

Table 2 - Pump Station Upgrade History

Year	Description
1965	Newton street gravity sewers were constructed downstream of PS
1966	Original pump station was constructed as part of surrounding development. The original station was equipped with two 415 gallon per minute (gpm) pumps.
1995	Pump station electrical and control system upgrades
2012	Force main and gravity sewers on Newton St. were lined utilizing cured in place pipe (CIPP) technique
2017	Gravity sewers along Vale Road were lined utilizing CIPP
2018	Pump station rehabilitation completed (replaced existing pumps with two 619 gpm pumps)
2019/20	Installed smaller impeller on lead pump
2021	Changed pump operating strategy to Duty-Standby configuration to prevent 2nd pump from activating and installed backflow preventers for several residential properties

1.3 Regulatory Requirements

Virginia Sewage Collection and Treatment (VSCAT) establishes reliability classification requirements for wastewater pump stations for the purpose of protecting public, environment, and surface waters. There are three levels of reliability classification, with Class I Reliability being the most restrictive. Per Preliminary Engineering

Report (PER) dated August 2013, and Virginia Department of Environmental Quality (VDEQ) application for Certificate to Construct (CTC) dated July 2015, Lakevale Estates Pump Station is classified as Class I Reliability station as defined in the VSCAT Regulations. This classification applies to pump stations whose location, or discharge is sufficiently close to residences, public water supply, or recreation waters, such that permanent or unacceptable damage could occur to the receiving waters or public health and welfare if normal operations were interrupted.

Upgrades to the Lakevale Estates Pump Station hydraulic and gravity system improvements will be in accordance with the latest iteration of Fairfax County Public Facilities Manual (PFM), Fairfax County Wastewater Guidelines for A/E (Volume 2), and the VSCAT Regulations 9VAC25-790 Part III, Articles 1 and 2, with a Reliability Classification of Class I. Provisions to ensure continuous compliance with the Class I standards and operability of pump station, will be evaluated in the PER.

1.4 Existing Conditions

1.4.1 Upstream Collection System

As shown in Figure 1, The Lakevale Estates Pump Station receives flows from three contributory sewers which combine in a manhole immediately adjacent to the pump station's wet well. This manhole is connected to the wet well through a short (approximately 5-foot long) 12-inch influent pipe. Basic statistics for the sub-sewersheds are provided in Table 3.

Table 3 - Sewershed Characteristics

PS Influent Pipe	Area (acres)	Length of Pipe (LF)	Pipe Material (Material, %)	Land Use (Type, %)	Lined Pipes (% of LF)
037-4-142	465	67,174	AC, 59% CONC, 4% DIP, 2% PVC, 35%	Commercial, 4% Residential, 96%	4%
037-4-084	14	2,557	AC, 100%	Residential, 100%	53%
037-4-093	127	12,054	AC, 89% PVC, 11%	Residential, 100%	32%

Two of the contributory sewers cross stream channels immediately upstream of the pump station however field observations did not readily identify concerns with infiltration and inflow during dry weather conditions, which may be a result of the large portion of sewers that are lined in the upstream system, as shown Figure 1.

The residential property at 10208 Kenbrooke Court, immediately adjacent to the pump station access road, on the 8-inch sewer from Kenbrooke Court is at risk of flooding when the water level in the pump station wet well nears the ground surface. Figure 2 summarizes the critical elevations in this location. Other homes along Kenbrooke Court and Meredith Drive are 2 feet higher in elevation, which raises the basement elevations above the ground surface at the wet well.

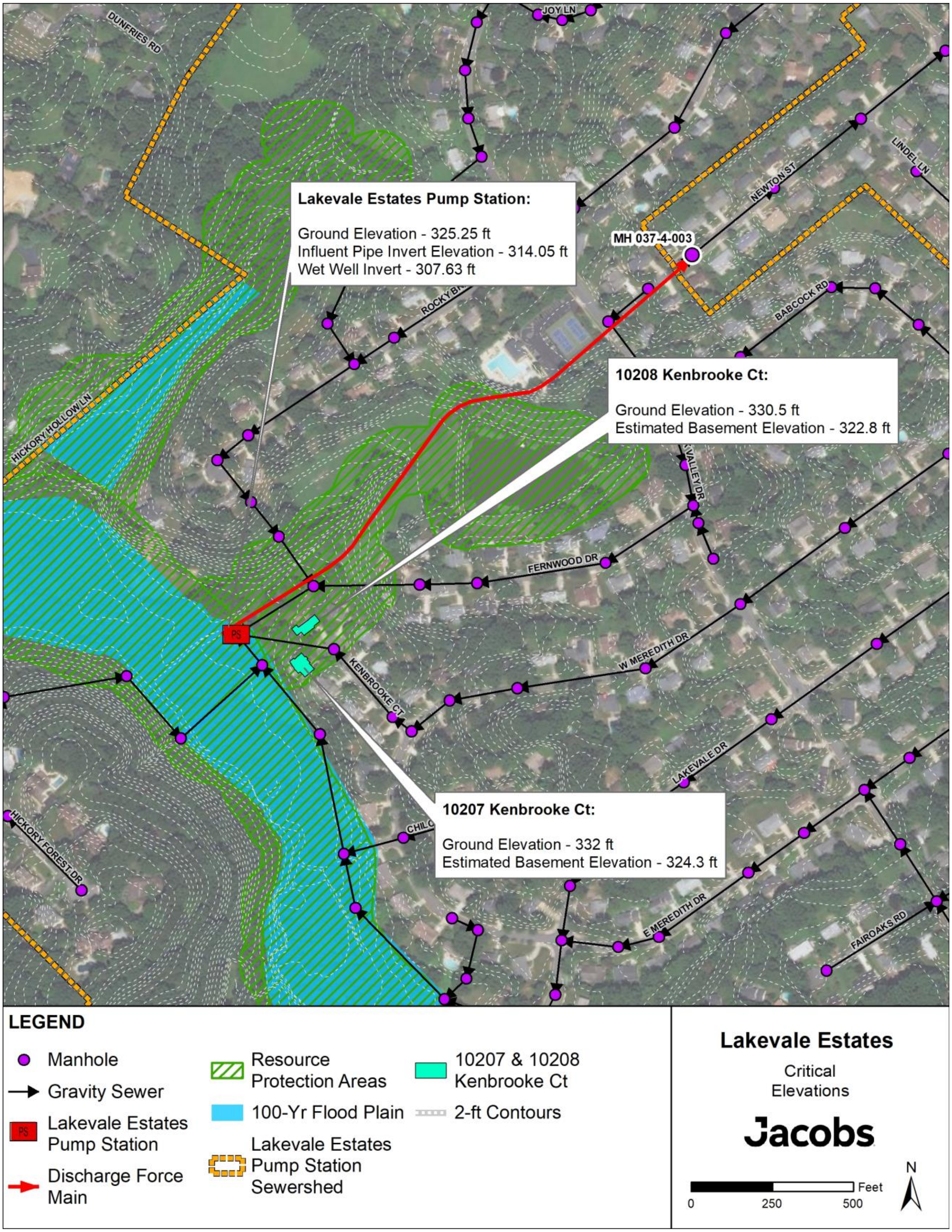


Figure 2 – Lakevale Estates Pump Station

1.4.2 Pump Station

The Lakevale Estates PS collects flow from a mostly residential gravity system, as shown in Figure 1. The station was constructed in 1966 and was rehabilitated in 2019. Existing wet well, dry well and influent manhole are located within the fenced boundaries of the pump station, as shown in Figure 3. The influent manhole receives flow from an 8-inch gravity sewer from north-east, an 8-inch gravity sewer from east and an 8-inch gravity sewer from south-east. The pump station has a design capacity of 619 gpm.

The influent flows are conveyed via a 12-inch gravity pipe into a single 10-ft diameter precast concrete wet well with sloped sides to prevent solids accumulation. A JWC Environmental sewage grinder is located immediately downstream of the 12-inch influent pipe that cuts and screens large objects. The invert of the 12-inch influent pipe is approximately 11.2 feet below grade. The invert of the wet well is approximately 17.6 feet below grade. The wet well also contains two level transducers, level float switches, and a 6-inch vertical bypass pipe.

The wet well is separated from the dry well and contains two 40-hp dry-pit submersible pumps which pull from the wet well through 8-inch suction lines. The Lakevale Estates PS is equipped with an 8-ft diameter pre-engineered underground dry-well package pump system designed and manufactured by Smith and Loveless (S&L).

Flows are conveyed to dry-pit submersible pumps via two separate 8-inch suction pipes located near the bottom of the wet well as shown in Figure 4. The two 8-inch suction pipes were installed during the original construction in 1966 and haven not been replaced as part of the subsequent rehabilitation. The interior piping and valves are ductile iron with flanged connections. Each pump can be isolated from the wet well using 8-inch knife gate valves on the suction piping.

The pump discharge piping contains check and isolation valves before combining into a single 8-inch force main which runs east and discharges to gravity at Manhole 037-4-003 as shown in Figure 2.

A review of 2019 design drawings and pump data show that the rehabilitated station was designed to include two identical dry-pit horizontal 12-inch impeller pumps. Design pump data are summarized in Table 4.

Table 4 - Dry-Pit Submersible Pumps (2019 Design)

Description	Design Value
Number of Pumps	2
Pump Model	4D3
Manufacturer	S&L
Pump Tag ID(s)	P1111, P1112
Impeller Size, inch	12
Capacity, gpm	619 [a]
Total Dynamic Head, ft.	133 [a]
Motor Horsepower, hp	40
Motor Speed, rpm	1800
Pump Type	Dry-Pit Submersible
Drive Type	Constant Speed
Electrical Data	3 phase/60 cycle/460 volts

a. *Pumping capacity and TDH based on Design Specifications completed by others. Referenced in this document for consistency.*

However, one of the originally installed 12-inch impeller pumps was trimmed to a 10 ¾ - inch impeller during construction. The change in impeller size is believed to be due to over conservative calculation of total dynamic head (TDH) throughout different stages of the feasibility study and design and was modified after an August 2019 change order. It should be noted that the County has recently replaced the second impeller 12-inch impeller pump with a 10 ¾-inch impeller. Pump data are summarized in Table 5.

Table 5 - Dry-Pit Submersible Pumps (2020 - 2021 Modifications)

Description	Installed Value	
Number of Pumps	1	1
Pump Model	4D3	
Manufacturer	S&L	
Pump Tag ID	P1111	P1112
Impeller Size(s), inch	10 ¾	10 ¾
Capacity, gpm	622 [a]	415 [b]
Total Dynamic Head, ft.	134 [a]	111 [b]
Motor Horsepower, hp	40	
Motor Speed, rpm	1800	
Pump Type	Dry-Pit Submersible	
Drive Type	Constant Speed	
Electrical Data	3 phase/60 cycle/460 Volts	

a. *Assumed pumping capacity and TDH based on Design Specifications completed by others. Referenced in this document for consistency.*

b. *Pump submittal (D-11320-004-A Lakevale Pump Replacement) by Archer Western Construction, LLC, dated November 21, 2019, and approved by Dewberry on January 3, 2020.*

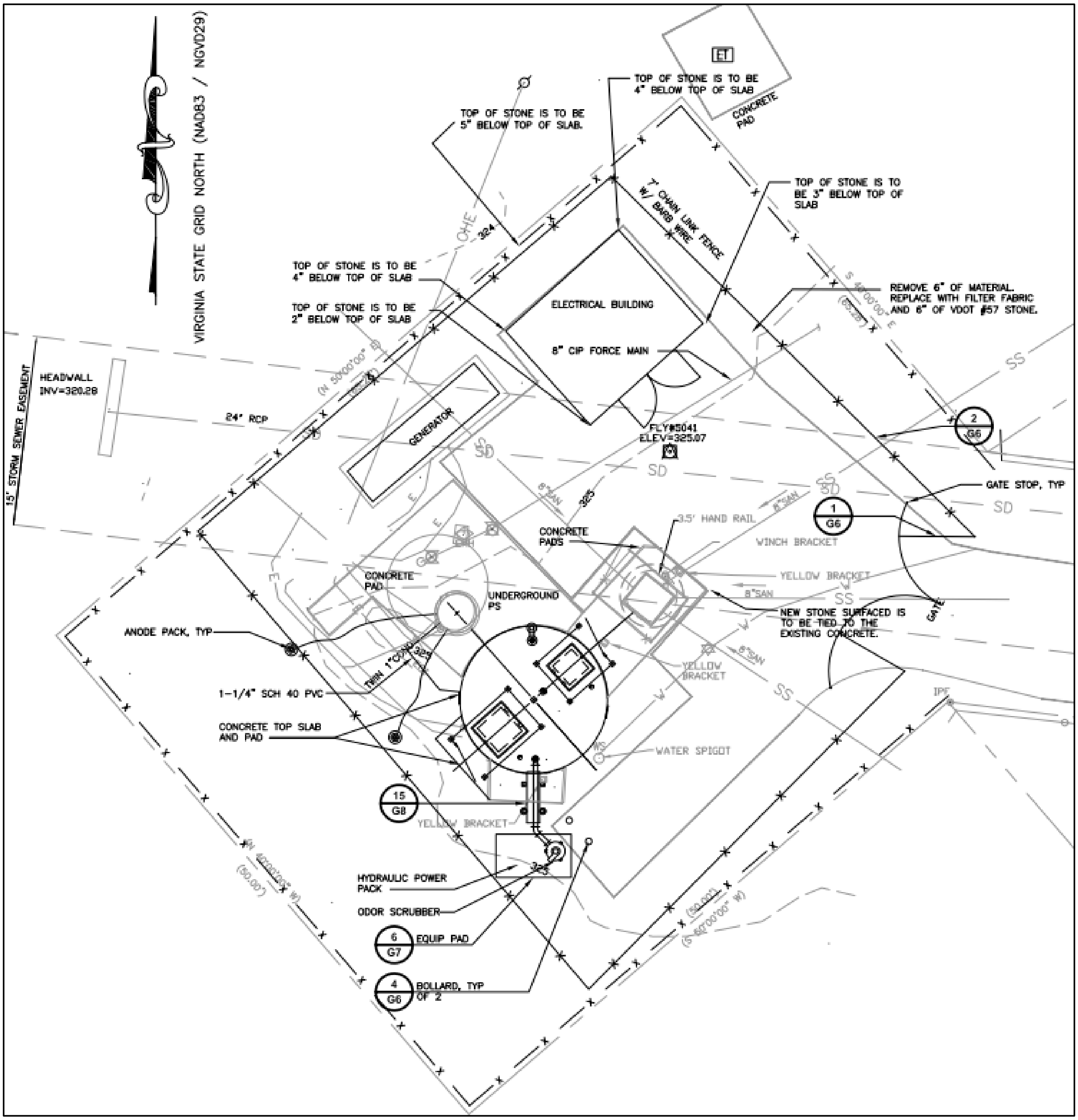


Figure 3 – Lakevale Estates PS Site Plan

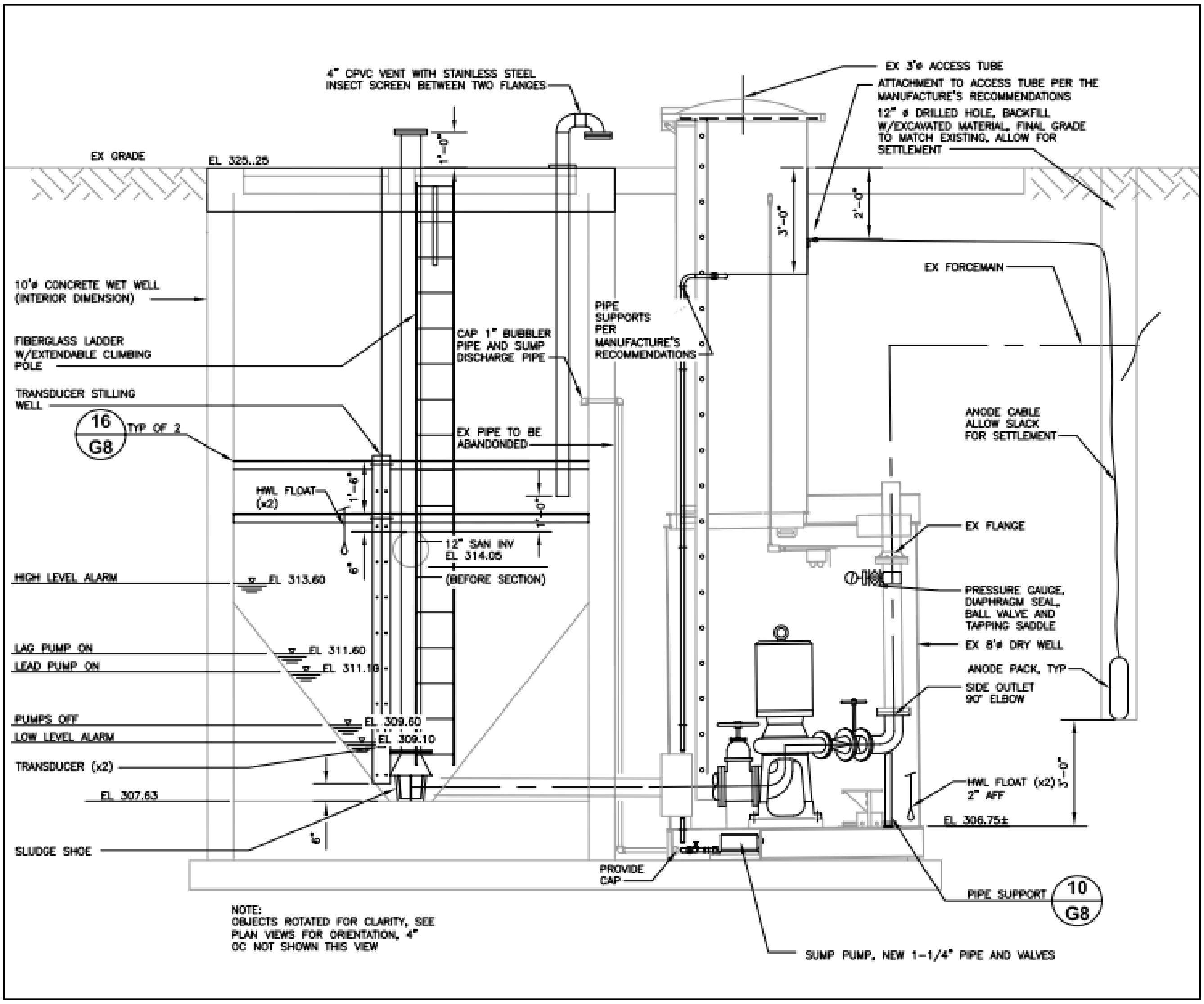


Figure 4 - Lakevale PS Wet Well and Dry Well - Section View

1.4.3 Wet Well Capacity

Record drawings were utilized to determine the approximate effective volume of the existing wet well. The current design (2019 upgrades) allows the pumps to automatically cycle through the pump lead-lag alternation sequence and it is configured to allow the second pump to operate during high flow conditions. The second pump starts at a high level (lag set point) to prevent pump station overflows. Table 6 and Figure 5 provide a summary of the existing wet well storage volume.

Table 6 - Pump Control Set Points

Pump Set Points	Depth (ft)	Elevation (ft)	Cumulative Volume (gallons)	Effective Storage Volume (gallons)
Pumps OFF	2	309.6	174.2	174.2 [a]
Lead Pump ON	5	312.6	1,23.6	1,060.4 [b]
Lag Pump ON	6	313.6	1,931.5	697 [c]
High Level Alarm	6.4	314.0	2,251.6	320.1 [d]

a. Dead storage to meet the Net Positive Suction Head required (NPSHr) condition
b. Lead Pump active storage volume
c. Lag Pump active storage volume
d. Reserve storage volume

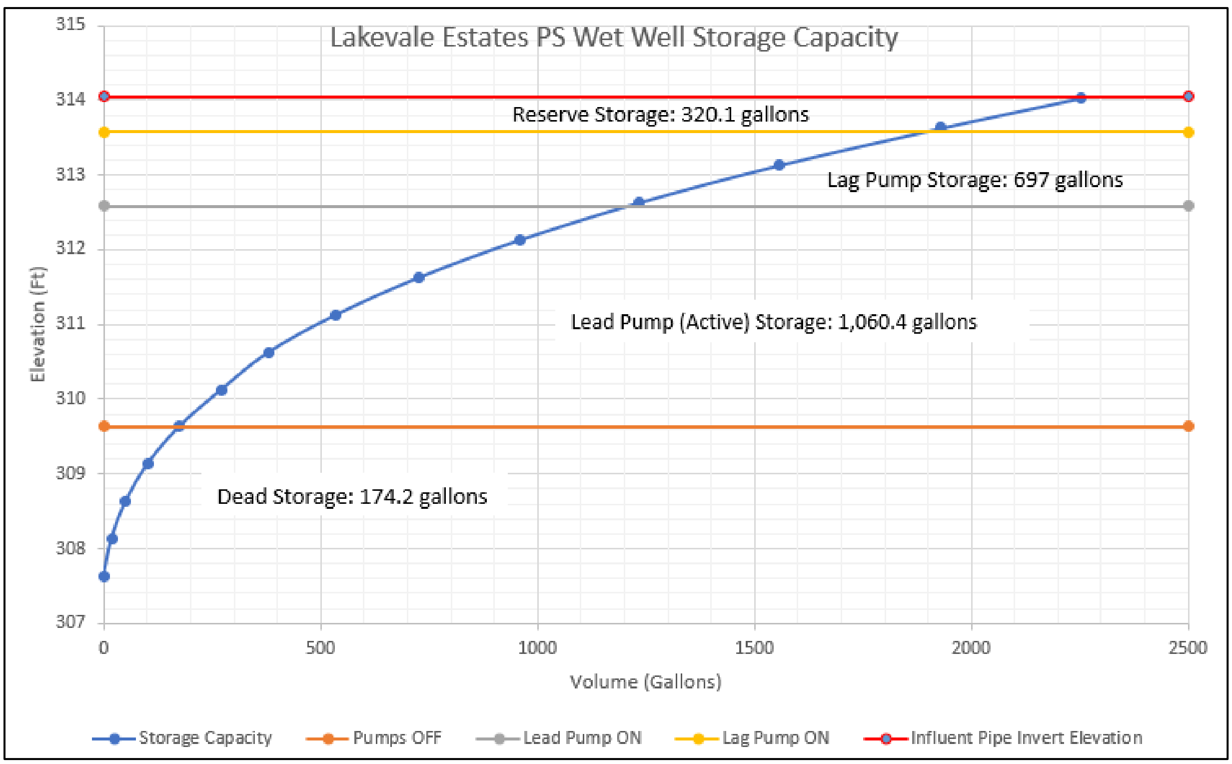


Figure 5 - Lakevale Estates PS Wet Well Storage Capacity

1.4.4 Force Main

According to the original construction record drawings, the existing force main was constructed of 8-inch class 150 cast iron (CI) pipe in 1966. The 1,944 linear feet force main conveys flows from Lakevale Estates Pump Station to manhole 037-4-003, located on Newton Street. The force main was lined in 2013 and data provided by Fairfax County was evaluated to determine the actual inner diameter of the existing CI force main. The lining was reported to have a 5.5 mm (0.22-inch) thickness, resulting in a 7.55-inch inner diameter. The County has not reported any issues with breaks or leakage associated with the force main.

At pumping capacities stated above, a minimum self-scouring velocity of two feet per second is achieved. With the current configuration, maximum velocities at higher pumping rates do not exceed eight feet per second, as defined in the VASCAT 9VAC25-790-440.1. The existing 8-inch force main currently operates at pressures in excess of 55 pounds per square inch (psi).

1.4.5 Downstream Collection System

The force main discharges to a 10-inch gravity sewer on Newton Street. The entire 10-inch line was constructed of asbestos cement (AC) in the 1960's and then lined between 2005 and 2012 from the force main discharge to Manhole 038-3-006, 3 pipe segments upstream of the project boundary, as shown in Figure 2. The portion along Newton Street was lined in 2012, in concert with the lining of the force main and the portion along Vale Road was lined in 2017. County records indicate the liner thickness is 6 mm for the pipe along Newton Street and Vale Road, although flow meter installation at MH 038-3-014 measured an interior pipe diameter of 9-inch. The sewer was surveyed by Dewberry during December 2020 and January 2021.

Laterals along this line are tapped directly into the pipe (not at the manhole) and connections are generally located at the 3-o'clock and 9-o'clock positions. Basement elevations were estimated using the assumptions shown in Figure 6. As shown in the figure, these estimates rely on 2-foot contour topography from GIS data and several assumptions about home construction. Based on this estimation of basement elevations, three basements are within a foot of the crown of the sewer (top of the pipe) and at risk of flooding if the water surface surcharges above the pipe at all. Table 7 summarizes the estimated point of flooding (crown of lateral under basement slab) for several of the downstream houses, and the adjacent crown of the sewer for reference.

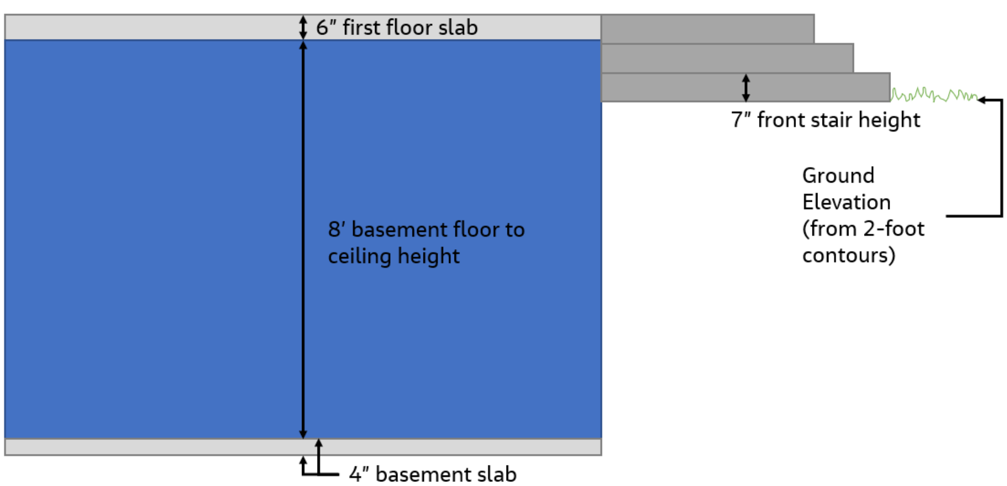


Figure 6 - Basement Elevation Estimation Schematic

Assumptions used for basement elevations: ground elevation + number of front stairs x 7" – 6" first floor slab – 8' basement floor to ceiling height – 4" basement slab.

Table 7 - Downstream home basement elevation estimates relative to crown of sewer

House Number	Estimated Basement Elevation (ft)	Conduit Crown at Lateral Elevation (ft)
2419 Newton St	**403.9**	**403.8**
2418 Newton St	407.1	403.8
2417 Newton St	405.2	403.4
2416 Newton St	407.8	403.4
2415 Newton St	407.5	403.1
2414 Newton St	407.8	402.9
2413 Newton St	**402.3**	**402.4**
2412 Newton St	406.1	402.5
2411 Newton St	404.1	401.6
2410 Newton St	406.1	401.5
2409 Newton St	403.5	400.8
2408 Newton St	403.8	400.9
2406 Newton St	408.1	400.1
9933 Newton St	**403.8**	**403.2**

Bold text indicates basement is within 1 foot of crown of the sewer

CCTV reports from 2018 to 2021 were reviewed. The lining was intact and generally in good condition along Newton Street and Vale Service Road, with no notes of significant leaks or infiltration, however there were several notes of the seam pealing, a bulge in the liner between 037-4-001 and 038-3-017 just upstream of Vale Service Road and a hole in the lining on Vale Service Road between 038-3-016 and 038-3-015. In addition, there were several notes of grease build-up impacting 5%-25% of the pipe cross-section and some notes of sags. The pipes were cleaned on August 31, 2021, and the County has traced the source of the grease and resolved this issue.

A high water mark was noted during the field investigation approximately 12 inches above the crown of the gravity sewer at manhole 037-4-003 (force main discharge point) indicating surcharging at this location, corroborated further by reported basement flooding incidents along this sewer. Flow conditions were noted to be more turbulent in the first segment downstream of the force main discharge manhole with notably laminar flow one segment downstream.

2. Available Data and Models

2.1.1 Rainfall

Rainfall data were provided by the County for the Fairfax Center and Hunter Mill Rain Gauges. The Rain Gauge locations are shown in Figure 7. The 5-minute rainfall data cover the period of record from March 1, 2021, through September 9, 2021, correlating to the flow meter data. Jacobs primarily focused on the data from the Hunter Mill Road gauge, given its closer proximity to the sewershed.

In addition to the data provided by the County, Jacobs had historical 15-minute data from Fairfax County's Pickett Road rain gauge that covered 2018 through 2021 (location shown in Figure 8), which was used to identify significant events within the longer period of record for the pump station SCADA data.

2.1.2 Sewer Flow Meter

In March 2021, ADS installed a flow meter for the County in manhole 038-3-013, approximately 1,900 feet downstream of the force main discharge manhole. The meter provides 5-minute flow, velocity, and depth measurements. The location of the flow meter is shown in Figure 7.

2.1.3 Pump Station SCADA

Wet well level at the Lakevale Estates PS and pump run times from April 2019 through September 2021; 1-minute time step data were provided. Details of the analysis of this data to estimate influent flow to the pump station are provided in Appendix B.

2.1.4 Collection Systems Model

The County provided a copy of a portion of the all-pipes Infoworks ICM collection system model currently under development by Black & Veatch (B&V). The model had been calibrated for dry weather flow but had not yet been calibrated to wet weather flows. Jacobs reviewed and updated the model file as necessary to suit the needs of this project. Model parameters, including wet well storage dimensions, pump/on off levels, and sewer inverts, were updated to reflect current conditions per provided construction drawings and system survey data. Review of the model also included a validation of B&V's dry weather flow (DWF) calibration against the project flow monitoring data and adjustment of the model parameters to effectively match the model-projected peak depth with the observed peak depth during the August 13th power outage event that recently caused flooding. Additional details on model modifications are provided in Appendix D.

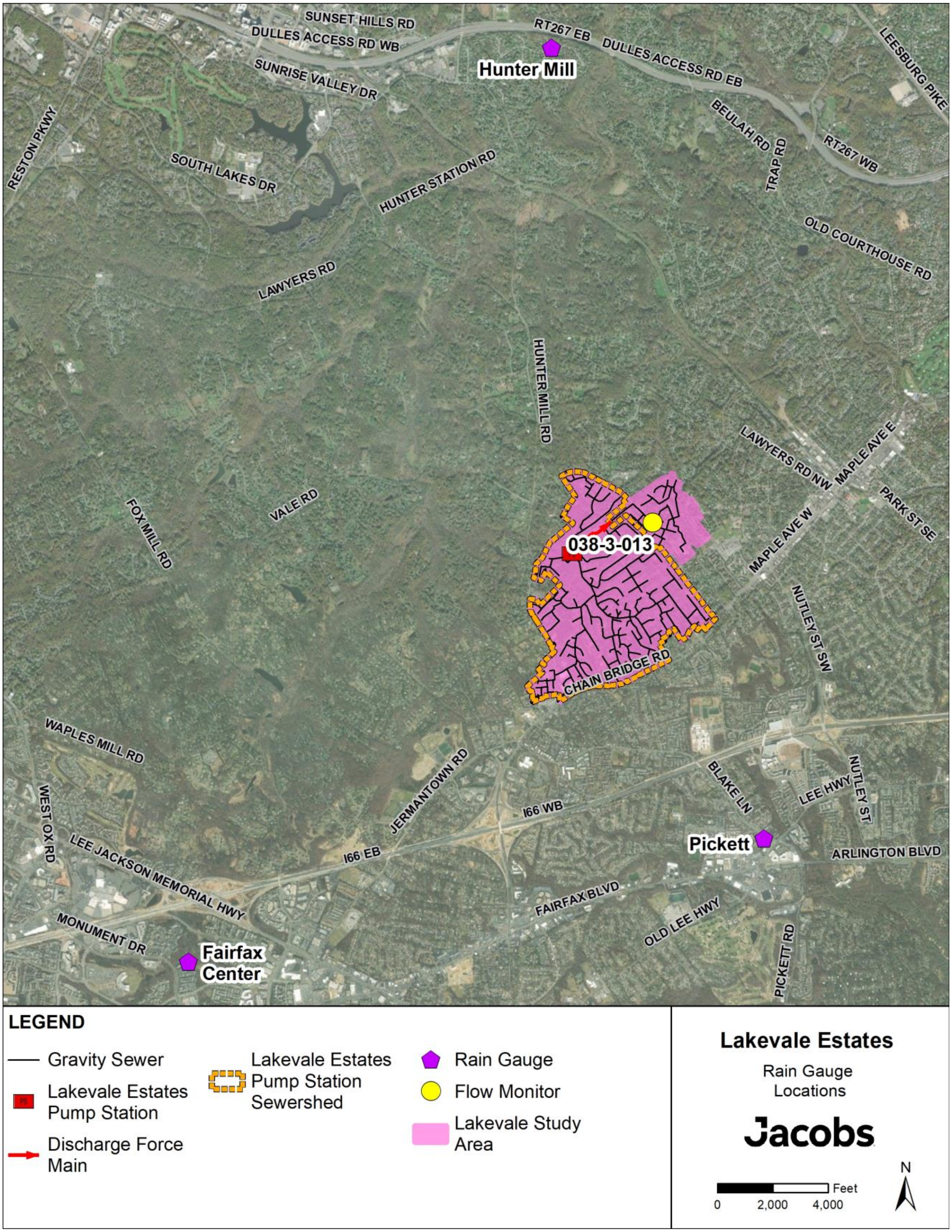


Figure 7 - Rain Gauge Locations

2.1.5 Growth Projection

Jacobs analyzed the MWCOG round 9.1a, which is the latest version of the cooperative forecast published on October 17, 2018, to establish future wastewater flow projections for the Lakevale Estates sewershed based on an increase in projected population within the pump station service area.

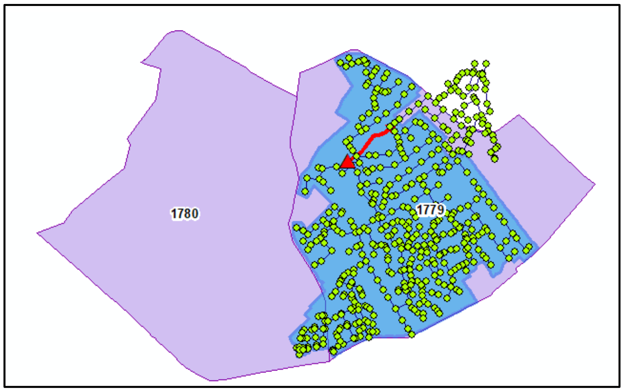


Figure 8 - Lakevale Estates PS Service Area

Figure 8 shows the Lakevale Estates Pump Station service area (represented by the light blue shaded region) with respect to two Transportation Analysis Zones (TAZs). Population, employment, and household data are grouped in each TAZ. TAZ's boundaries typically follow census data geography and boundaries. As shown in Figure 8, the majority of the Lakevale Estates sewershed is located in TAZ No. 1779, with a small portion located in TAZ No. 1780. Jacobs utilized area weighted calculations using the MWCOG GIS shapefiles to identify the population projection within the Lakevale Estates Pump Station sewershed. Based on the MWCOG projections, the current population within the Lakevale Estates sewershed is estimated to increase 5% by year 2045. Similarly, Fairfax County's population is expected to grow steadily through the forecast period, adding an average of approximately 48,000 person per year. As shown in Table 9, Fairfax County's population is projected to increase by approximately 22% during the same forecast period. It should be noted that the MWCOG round 9.1a was published prior to the COVID-19 pandemic and MWCOG is currently analyzing the impacts of the COVID-19 pandemic on the region's long-term growth projections.

Table 8 includes a summary of population projection for this sewershed.

Table 8 – MWCOG Round 9.1a Growth Projection (TAZ 1779 and 1780)

Year	2020	2025	2030	2035	2040	2045	Percent Growth 2020 to 2045
Population	4,038	4,043	4,084	4,141	4,186	4,230	5%

Table 9 - Summary of Intermediate Population Forecast-Round 9.1 Cooperative Forecast (thousands)

JURISDICTION	2015	2020	2025	2030	2035	2040	2045
District of Columbia	672.2	729.5	787.1	842.2	893.9	940.7	987.2
Arlington County	220.9	238.3	249.5	261.8	274.6	287.6	301.2
City of Alexandria	147.6	159.2	167.5	172.8	180.5	190.8	208.5
Central Jurisdictions	**1,040.8**	**1,127.0**	**1,204.1**	**1,276.7**	**1,348.9**	**1,419.1**	**1,496.8**
Montgomery County	1,015.3	1,052.0	1,087.3	1,128.8	1,167.7	1,197.1	1,223.3
City of Rockville	66.3	72.2	78.2	83.3	86.7	91.8	96.1
City of Gaithersburg	67.1	70.7	74.6	78.7	82.4	86.1	89.3
Prince George's County	904.4	923.1	938.0	953.0	967.8	982.8	995.9
Fairfax County	1,125.4	1,161.8	1,210.8	1,271.2	1,325.3	1,373.7	1,416.8
City of Fairfax	24.1	25.6	29.2	31.6	32.7	33.9	35.2
City of Falls Church	13.1	14.2	15.5	16.4	17.0	17.3	17.6

3. Hydraulic Analysis

Jacobs analyzed historical flow data provided by the County to estimate the future average and peak daily flow for the station. The planning period for this study is 24 years (2045), which is typical of a municipal facility. A copy of the detailed analysis is included in Appendix A. The following section of this TM highlights the findings.

3.1.1 Methodology

The peak influent flows can be computed using several approaches and the following three approaches were applied for this project:

- Average dry weather method
- Historical flow analysis
- Development flow factors

Three sources of data were used to determine the most appropriate base flow for use in each of the three projected peak flow calculation approaches. Sources of data included downstream flow meter data, SCADA data at the wet well, and the provided Infoworks ICM model calibrated for dry weather flow (DWF). Results from these analyses are summarized in Table 10 and the details on each approach are in Appendix A.

Table 10 - Estimated Peak Inflows to Lakevale Estates Pump Station

Method	**Description**	Peak Flow Rate, gpm
1	Average Dry Weather Method	593 [a]
2	Historical Peak Flow Method	551 [a]
3	Development Flow Factors	799

a. Values include 5 % population growth

The results of the three methods were compared and Method 1 was selected for the design flow. The per capita and unit-based wastewater estimates used in Method 3 are known for being very conservative, and measured flows suggest this approach is substantially overestimating flows delivered to Lakevale Estates PS, therefore this method was not selected. Because the period of record was relatively short, and more significant wet weather events could produce higher peaks, Method 1 was selected as the more conservative estimate. It is still within the order of magnitude of actual measured peaks and provides an additional level of conservatism. Refer to Appendix A for a detailed analysis of the aforementioned approaches.

3.2 Downstream Gravity System Hydraulic Capacity

The following section summarizes the results of downstream gravity system hydraulic modeling. Refer to Appendix B, for detailed information about the gravity system hydraulic analysis completed for Lakevale Estates Pump Station.

3.2.1 Hydraulic Evaluation Summary

The collection system model provided by the County was used to evaluate the downstream collection system. Data from the flow meter installed in MH 038-3-013 were used to calibrate the parameter to replicate flow depth at a range of flow rates. Flow depths were calibrated to depth at the flow meter as well as reported flooding at five homes downstream of the force main discharge (MH 037-4-003). Details on the model calibration are summarized in Appendix B.

Based on the calibrated model, the gravity pipe immediately downstream of the force main discharge has a current capacity to convey 580 gpm when flowing full. Based on survey data this pipe segment has the flattest slope (0.52%) and therefore has a slightly lower capacity than pipes further downstream.

Comparing this pipe capacity to the range of pump station flow rates, the existing gravity sewer has the capacity to accommodate the smaller of the two existing pumps but does not have the capacity to accommodate the larger of the two pumps, supporting the County's decision to take the larger pump out of service until a smaller impeller can be obtained or the downstream gravity system can be improved.

The County has replaced the 12-inch impeller with a 10 ¾ -inch impeller. Our analysis computes the capacity of two 10 ¾ -inch pumps to be 600 gpm at the lowest TDH (117 ft). This has the potential to put at least one house downstream of the force main at risk of flooding under a lead-lag pumping configuration which allows concurrent operation of both pumps. It is advisable to set up limitations on the use of the 2nd pump.

Table 11 provides existing pipe capacity and estimated existing flow with 1 and 2 pumps running for each pipe segment. Pipes through manhole 038-3-013 are currently near or above full-flow capacity if two pumps operate.

Table 11 - Existing Pipe Capacity

Upstream Manhole	Diameter*	Lining	Existing Full-Flow Capacity [gpm]	Flow with 1 existing pump running [gpm]	Flow with 2 existing pumps running [gpm]
037-4-003	9"	CIPP	583	482	600
037-4-002	9"	CIPP	653	488	607
037-4-001	9"	CIPP	625	492	610
038-3-017	9"	CIPP	639	500	618
038-3-016	9"	CIPP	639	500	618
038-3-015	9"	CIPP	604	514	632
038-3-014	9"	CIPP	604	522	641
038-3-013	9"	CIPP	639	520	639
038-3-012	9"	CIPP	1153	541	659
038-3-011	9"	CIPP	806	544	662
038-3-010	9"	CIPP	2299	550	668
038-3-009	9"	CIPP	1424	595	714

Upstream Manhole	Diameter*	Lining	Existing Full-Flow Capacity [gpm]	Flow with 1 existing pump running [gpm]	Flow with 2 existing pumps running [gpm]
038-3-008	9"	CIPP	1104	598	716
038-3-007	9"	CIPP	1097	598	716
038-3-006	10"	N/A	1333	596	715
038-3-005	10"	N/A	1326	597	716
038-3-004	10"	N/A	1465	603	722

*Lined pipes are 10-inch Asbestos Cement pipes, with an internal diameter of 9-inches after lining.

Flow is computed as pump capacity plus incremental base flow times peaking factor of 5

Red text indicates flow exceeds pipe capacity.

4. Alternatives Description and Analysis

4.1 Introduction

This section describes the development of the project alternatives, presents the project alternatives, summarizes and compares the feasibility of the alternatives, in order to make recommendations on the superior alternative. Alternatives considered, but not found to be technically feasible or reasonable, along with the reasons they were eliminated from further analysis, are included Table 18.

4.2 Alternatives Considered

Three alternatives (DS-1 through DS-3) are provided for upgrading the downstream system. The intent for three downstream alternatives is to improve hydraulic capacity and provide relief for the existing sanitary sewer. The following sections provide a description of the alternatives considered in this TM. Figure 9 highlights the scope of the downstream alternatives.

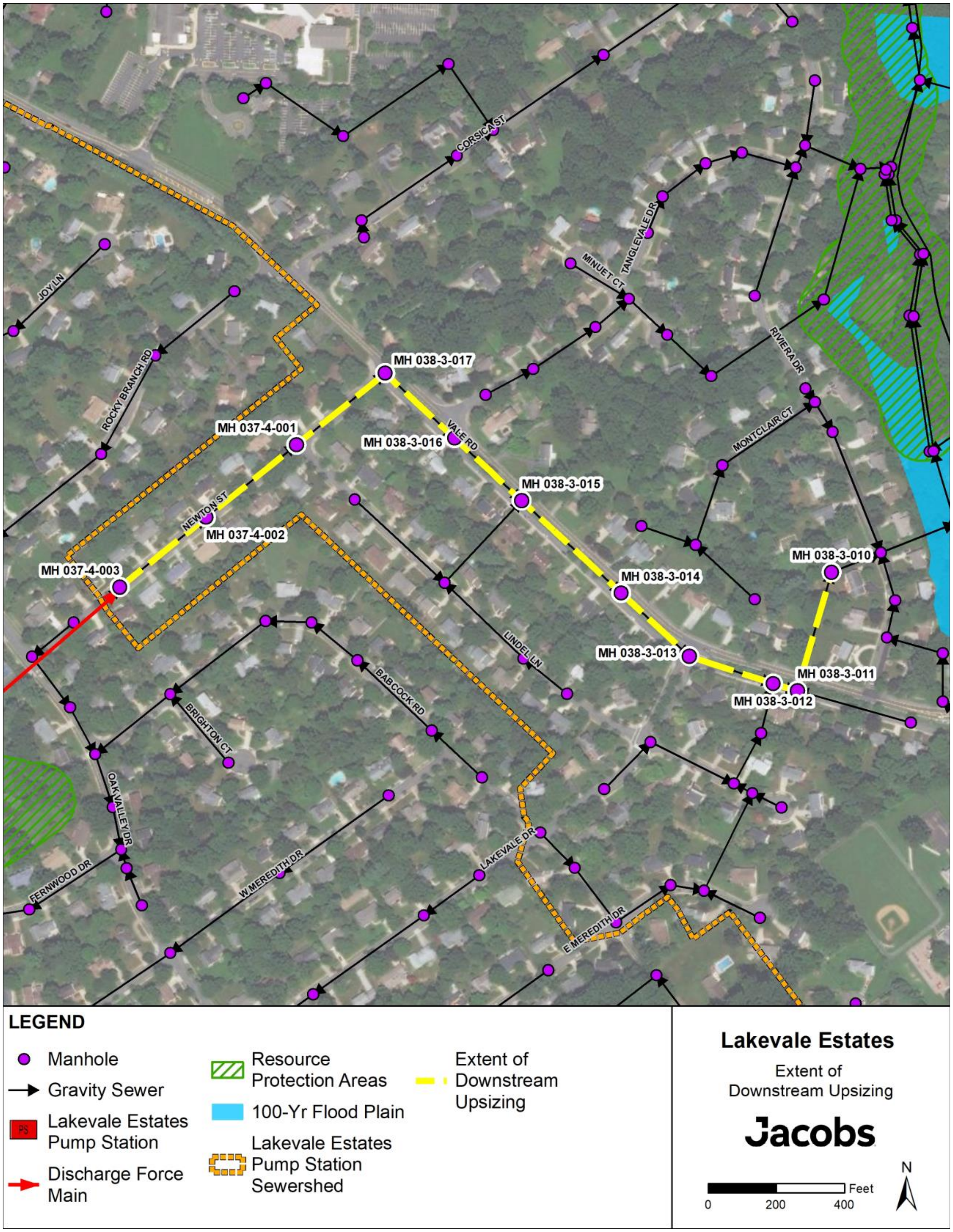


Figure 9 – Extent of Downstream Alternatives

4.3 Alternative DS-1 – Pipeline Replacement

4.3.1 Description

The first alternative for upgrading the downstream gravity sewer is to replace the existing 10-inch lined asbestos concrete pipe with a new 12-inch pipe. This alternative would require re-connecting all sewer laterals and replacing all manholes along the alignment. Approximately 2,900 linear feet of existing pipe will be replaced (through manhole 038-3-010). Replacing the pipe in place at the existing slope of 0.52% provides an upgraded capacity of 1,300 gpm, which meets PFM requirements and accommodates the maximum peak flow when both pumps are active at the maximum TDH for the pump station.

This alternative would be to construct a new gravity sewer line as a parallel sewer, reconnect all laterals to the new sewer and abandon the existing asbestos line in-place by demolishing top 4-6 feet of the manholes and backfill manhole and sewer with flowable fill.

Open cut is a traditional construction method that is typically less costly for shallow excavations. In general, excavation for the proposed sewer line will be deeper than 5 feet. Therefore, trench box or other excavation support method will be required, considering sloped excavation will not be feasible because of limited space on the existing roadways. For the proposed sewer lines, excavation may be up to approximately 20 feet deep, which may require stacked trench boxes and a relatively wide excavation.

Many existing utilities cross the proposed sewer line and will require support, or temporary or permanent re-routing during excavation, to protect these existing utilities from damage. Coordination with the utility providers and their contractors can be very time-consuming, which will further delay construction. Also based on underground utility data provided by the County, two gas lines located in Vale Service Road run in parallel with the existing gravity sewer line, which makes open cut excavation a very unattractive option in this area.

One of the major disadvantages of using open cut installation is the significant impact on neighborhood traffic. It is likely that the contractor's work hours will have to be restricted and traffic control measures will need to be implemented and dismantled on a daily basis to allow homeowners vehicular access to their properties. Open trenches must either be filled in or covered with steel-plate every day. Additionally, higher risk of traffic accidents is generally associated with the open cut installation method.

4.3.2 DS-1 Alternative Analysis Summary

Table 12 – DS-1 Alternative Advantages and Disadvantages

Advantages	Disadvantages
Eliminate aging infrastructure. New line takes all flows from both pump station and lateral connections (not a relief sewer)	Major utility conflicts and utility relocation delays
Pipe capacity will be increased to allow for both pumps to operate without surcharging downstream system	Significant public impact due to open cut installation and traffic control
	Construction sequencing: requires all lateral connections to be reestablished along the entire length the new sewer line
	Requires extensive bypass pumping downstream of a force main during lateral reconnection.
	Requires installation of new manholes
	Require partial demolition of existing manholes
	Cannot take advantage of remaining useful life the existing pipe

4.4 Alternative DS-2 – Parallel Downstream Gravity Sewer

4.4.1 Description

The second alternative for upgrading the downstream gravity sewer is to install a parallel gravity pipe adjacent to the existing 10-inch lined asbestos concrete pipe. Similar to DS-1, under this scenario, flows from Lakevale Pump Station force main will be discharged to the new 12-inch pipe and the new pipe would be installed to match the inverts of the existing pipes. This alternative would require re-connecting approximately half of the sewer laterals and installing new manholes along the alignment. At a minimum, approximately 2,900 feet of new pipe will be added (through manhole 038-3-010).

Adding a parallel 12-inch pipe at the existing slope of 0.52% provides an additional 1300 gpm of capacity to the existing line, which meets PFM requirements and accommodates the maximum peak flow when both pumps are active at the maximum existing design TDH for the pump station. Slope of sewer line can be accurately controlled using available trenchless technologies, but unforeseen buried objects, underground utility conflicts, or major variation in ground condition can cause significant difficulties. It should be noted that geotechnical investigation is required to assess the general suitability of the site and appropriateness of the proposed construction method.

In conventional open cut excavation, service lateral connections can be made as the work progresses. The open trenches necessary for the lateral connections do not cause any significantly greater disruption than that caused by construction of the main sewer line. However, making lateral connections using trenchless methods is generally not feasible and the open trenches required to connect a large number of laterals may almost completely nullify the benefits that may be gained from using a trenchless method to install the new gravity sewer line. Each lateral would need to be located and connected to the new line by using open cut excavation method.

If a Pilot Tube Microtunneling is used to install the new gravity sewer line, the possibility of connecting all laterals directly to the new manholes that will be placed in the launching/receiving shafts, rather than the main gravity sewer, could be considered, which will require open cut excavation. Although this would result in longer runs for the laterals (assuming proper slope can be maintained), it would have the added advantage of preserving the structural integrity of the new gravity sewer.

4.4.2 DS-2 Alternative Analysis Summary

Table 13 - DS-2 Alternative Advantages and Disadvantages

Advantages	Disadvantages
Pump station discharge flows will be diverted to the new parallel pipeline	Requires restoration of about 50% of existing lateral connections which will require open cut excavation
Continue to utilize the existing gravity system conveyance capacity	Requires installation of new manholes
Trenchless construction method	Requires longer runs for laterals
Existing pipeline can stay in service during construction	
Pumps can be operated in lead-lag configuration	

4.5 Alternative DS-3 – Parallel Force Main

4.5.1 Description

The third alternative for upgrading the downstream gravity sewer is to extend the force main to a point where the gravity system is no longer constrained. Trenchless construction techniques should be utilized for installation of the extended force main, where grade is not as critical as the gravity system and provides more construction flexibility with respect to underground utilities and geotechnical conditions. Jacobs conducted a preliminary evaluation of the existing pumps and coordinated with the S&L's representative to confirm the existing pump model's operation based on the extended force main configuration, static and dynamic heads. Preliminary evaluations show that the existing pump model with the largest impeller can discharge 593 gpm to a distance between manhole 038-3-011 and 038-3-010, approximately 2,600 linear feet from its current discharge location. Further pump station hydraulic analysis will be completed as part of the subsequent phases to select the impeller size.

One of the biggest advantages of this alternative is eliminating the need to reestablish the service lateral connections, which typically has a significant impact on the residents and requires public coordination. Although the time required to complete a sewer installation project will depend on a number of factors, it is generally recognized that most trenchless methods will be quicker than open cut excavation. However, it should also be noted that significant delays could occur in the event of a failure (e.g., stuck machines, heave, utility strike, etc.) with a trenchless method. A typical drive rate for a microtunneling is about 3 to 6 feet per hour, but could be up to 12 feet per hour in favorable geotechnical conditions. It should be noted that geotechnical investigation is required to assess the general suitability of the site and appropriateness of the proposed construction method.

Other benefits of trenchless force main installation include less disruption to surroundings, social factors such as reduction in noise, dust, traffic, and surface distribution, and maintaining existing gravity sewer system.

4.5.2 DS-3 Alternative Analysis Summary

Table 14 - DS-3 Alternative Advantages and Disadvantages

Advantages	Disadvantages
Completely divert flows from the pump station to the gravity system in Newton St. to the proposed force main	Air release valve(s) will be required at intermediate high point(s)
Existing line in service during construction (short bypass duration to divert flows to the new force main)	
Trenchless construction technique. Safer and more neighborhood friendly installation method due to the smaller footprint of the work shafts compared to open cut installation method	
No lateral re-connections and shorten construction time	
Avoid asbestos concrete pipe disposal	
Existing pumps equipped with largest impeller can be utilized for this application	
No manhole on force main	
Pump station can be operated in lead-lag configuration	

4.6 Other Considerations for Downstream Alternatives

4.6.1 Traffic

Majority of the new sewer will be installed under the pavement parallel to the roadway. If open cut is used for sewer pipe installation, one lane closure is anticipated. Construction of the pipe segment crossing Vale Rd will require a robust traffic control plan. Temporary traffic controls shall be implemented in accordance with Manual on Uniform Traffic Control Devices (MUTCD) in all phases of construction. The discussion and preparation of maintenance of traffic (MOT) details are beyond the scope of this report. Temporary traffic controls will also be required at the locations of sewer tie-ins since the connections are in the middle of the road. In all cases, proper warning signs and channelization devices shall be designed and implemented in accordance with MUTCD during all phases of construction. It should be noted that in a residential neighborhood, pedestrian traffic and safety are crucial. Although the project site is located in a single-family subdivision, unobstructed access to fire hydrants and emergency egress from premises in the event of fire must also be considered.

4.6.2 Material Storage

Project site is located in developed residential area. As a result, it may be difficult to find appropriate spaces for material storage. The best recommendation is to coordinate schedules of material handling and construction to minimize required storage space on site or find a nearby storage area that can be provided by the County. In general, pipe materials can be stored along a trench with a minimum of 2 feet clearance to the edge of trench excavation. Excavated soils can be stockpiled at a higher side along the trench to be used as a backfill material if meeting gradation requirements specified in a typical trench backfill specification. Excavated soils not meeting gradation requirements should be disposed of off site.

Furthermore, the location of material storage shall be coordinated with the temporary traffic control to ensure adequate access to private properties at all times.

4.6.3 Easements

Proposed permanent structures, including pipes and manhole structures in Newton Street and Vale Road Service Road, are within limits of right-of-way (ROW). Thus, no acquisition of a permanent easement is anticipated at this stage in the project. However, the project may require temporary construction easements, depending on the installation methodology.

5. Scoring Methodology

5.1.1 Introduction

The planning phase of engineering projects is often a complex balance of the project's technical needs with those of other factors influencing the work. Pipeline and conveyance projects can be among the most complex project during the planning stage since their linear influence zones often result in some type of interaction with a significant number of external factors. Because of this complexity, effective early planning is often a key step in successful implementation of pipeline and conveyance projects.

Non-cost analysis involves the evaluation of each alternative to a specific set of evaluation categories. Selection of these categories is also an opportunity for project stakeholders to provide input into the alternative analysis. Homeowners, operations staff, regulators, and other stakeholders often have unique issues that require analysis in this portion of the overall project. Non-cost criteria included Community Impacts, Operations and Maintenance, Constructability and Environmental Impacts.

Cost estimate analysis of alternatives is always a key consideration for evaluating project alternatives. Given the importance of the cost analysis, it is a key goal to be able to provide meaningful cost data without the need for a detailed cost assessment of the specific features of each alternative. The cost analysis method described in this section is considered equivalent to order-of-magnitude (class 5) cost estimate accuracy. This level of cost estimating is typical for pipeline and conveyance studies and should provide a suitable basis for proper alternative selection.

Using the selected categories and key factors applicable within each category, the project team conducted an analysis of each alternative relative to each category. The analyses are presented in Section 5. Once the analyses were completed, each alternative was given a rating for these categories. These ratings were quantitative numerical score, ranging from 4 points for more favorable characteristics to 1 point for relatively negative/poor characteristics.

Weighing factors summing to 100% were assigned to each of the categories. The selection of the weighting factors involved participation from key stakeholders from Fairfax County and Jacobs project team. Figure 10 below shows the weighting factors for the aforementioned categories.

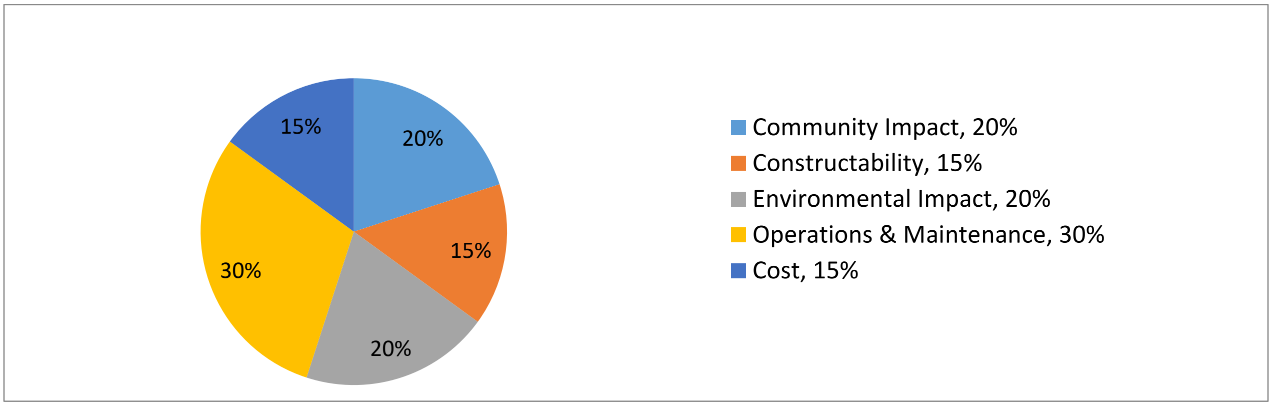


Figure 10 - Weighting Factors

Note that one advantage of the quantitative numerical method of combining alternative ratings is the ability to conduct a sensitivity analysis on the results. The category weighting factors can be adjusted up and down in several combinations (always summing to 100%) to assess the sensitivity of the results versus the relative importance of individual categories. The magnitude of the changes in combined ratings due to variations in the weighting factors gives the project team a better appreciation of the validity of the computed results. The following sections include a brief description for each category.

5.1.2 Community Impacts

Community acceptance pertains to the potential impacts the proposed alternatives may have on the public and the surrounding community. Public relations issues for consideration would include construction concerns, such as higher visibility of construction operations, dust and noise (due to 24/7 bypass pumping and general construction activities), odor, and proximity to the residential area/public; potential interruptions of utility services to a neighborhood; and other factors which could cause disruptions to residential neighborhoods.

5.1.3 Constructability

Constructability is a project characteristic that reflects the ease with which a project can be built. Cost savings is one recognized benefit of constructability. At its finest, constructability is the integration of design and construction knowledge during the early stages of a project development process to ensure the project is buildable, cost effective, biddable, and maintainable. Constructability requirements include potential conditions for each alternative that may impact the construction of the proposed improvements. Provided below is a list of applicable criteria:

- Construction staging area
- Construction method: open-cut vs. trenchless
- Demolition and asbestos cement pipe disposal (health and safety)
- Gravity system hydraulics, pipe material, and alignment (length of sewer)
- Temporary and permanent easements (new sewer alignment)
- Construction sequencing
- Bypass pumping duration
- Lateral connections
- Site constraints
- Existing underground utilities
- Level of impact on the existing pump station operation
- Pavement and traffic control
- Permits and approvals

5.1.4 Environmental Impact

Environmental factors will be considered for compliance with existing regulations and to assess the potential for adverse impacts during construction, which could result in additional costs and delays to the project. Applicable environmental factors would include RPA, hazardous substances (asbestos cement) and stream crossings.

5.1.5 Operation and Maintenance

Operation and maintenance (O&M) are important components of a well-functioning collection system management. The operation of the pump station, force main, and downstream gravity system is automated and will not require continuous on-site operator presence. However, the system will be subject to inspection and preventative maintenance on a regular schedule. Therefore, the alternatives will be evaluated for ease of accessibility along the alignment and at the discharge structure to ensure long-term viability of sewer infrastructure improvements.

5.1.6 Cost

Capital improvement cost as well as the life cycle costs of the proposed alternatives will be estimated for the preferred alternative selection process. The capital costs include the materials and equipment costs, construction costs, soft costs, and contingency.

6. Cost Estimate

Rough order-of-magnitude (class 5) estimates for the proposed alternatives are included in Appendix C, Alternatives *Cost Estimate*. Table 15 provides a cost summary breakdown. Estimated costs are provided within a -50 to +75 percent accuracy range. Estimated construction cost developed for the alternatives were based on opinion of cost for previous Jacobs' project as well as cost for projects of similar size and scope. All lengths for pipelines were scaled from as-built and GIS layers obtained from Fairfax County.

Table 15 – Preliminary Construction Cost Estimates

Item	Low Range	Estimate	High Range
DS-1 – Pipe Replacement	$1,885,618	$3,771,237	$6,599,664
DS-2 – Parallel Gravity Sewer	$1,939,114	$3,878,229	$6,786,900
DS-3 – Parallel Force Main	$1,776,809	$3,553,618	$6,218,831
	-50%		+75%

6.1 Markups

The following typical contractor markups were applied to the cost estimate:

- General Conditions 12%
- Contractor Overhead 8%
- Profit 8%
- Mobilization/Demobilization 5%
- Bonds & Insurance: 3%
- Escalation Rate: 7%

6.2 Major Assumptions

The estimate is based on the assumption that the work will be done on a competitive bid basis and the contractor will have a reasonable amount of time to complete the work. All contractors are equal, with a reasonable project schedule, no overtime, constructed as under a single contract, no liquidated damages. This estimate should be evaluated for market changes after 90 days of the issue date. List of other major assumptions:

- Sales tax is not included, the project is assumed to be sales tax exempt.
- Rock excavation is not required.
- Dewatering is not included.
- Assumed excavated material is suitable for backfill.
- Assumed trench dimensions are 6 feet wide by 12 feet deep.
- Temporary trench protection is included.
- Maintenance of traffic is included.
- Relocation of existing storm and sanitary drains are not included.
- Surface restoration of disturbed asphalt pavement areas is included.
- Repairs to existing sidewalks, curbs and gutters are not included.
- Non-construction or soft costs for design, services during construction, land/easement, legal, and owner administration costs are not included.

7. Alternatives Evaluation and Ranking

This section represents the findings and evaluation of alternatives to compare each upstream and downstream alternative. The evaluation to select a preferred alternative was based on the selection factors listed Section 6 and shown in Table 16 below. Each criterion was assigned a subjective rating: Very Low=4 (most favorable), Low=3, Moderate=2, and High =1. These ratings were in respect to the indicator's impact on that particular criterion. That rank was then multiplied by the weighting factor shown in Table 17 to obtain the alternative's score for that criteria.

Table 16 - Subjective Alternative Ratings

Criteria	Indicator	DS-1	DS-2	DS-3
Community Impact	Community disruption during construction	High	Moderate	Low
	Long-term impact (aesthetics, odor, noise,etc.)	Very Low	Very Low	Low
	Traffic management (during construction)	High	Moderate	Moderate
Constructability	Construction method difficulties/risks	High	Moderate	Low
	Utility conflicts/site constraints	Low	High	Moderate
	Bypass pumping	High	Moderate	Low
Environmental Impact	Excavation Activities and disposal of excavated material	High	Low	Very Low
	Dust, noise, damage to trees, pavement and soil erosion	High	Moderate	Low
	Contaminant disposal (AC pipe)	High	Low	Very Low
Operations & Maintenance	Asset maintenance (cleaning, inspection, assessment, & repair)	Low	Low	Moderate
	Operational constraints (lack of redundancy)	Moderate	Low	Very Low
	Risk of failure due to aging infrastructure	Very Low	Low	Low
Cost	Construction cost	High	High	High
Criteria Points		24	31	37
Weighted Ranking		3	2	1

Table 17 - Decision Matrix for Alternatives Comparison

Category	**Category Weight Factor**	**DS-1**		**DS-2**		**DS-3**	
		Raw Scores	Weighted Scores	Raw Scores	Weighted Scores	Raw Scores	Weighted Scores
Community Impacts	20%	6	1.2	8	1.6	8	1.6
Constructability	15%	5	0.75	5	0.75	8	1.2
Environmental Impacts	20%	3	0.6	8	1.6	11	2.2
Operations and Maintenance	30%	9	2.7	9	2.7	10	3
Cost	15%	1	0.15	1	0.15	2	0.3
Total Scores		25	5.40	31	6.8	39	8.3
Normalized Scores		69%		87%		100%	
Overall Ranking		**3**		**2**		**1**	

8. Conclusions and Recommendations

The ranking table provided in Section 7 (Alternatives Evaluation and Ranking) indicated that Alternative DS-3 is the most favored alternative for downstream upgrades. Based on the estimated construction cost, both DS-1 and DS-3 are roughly similar, between $3.7 and $3.5 million, as presented in Table 15. However, DS-3 will have much less community impact, will shorten the construction schedule, eliminate the need to reestablish lateral connections, and provides lower potential risk of accidents during construction because open cut areas will require much more extensive traffic control. Based on all considerations, selection of DS-3 as the installation method for the proposed sewer lines is recommended.

Pump Station Capacity: It is recommended to upgrade the pump station capacity to meet 593 gpm peak flow. This recommendation is applicable to all alternatives. Table 18 presents the recommendation summary.

Table 18 - Lakevale Estates PS Alternatives Evaluation

Alternative	Name	Description	Overall Ranking	Recommendation
DS-1	Pipeline Replacement	Upsize downstream system: install new 12-inch gravity pipe and demolish existing gravity system.	3	Eliminate
DS-2	Parallel Gravity Sewer	Add new gravity pipe parallel to existing sewer to convey pumped flows to location where gravity system has capacity. Leave existing gravity line in place to capture at least some of the local flows.	2	Eliminate
DS-3	Parallel Force Main	Bypass pumped flows in a new force main to a point where gravity system has capacity and leave existing gravity line in place to capture local flows	1	Recommended for further evaluation under the PER
Alternatives Considered and Eliminated				
X-1	Match Downstream Capacity	Downgrade pump station's capacity to match downstream gravity system capacity and maximize utilization of existing infrastructure if conditions allow	Eliminated because alternative would not allow pump station to meet the peak influent flows and limit the pump station's operating strategy to a duty/standby pump configuration	
X-2	Downstream Equalization	Install an equalization basin downstream of the force main discharge	Eliminated due to constructability, O&M and site constraints	
X-3	Extend Force Main across Vale Road	Offload flows to relieve flooding in downstream system	Eliminated due to insufficient hydraulic capacity in the gravity system on Tanglevale Drive	
X-4	Upstream Equalization	Dampen peak pump station influent flows	Eliminated due to constructability, O&M and site constraints	
X-5	Upstream Storage Tank	Provide operational response time and protect upstream homes	Eliminated due to constructability and site constraints	

9. References

- *Fairfax County Public Facilities Manual – Chapter 10 – Sanitary Sewer Design Criteria*
- *Fairfax County Wastewater Guidelines for A/E – Volume 2 – Facility Design Guidelines, July 2021*
- *Infrastructure GIS data provided by Fairfax County*
- *Lakevale Estates Discharge Realignment Basis of Design Review – Final, Dewberry, January 2021*
- *Lakevale Estates PS – Contract No. CN15402004 Record Drawings, May 2019*
- *Lakevale Estates PS – Original Construction Drawings, 1966*
- *Recommendation for Project Initiation – Lakevale Estates Discharge Realignment, WDCD, August 2020*
- *Rehabilitation of Four Small Pump Station – Preliminary Engineering Report – Final, Hazen, August 2013*
- *The Metropolitan Washington Council of Governments (COG) Round 9.1a Growth Projection Data, October 2018*
- *Topo Survey files provided by Fairfax County*
- *Virginia Sewage Collection and Treatment – Chapter 790. Sewage Collection and Treatment Regulations, Part II – Articles 1 and 2.*

Appendix A – Flow Analysis

Four data sources were used to characterize observed wet weather events over the past several years, performing an analysis of the available flow and rainfall data to determine the appropriate peaking factor and design peak flow to be used when evaluating pump station alternatives. Data sources used in the analysis include:

- *Recent flow monitoring data* – flow meter installed in manhole 038-3-013 downstream of the pump station from March 2021-September 2021; 5-minute flow, velocity, and depth data provided.
- *Recent rain gauge data* – Fairfax Center rain gauge data observed from March 2021-September 2021; 5-minute rainfall data provided
- *Historical SCADA data* – influent level at the Lakevale Estates Pump Station wet well and pump run times from April 2019 through September 2021; 1-minute data provided
- *Historical rain gauge data* – Pickett Rd rain gauge data observed from April 2019 through September 2021; 15-minute rainfall data provided

The provided SCADA wet well influent level data and pump run-time data were analyzed to compute the volume of flow through the wet well and ultimately develop the 5-minute flow rate influent to the wet well. This computation is based on a flow balance of wet well volume and pumped flow at each time step. The wet well volume was computed in Revit using the record drawings and the capacity of the wet well based on the pump station operation was evaluated using AFT Fathom. Figure 1 shows the wet storage capacity curve in blue. Using curve fitting techniques within Excel, the following equation for wet well volume was developed:

$$5.051(x)^3 + 18.44(x)^2 + 28.138(x) + 0.2071$$

Where x equals the wet well level in feet. The volume over each recorded time step in minutes was used to generate the flow rate over that time step. When a pump is running, the level in the wet well will drop, and this equates to a negative change in volume or flow rate. Therefore, the calculation assumes that the flow into the pump station is equal to the pumped flow plus any change in volume in the wet well over the time step.

Hydraulic modeling of the Lakevale Estates PS including suction piping and force main, was performed using the Applied Flow Technology (AFT) Fathom software. AFT Fathom is a fluid dynamic simulation software that can be utilized to calculate pressure drop and flow distribution in a hydraulic system. It provides capabilities to model, analyze, and simulate hydraulic systems in order to evaluate the performance of existing and/or new designs and assure compliance with design requirements. System configuration information for input into the AFT Fathom model (such as pipe length, elevation, fittings, etc.) was determined using available record drawings. This model was used to evaluate the system head conditions for the Lakevale Estates PS over the range of operating conditions. Also, Jacobs modeled various flow scenarios to assist with the evaluation of the historical SCADA and flow data.

A typical flow rate of 492 gpm for the 10 ¾-inch impeller pump and 720 gpm for the 12-inch impeller pump was used for this computation, based on output from the AFT Fathom model. It was assumed that when the SCADA indicates two pumps are on that the pumped flow rate out of the pump station is equal to the capacity of the larger impeller pump only (720 gpm).

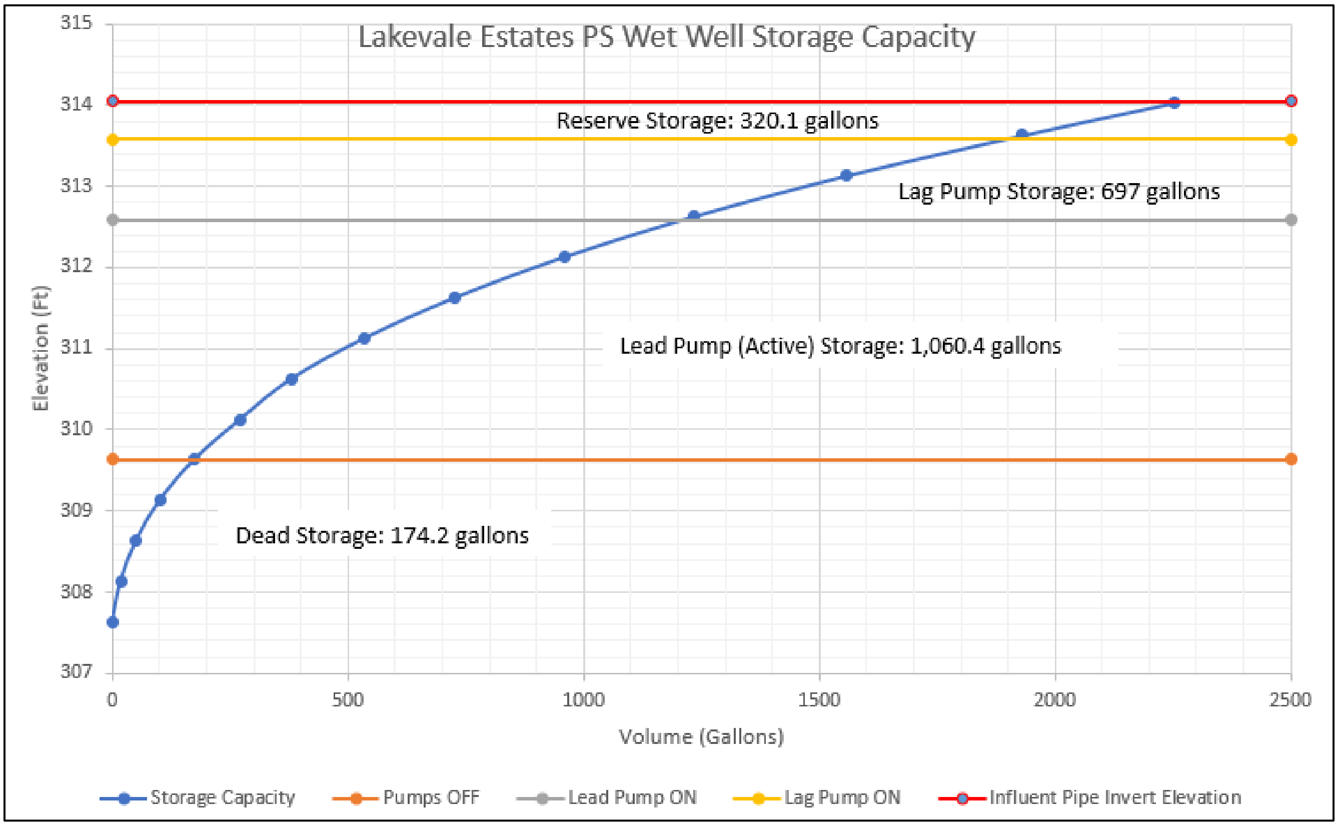


Figure B-1: Lakevale Estates PS Wet Well Storage Capacity

The SCADA flow data was reviewed against the observed rainfall data from the Fairfax Center rain gauge during the flow monitoring period to determine the overall system response to rainfall influence and determine where data gaps may exist. Additionally, the observed flow monitoring data was reviewed against the observed Fairfax Center rainfall data to examine system response to rainfall influence and look for data gaps that may exist, if any.

After review of each of the flow data sources, the project team decided to use the SCADA wet well influent data as the primary flow data source for analysis of peak flows into the pump station because peaks at the downstream flow meter are attenuated (reduced and extended) by the pump capacity and gravity sewer pipe capacity limitations. In addition, the SCADA data covered a longer period of record. In addition, the cycling of the pumps impacted the flow meter data, creating frequent fluctuations. As such the SCADA data was determined to be better quality and more accurately captured observed peaks. The Pickett Rd rain gauge data was used only to characterize storm events prior to the flow monitoring period.

Average Daily Flow

The average daily flow, or dry weather flow (DWF), used in the wet weather peaking factor calculations was 226 gpm. To determine this value, the flow data was analyzed using the Environmental Protection Agency (EPA) Sanitary Sewer Overflow Analysis and Planning (SSOAP) Toolbox. This tool disaggregates flow into base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall derived infiltration and inflow (RDII) for sanitary systems using flow data, rainfall data, and sewershed area. For the purposes of this analysis, the calculated flow data at the wet well from SCADA was used, as well as the observed rainfall from the Fairfax Center rain gauge. The sewershed area was calculated as the area tributary to the wet well (approximately 700 acres). The following sections include a more detailed description for selection of the average daily flow.

An observed DWF pattern was developed within SSOAP to determine the average daily flow and the diurnal peaking factors. The GWI and BSF were also calculated. Note that the sum of these two values is equal to the total DWF. The observed DWF data was compared against the received model previously calibrated by Black & Veatch for DWF. In the model, the subcatchments include both of the DWF parameters: GWI is modeled as "Base Flow" and BWF is modeled as "Additional Foul Flow". Base Flow is a constant flow input used to represent the GWI within the subcatchment area. Additional Foul Flow represents the BWF which is multiplied by the diurnal pattern peaking factors and varies over time. This comparison serves as a method for validation of the model data against the observed data. A more detailed description of the model updates and additional DWF validation is presented in Appendix B of this report.

Table A-1 below summarizes the DWF parameters for the observed SCADA data and the received model calibrated for DWF. The calibrated model average DWF is within 5 percent of the observed flow data from SCADA. A comparison of these values suggests the calibrated model dry weather flow estimate of 226 gpm is a good estimate for these sewersheds, and this value was used as a basis for remaining computations.

Table A-1: Summary of DWF parameters for observed SCADA data and the calibrated model

	Average DWF (gpm)	Average GWI (gpm)	Average BWF (gpm)	Diurnal Peak Flow (gpm)	Diurnal Peaking Factor
SCADA Data	220	94	126	316	1.44
Calibrated Model	226	100	126	294	1.53

Wet Weather Peaking Factor

The wet weather flow peaking factor developed as part of the flow analysis was 2.5. This factor is a ratio of the peak observed hourly average flow to DWF used to develop the pump station design flow. VSCAT regulations suggest pump stations should be designed for a minimum of 2.5 times the average daily flow, or the expected maximum flow, whichever is greater. An analysis of historical rainfall and SCADA flow data was performed to determine the most appropriate peaking factor.

Table A-2 summarizes the results of the flow data analysis providing total rainfall, storm recurrence, peak hourly average flows, and computed peaking factor for 14 identified storm events from 2019 to 2021. 5-minute peak flows were computed and are provided in the table for reference, however for the purpose of designing a pump station hourly peak flows are more appropriate. The analysis indicates that a peaking factor of 2.5 would be a conservative estimate of flows as the range of peaking factors developed is from 1.4 to 2.3. Therefore, a factor of 2.5 was chosen for the pump station analysis.

Table A-2: Summary of wet weather flow analysis, including storm intensity, frequency, observed flows, and calculated peaking factor

Event ID	Event Start	Event End	Duration (Hours)	Total Rainfall Volume (Inches)	15-minute Rainfall Intensity (in/hr)	1-hr Rainfall Intensity (in/hr)	6-hr Rainfall Intensity (in/hr)	1-hr Rainfall Intensity Storm Recurrence (Frequency)	Peak Observed Hourly Average Flow (gpm)	Average DWF (gpm)	Hourly Average Peaking Factor	Peak Observed 5-minute Flow (gpm)
1	7/8/2019 5:30	7/8/2019 15:00	9.50	2.17	2.22	1.95	0.36	5-year	341	226	1.51	645
2	8/15/2019 16:00	8/15/2019 17:45	1.75	1.95	3.36	1.62	-	2-year	476	226	**2.10**	617
3	10/20/2019 5:15	10/20/2019 19:00	13.75	1.70	0.39	0.24	0.17	<1-year	341	226	1.51	473
4	10/27/2019 3:15	10/27/2019 17:15	14.00	1.43	1.59	0.69	0.22	<1-year	344	226	1.52	493
5	10/31/2019 20:45	10/31/2019 23:45	3.00	0.99	1.48	0.76	-	<1-year	363	226	1.61	490
6	1/24/2020 23:15	1/26/2020 0:00	24.75	1.55	0.84	0.44	0.25	<1-year	359	226	1.59	483
7	4/12/2020 20:00	4/14/2020 0:00	28.00	1.76	1.18	0.54	0.23	<1-year	405	226	1.79	480
8	4/30/2020 8:15	4/30/2020 18:15	10.00	1.01	0.24	0.21	0.13	<1-year	360	226	1.59	485
9	6/19/2020 3:30	6/20/2020 2:30	23.00	2.90	3.11	1.96	0.35	5-year	317	226	1.40	464
10	6/10/2021 18:45	6/12/2021 8:50	38.08	1.36	1.6	0.52	0.12	<1-year	426	226	1.88	475
11	6/14/2021 22:15	6/16/2021 6:40	32.42	1.60	2.72	1.52	-	2-year	525	226	**2.32**	612
12	7/1/2021 14:45	7/2/2021 7:15	16.50	1.36	2.72	0.84	0.17	<1-year	432	226	1.91	546
13	8/19/2021 19:15	8/22/2021 0:00	52.75	2.04	0.96	0.76	0.31	<1-year	462	226	**2.04**	525
14	9/1/2021 1:35	9/2/2021 21:00	43.42	1.76	3.84	1.64	-	2-year	360	226	1.59	535

Collection System Flow Projections

This Section summarizes the three different methods utilized to evaluate existing flows measured during the study period and presents the calculation for the design flows used to model the existing and future collection system.

Method 1 - Average Dry Weather Flow

Under this method, VSCAT recommended peaking factor was applied to a calculated average dry weather flow to determine the appropriate peak flow.

As described in the previous sections, historical flow data was used to determine the most appropriate DWF for each of the three projected peak flow calculation approaches. Sources of data included the provided downstream flow meter data, SCADA data at the wet well, and the provided Infoworks ICM model calibrated for DWF.

Data from the downstream flow meter was analyzed first. Two weeks of DWF were selected based on observed rainfall data and the average flow over those weeks was calculated. A week in the spring as well as a week in the summer were chosen to evaluate potential impacts of GWI, which is typically higher in the spring than it is in drier summer months. The following weeks and average flows were computed, including an average of the entire data set:

- Week of March 4-11, 2021, Average DWF: 236 gpm
- Week of August 2-9, 2021, Average DWF: 160 gpm
- Average DWF for Identified DWF Periods: 198 gpm
- Average Daily Flow, including wet weather periods (Mar-Aug 2021): 182 gpm

The provided SCADA data was also analyzed to determine the average flows influent to the Lakevale Estates PS wet well. Wet well levels were provided and volume into and out of the wet well was computed based on wet well dimensions and estimated flow rates for each pump developed in the AFT Fathom model (more information is provided in Appendix B). Using this flow balance approach, the same two-week dry weather periods were evaluated as well as an average of the entire dataset, as follows:

- Week of March 4-11, 2021, Average DWF: 265 gpm
- Week of August 2-9, 2021, Average DWF: 181 gpm
- Average DWF for Identified DWF Periods: 220 gpm
- Average Daily Flow, including wet weather periods (Mar-Aug 2021): 224 gpm

Finally, the received calibrated model was evaluated to determine the calibrated base flow parameters and how they compare to the other data sources. There are two input parameters in the model that contribute to the total base flow: GWI and base sanitary flow. A summation of the calibration data input into the model upstream of the pump station provides the follow estimate of DWF:

- Base sanitary flow: 100 gpm
- Groundwater infiltration: 126 gpm
- Average DWF at the wet well: 226 gpm

A comparison of the three calculation methods suggests that the calibrated model DWF estimate of 226 gpm is a reasonable estimate for this sewershed as this value, though similar to the other averages computed, is the most conservative and is based on observed flow meter data that would not have been as significantly impacted by pump station operation as the flow meter data that was provided.

Table A-3: Summary of flow data

Description	DWF (gpm)	ADF (gpm)
Flow Meter Data	198	182
SCADA Data	220	224
Calibrated Model	226	-

DWF – Dry weather flow is average of two 1-week dry-weather periods (spring and summer)
ADF – Average daily flow based on full period of record (including wet weather flow)

The observed peaks within the provided flow meter data are attenuated (reduced and delayed) by capacity limitations in both the pump capacity as well as the downstream gravity system and therefore the downstream flow meter data received for this project does not capture the full peak flow influent to the pump station. Therefore, 226 gpm was the value used as the basis for the peak flow projections.

According to VSCAT, a minimum peaking factor of 2.5 should be applied to average flows to develop projected pump station peak flows. Therefore, applying a 5% growth factor based on the MWCOG population projections and typical pump station peaking factor of 2.5 produces a peak flow of **593 gpm.**

Method 2 – Historical Peak Wet Weather Flow

For this analysis, the provided SCADA data was analyzed to determine influent flows at the wet well during historical rain events. Rainfall data from Fairfax Center and Pickett rain gauges were used to characterize the rainfall events that produced the peak flows. The peak flows from the SCADA analysis and event characteristics are summarized in Table A-2. Based on this data, the peak hourly flow recorded entering the pump station between January 2019 and August 2021 was 525 gpm. This equates to a peaking factor of 2.32. The largest storm events that occurred during this timeframe were 2-year to 5-year return period events based on 1-hour peak rainfall intensity. Larger peaks are likely during larger storm events. Pump stations are typically designed to accommodate peak hour discharges, rather than shorter duration peaks, as long as sufficient storage is available upstream to provide sufficient equalization.

Applying a 5% growth factor (based on the MWCOG population projections) to the peak hourly flow rate of 525 gpm (2021 data), suggests a projected peak hourly discharge of 551 gpm.

It should be noted that historical reports were reviewed to confirm if there were historical events that exceeded the recent records. According to the 2013 PER, the highest recorded flow prior to the design of the pump station rehabilitation was during Tropical Storm Lee in September 2009. They estimated peak hourly flows at that time of 398 gpm (0.574 MGD), relatively lower than flows measured between 2019 and 2021.

According to the County TM, "The highest recorded single data point for wet well level rise over 6 minutes was 728 gpm which occurred on October 22, 2015, at 2:06 pm (data point 85341) when the wet well level rose over 7 feet in 6 minutes. The other annual maximum inflow data points ranged from 324 gpm to 517 gpm, suggesting that normally the net inflow is less than 517 gpm." These values are in a similar range to the 5-minute peaks of 500-645 gpm observed in 2019-2020.

Method 3 – Wastewater Development Flow Factors

Flow factors were utilized to develop wastewater flow projections and allocate future flows to the collection system in Lakevale Estates pump station sewershed. Table A-4 includes the number of residential and commercial units in each subdivision along with the estimated average daily flows from these properties, which are based on Fairfax County PFM, VSCAT, and engineering judgment. This analysis generated an average daily flow of 380,730 gpd from residential properties and 79,214 gpd for commercial properties, for a total ADF of 459,944 gpd. Applying a typical peaking factor of 2.5 to this ADF generates a peak flow of **799 gpm**.

Table A-4: Wastewater Flow Computations

Design Flow Calculations Based on Existing Residential and Commercial Development in Lakevale Estates PS Service Area - Update November 29, 2021									
Residential Development	Single Family Units	Flow, gpd = Count x 350[1] gpd	Commercial Development (Oakton Shopping Center)		Unit of Flow Measurement	Flow/Unit	Estimated Number of Units	Estimated Flow
	(Count)	(gpd)			(Units)	(gpd/Unit)	(Units)	(gpd)
Ashlawn Hill	16	5,600	Tropical Café Smoothies	Restaurant	seats	50 [2]	25	1,250
Ashlawn Section 2	40	14,000	Wild Bird Center Retail	Retail Store	sq. ft.	0.20 [1]	10,000	2,000
Lakevale Court	48	16,800	Pure Organic Nails and Spa	Barber Shop	sq. ft.	0.20 [1]	10,000	2,000
Lakevale Estates	200	70,000	Music Arts	Retail Store	sq. ft.	0.20 [1]	10,000	2,000
Acredale	32	11,200	Cleaners	Cleaners	sq. ft.	0.45 [3]	10,000	4,500
Five Oaks	35	12,250	Pet Value	Retail Store	sq. ft.	0.20 [1]	10,000	2,000
Oakton Grove 1	32	11,200	Famous Daves Barbecue	Restaurant	seats	50 [2]	50	2,500
Oakdale Woods	18	6,300	Oakton Urgent Care	Medical Office	sq. ft.	0.5 [2]	20,000	10,000
Oakvale Estates	32	11,200	Starbucks	Restaurant	seats	50 [2]	20	1,000
Rainey vale	7	2,450	UPS	Retail Store	sq. ft.	0.20 [1]	10,000	2,000
Oak Valley Court	6	2,100	Virginia ABC	Retail Store	sq. ft.	0.20 [1]	10,000	2,000
Oakton Grove 2	9	3,150	Yobe Frozen Yogurt	Restaurant	seats	50 [3]	25	1,250
Vienna Glen	5	1,750	Hallmark	Retail Store	sq. ft.	0.20 [1]	10,000	2,000
double circle 1	4	1,400	Clawes Carpets	Retail Store	sq. ft.	0.20 [1]	10,000	2,000
Oakton Glen Sec. 1&2	148	51,800	California Tortilla	Restaurant	seats	50 [3]	50	2,500
Cody Tract	31	10,850	Bank of America	Bank	sq. ft.	0.20 [1]	15,000	3,000
Oakton Glade	25	8,750	Giant Food Store	Supermarket	sq. ft.	0.20 [1]	30,000	6,000
Soltas Manor	34	11,900	Sunoco Gas Station	Service Station	sq. ft.	0.20 [1]	10,000	2,000
Oakcrest Farms Sect. 1	14	4,900	RGS Title Co.	Office	sq. ft.	0.20 [1]	10,000	2,000
Lewis Manor	24	8,400	CVS Pharmacy	Drug Store	sq. ft.	0.20 [1]	20,000	4,000
Oakcrest	30	10,500	Squisit Pizza	Restaurant	seats	50 [3]	25	1,250
Hickory Hollow Forest	21	7,350	Oakton KinderCare	School	capita	16 [1]	100	1,600
Oakton Green	6	2,100	Sunrise of Hunter Mill	Nursing Home	beds	200 [2]	50	10,000
Oakcrest Farms Sect. 2	69	24,150	Oakton Library	Library	sq. ft.	0.20 [1]	20,000	4,000
English Oaks	7	2,450	Unity of Fairfax Church	Church	seats	5 [3]	200	1,000
Wyant Property	10	3,500	Oakton Auto Service Center	Garage	sq. ft.	0.2 [2]	10,000	2,000
Oakton Manor	38	13,300	Pinnacle Academy	School	capita	16 [1]	75	1,200
Hearthstone Village	54	18,900	Lakevale Center Tennis	Tennis Club	courts	432 [3]	2	864
Total Single Family Units	**995**	**348,250**	Lakevale Estates Community Center		*(estimated)*			300
Wyant Townhouses (280[1] gpd)	116	32,480	Lakevale Pool	Pool/Showers	members	10 [2]	100	1,000
Total Residential Average Daily Flow		**380,730**	**Total Commercial Average Daily Flow**					**79,214**
Design Flow Value Sources:			**Design Total Average Daily Flow (gpd)**					**459,944**
[1] Fairfax PFM Requirements			**Peak Factor**					**2.5**
[2] VSCAT Regulations			**Design Peak Flow Rate (mgd)**					**1.15**
[3] Engineering Judgment			**Design Peak Flow Rate (gpm)**					**799**

Recommended Peak Flow Rate

The results of the three methods were compared (Table A-5) and Method 1 was selected for the design flow. The per capita and unit-based wastewater estimates used in Method 3 are known for being very conservative, and measured flows suggest this approach is substantially overestimating flows delivered to Lakevale Estates PS, therefore this method was not selected. Because the period of record was relatively short, and more significant wet weather events could produce higher peaks, Method 1 was selected as the more conservative estimate. It is still within the order of magnitude of actual measured peaks and provides an additional level of conservatism.

Table A-5 - Estimated Peak Inflows to Lakevale Estates Pump Station

Method	Description	Peak Flow Rate, gpm
1	Average Dry Weather Method	593 [a]
2	Historical Peak Flow Method	551 [a]
3	Development Flow Factors	799

a. Values include 5 % population growth

Appendix B - Collection System Model Updates

Black & Veatch (B&V) is currently working on a system-wide hydraulic model for the County. This work is being completed concurrently to the Lakevale Estates Pump Station Gravity System Modeling & Hydraulic Improvements project. The County provided Jacobs with a clipped version of the B&V model that contains the Lakevale Estates area. B&V calibrated this model for dry weather flow and Jacobs reviewed and updated the model file as necessary to suit the needs of this project. Model parameters, including wet well storage dimensions, pump/on off levels, and sewer inverts were updated to reflect current conditions per provided construction drawings and system survey data. Review of the model also included the validation of B&V's dry weather flow (DWF) calibration against the project flow monitor data and adjustment of the model parameters to effectively match the model-projected peak depth with the observed peak depth during the August 13th power outage event that recently caused flooding.

Initial Model Review and Updates

The received model was reviewed, and specific parameters were updated based on provided construction drawings and recent survey data. Key changes included updates to the wet well dimensions, pump on/off levels, and sewer inverts. A summary of the updated information is presented in Table B-1 below.

Table B-1: Summary of Updated Model Parameters Upon an Initial Review of Received Model

Model Parameter	Received Model Data	Updated Model Data	Justification/Source of Change
Wet well storage array	Cylindrical with volume = 1,320 gallons	Conical shape with volume = 2,252 gallons	Updated to match as-built drawings and field visit photos as best as possible. This change was flagged 'ES' in the updated model to reflect that this is an estimated value/change.
Pump on/off levels	Lead on = 313.9 ft Lead off = 309.8 ft Lag on = 314.4 ft Lag off = 309.8 ft	Lead on = 312.58 ft Lead off = 309.63 ft Lag on = 313.58 ft Lag off = 309.63 ft	Updated to match pump station control setpoints recorded from site visit. This change was flagged 'FV' in the model to reflect that it is a field verified value.
Lead/lag pump configuration	Lead pump = larger, 619 gpm pump Lag pump = smaller, 450 gpm pump	Lead pump = smaller, 450 gpm pump Lag pump = larger, 619 gpm pump	Review of the observed flow data against the modeled flow data suggests that lead pump is the smaller of the two pumps; the County confirmed this is the current operating procedure. This change was flagged 'FV' in the model to reflect that it is a field verified value.
Force main roughness (Hazen-Williams coefficient)	80	120	The force main was recently lined and is in fair condition. 120 is a more suitable roughness than 80. This change was flagged 'ES' in the updated model to reflect that this is an estimated value/change.
Force main diameter	7.57 inches	7.55 inches	Updated to match Fathom model input – as described in Appendix B.

Model Parameter	Received Model Data	Updated Model Data	Justification/Source of Change
			This change was flagged 'ES' in the updated model to reflect that this is an estimated value/change.
Force main length	1,958.5 ft	1,944.2 ft	Horizontal length/distance provided in the received model/GIS is greater than the actual length when accounting for the depth of the force main. This change was flagged 'ES' in the updated model to reflect that this is an estimated value/change.
Sewer inverts	Various	Various changes	Sewer inverts were adjusted to match the provided survey results. All changes were flagged 'SUR' in the updated model.
Manhole invert and ground elevations	Various	Various changes	Manhole inverts and ground elevations were adjusted to match the provided survey results. All changes were flagged 'SUR' in the updated model.

Dry Weather Flow Validation

Once the model was updated to reflect current conditions, the model was run over different dry weather flow periods to validate the DWF calibration against the flow monitor data. The received model subcatchments included two DWF parameters: groundwater infiltration (GWI) modeled as "Base Flow" and base wastewater flow (BWF) modeled as "Additional Foul Flow". One diurnal pattern was developed for the project area. In the model, Base Flow is a constant flow input used to represent the GWI within the subcatchment area. Additional Foul Flow represents the BWF which is multiplied by the diurnal pattern and varies over time.

An initial run of the model for DWF conditions in August 2021 indicated that the model is over predicting DWF when compared against the project flow monitor data. The general DWF pattern projected by the model seems to correspond to the flow monitor data, but model-projected DWF was nearly 50% higher than the observed value (0.23 MGD observed vs. 0.33 MGD modeled). An initial thought was that the modeled GWI may be too high. Another model run was completed without the Base Flow component. The results of each run are compared against the observed data in Figure D-1.

August is typically a drier month and GWI is typically lower than in winter or spring months. An additional run during DWF conditions in March 2021 was completed as a check of this assumption. The modeled flows with Base Flow were a close match to the observed, while the modeled flows without Base Flow were much lower, as shown in Figure D-2. The B&V team confirmed that the model was calibrated for flows recorded during winter/spring, when GWI would be higher. This higher estimate is more conservative and therefore the Jacobs team did not make any changes to the DWF calibration.

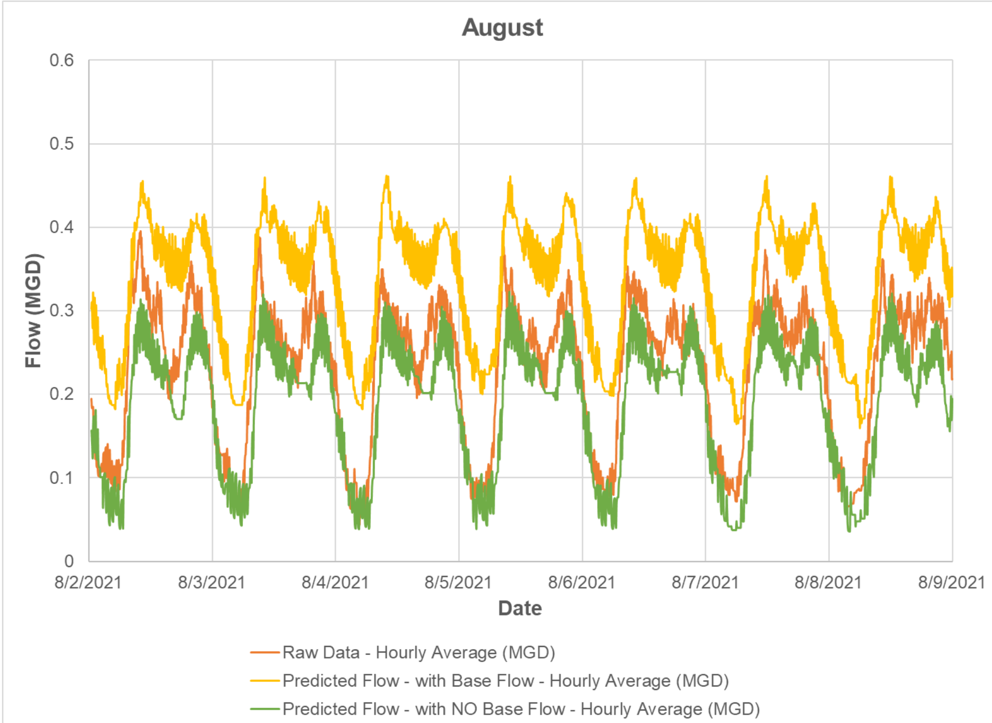


Figure D-1: Modeled flow data with and without Base Flow component for August DWF period against observed data.

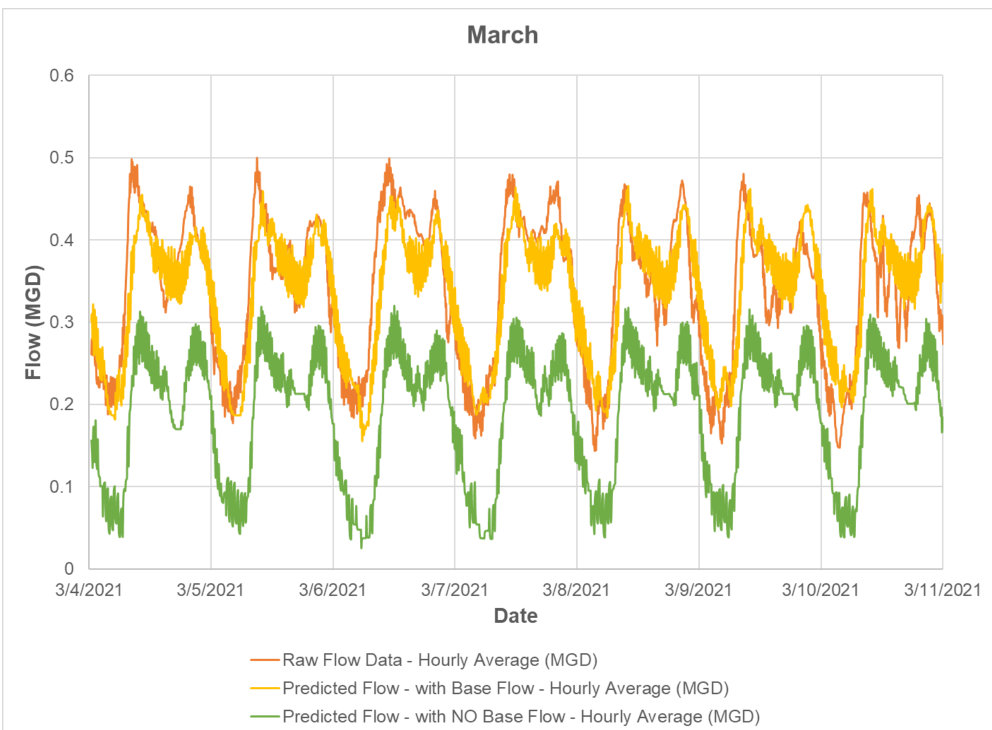


Figure D-2: Modeled flow data with and without Base Flow component for March DWF period against observed data.

Depth Calibration

Because the main purpose of this project is to prevent future basement flooding, a critical part of the model review process was to ensure that the model is correctly projecting peak depths when compared to the observed flow data. To check this, the observed flow data was loaded into the model at the discharge force main point and the peak modeled flow depth was compared against the observed peak flow depth at the meter location. Additionally, the projected peak HGL along the main sewer trunk downstream of the pump station was compared against basement elevations along the modeled sewers to determine whether the model correctly predicts flooding during high flow events, such as the August 13th power outage.

Review and Update of Pipe Calibration Data in the Model

Many of the gravity sewers both upstream and downstream of the pump station have been lined and the received model reflected this through adjustments to the pipe diameter. Lined pipes in the model were about 0.5 inches smaller than their nominal diameter provided in GIS to account for the thickness of the liner, as indicated by the County to be about 6 mm thick. Therefore, the pipe segments downstream of the force main that were specified as 10-inches in the sewerline GIS were included in the dry weather model as 9.5-inches. The received model also had applied minor losses throughout the system and a Manning's roughness coefficient of 0.013 was applied system-wide. An initial model run over the August 13th power outage event produced model-projected peak depths nearly 30% lower than observed and little to no surcharge in the gravity system, projecting no basement flooding. Therefore, further investigation into the received model data was warranted and changes were made to the model to better match the observed flow depths and projected flooding incidents.

After reviewing the flow monitoring report and discussing field measurements and practices in more detail with ADS, the following key changes to the model were made:

- *Updated lined pipe diameters* – ADS indicated that the flow monitoring pipe is 9-inches in their flow monitoring site report. In follow-up discussions, they confirmed that their measurements are accurate to within 1/12 of an inch. Jacobs assumes that the liner thickness initially provided to the County as 6 mm is actually closer to 1.2 mm. Because all pipes were lined at the same time, all lined pipes in the model from manhole 037-4-003 to manhole 038-3-006 were updated to reflect this diameter change.
- *Added sediment depth to flow monitor pipe in the model* – Analysis of the depth-velocity scatterplot provided by ADS against Manning's equation for pipe capacity indicates that the pipe operational diameter is about 8.6-inches. Though no silt was observed during the flow monitor installation, evidence of significant grease build-up was observed by ADS during flow monitoring, and also noted in the CCTV reports. For this reason, half an inch of sediment depth was added to the pipe, making the effective diameter 8.5-inches. Sediment depth was only added to the pipe segment with the flow meter.

Minor loss coefficients applied to the received model varied from 0 to 1.04, with more than half of pipes having no minor losses applied. To ensure adequate system response in the model for peak depth, an entrance and/or exit loss coefficient of 0.15 was added to each model pipe that previously had no losses applied. Additionally, greater losses were applied to pipes downstream of the pump station discharge force main, particularly at 90-degree and 45-degree bends in the system.

The Manning's roughness coefficient for the lined pipes was adjusted. Industry standards suggest the use of 0.09-0.011 for newly lined pipes, but CCTV records show that the lined pipes have some grease build up and are not as smooth as a newly lined pipe. The roughness coefficient was used as a calibration parameter to match measured flow depth as closely as possible. The resultant roughness coefficient after calibration was 0.012.

Finally, review of the predicted flow hydrograph as it reached the location of the flow meter indicated there was significant attenuation of the peak flow due to the amount of surcharge during this event. With the knowledge that the pump station could generate a higher peak flow under the low head conditions that occurred in this August

2021 event, and the amount of attenuation seen in the model, the peak of the hydrograph was increased by 40 gpm to produce a hydrograph that matched the observed hydrograph at the flow meter.

Model vs. Monitored Peak Depth Results

Using the updated pipe data to account for liner thickness, losses, and grease build-up, the peak depth results during the observed August 13th power outage event look much better than results with the pipe data in the originally received model. Figure B-3 shows the depth comparison of the two model runs against the observed data. Note that hourly averages are shown in the graph due to the noisiness of the data from the pump upstream. The absolute differences in peak depth are shown in Table B-2. Additionally, the depth-velocity and depth-flow scatterplot comparison for both runs are shown in Figure B-4 and Figure B-5, respectively. The results for the run with updated diameter, roughness, and minor losses are shown in orange and closely match the observed data, shown in blue.

Table B-2: Summary of peak depth results for received and updated model runs against observed data for August 13th power outage event

Model Run/Scenario	Peak Depth (ft)	Difference (ft)	Difference (in)	% Error
Observed Depth	0.998	-	-	-
Received model parameters – **0.013 roughness and 9.5” lined pipes**	0.621	-0.377	-4.5	-38%
Updated model parameters – **0.012 roughness, 9” lined pipes, updated losses, and ½” sediment depth**	1.127	0.129	1.5	13%

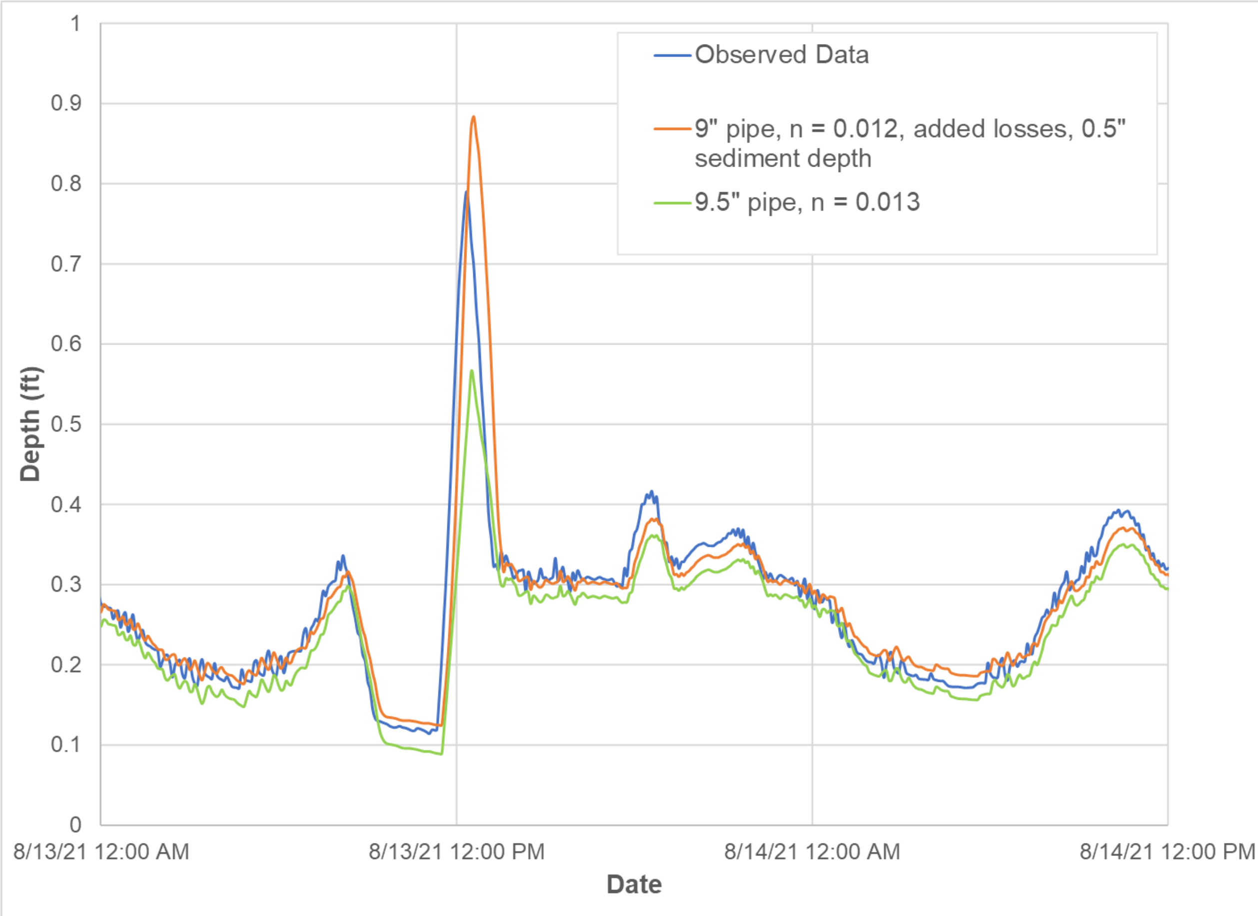


Figure B-3: Peak depth comparison of model-projected results against observed flow data for flows during August 13th power outage event

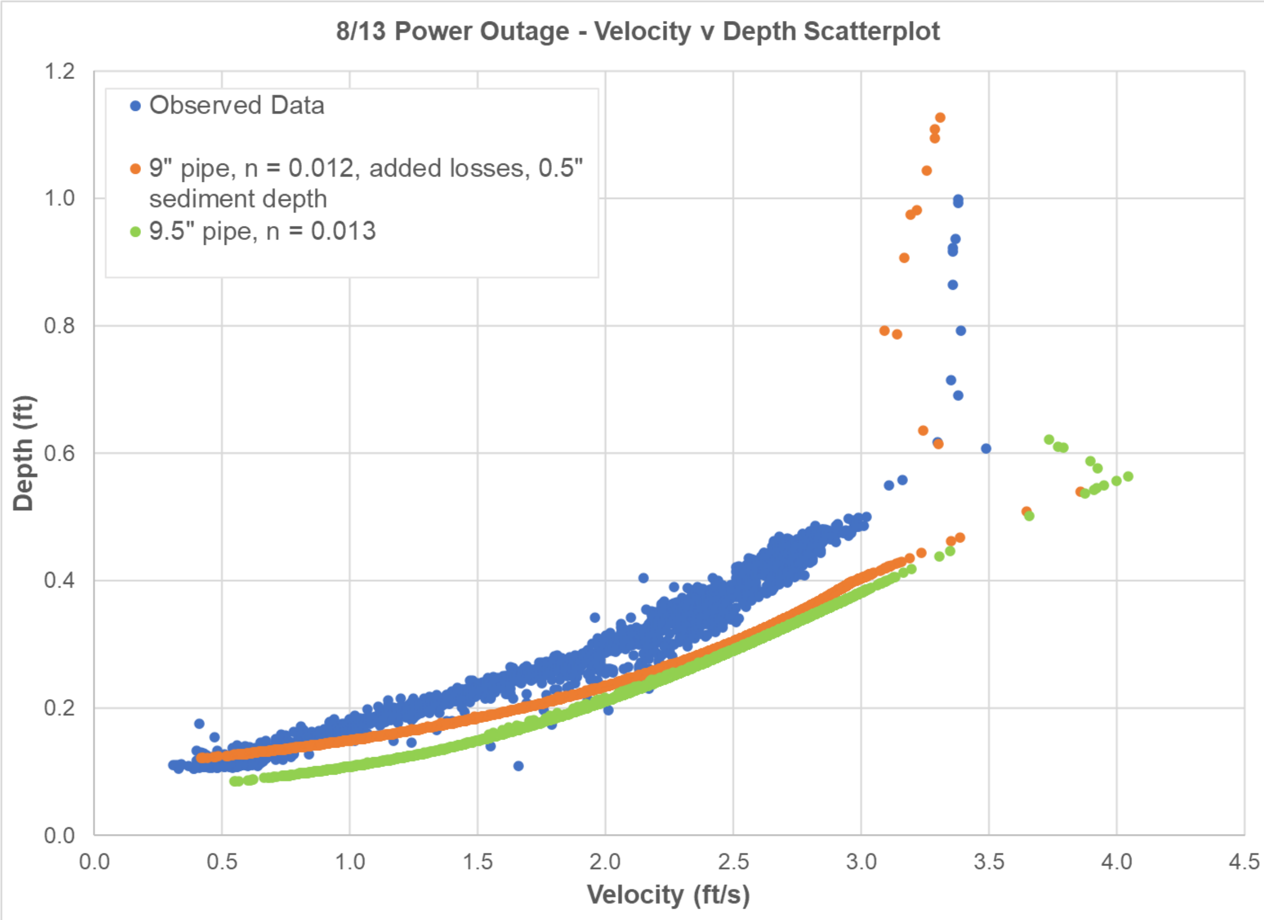


Figure B-4: Velocity-depth scatterplot of model-projected results against observed data for flows during August 13th power outage event

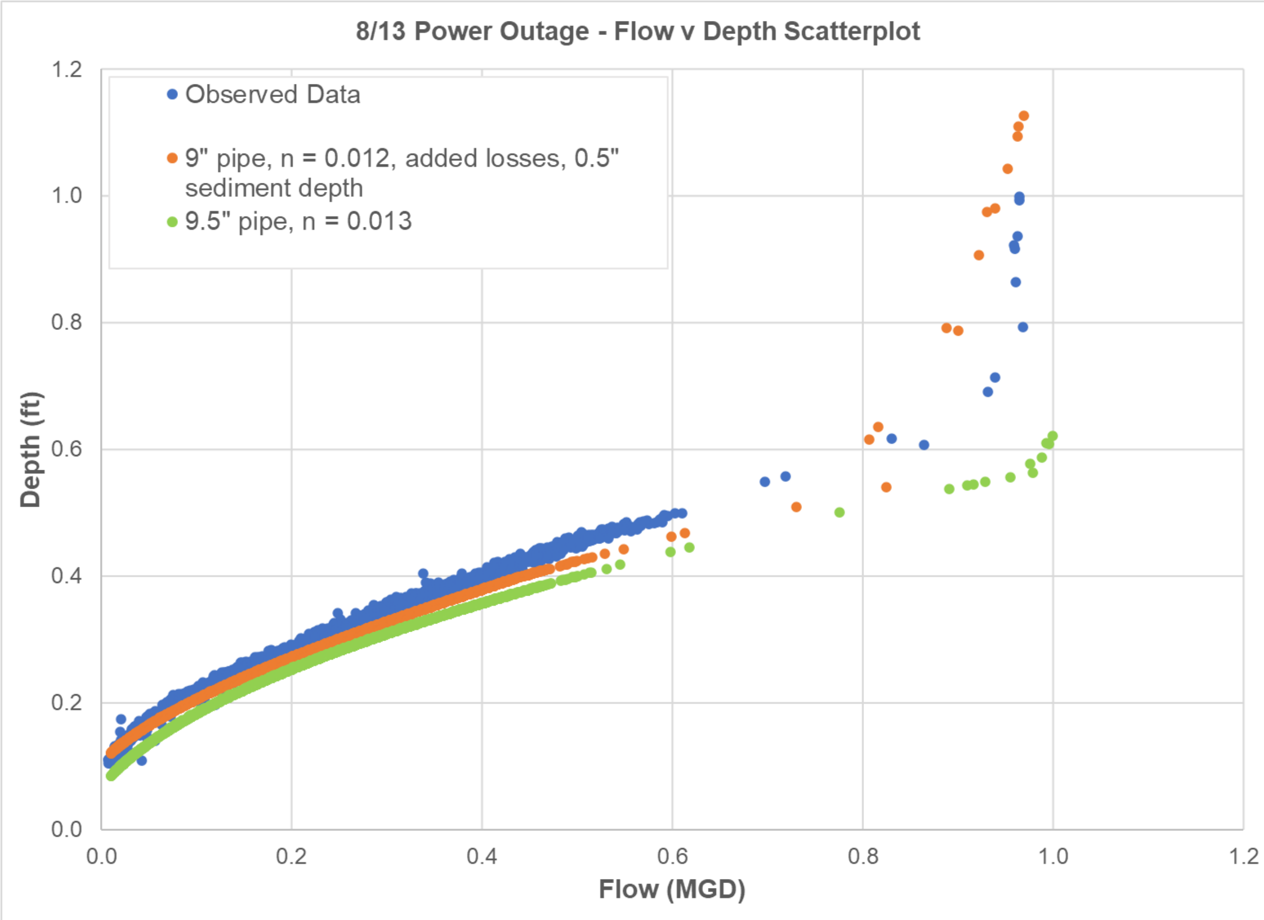


Figure B-5: Flow-depth scatterplot of model-projected results against observed data for flows during August 13th power outage event

Model-Projected vs. Observed Flooding Results

Buildings are projected to be at risk of basement back-up (BBU) if the model maximum HGL elevation is within 8.8 feet of grade at a building along a modeled pipe. Figure B-6 shows an example of this approach. This analysis uses ground elevations based on provided 2-foot contour data and assumes basements elevations are the ground elevation plus the number of front stairs x 7 inches each, less 6 inches for first floor slab height, 8 feet floor to ceiling basement heights, and 4 inches for basement slab height at each structure. CCTV records were used to evaluate the location of laterals along each sewer pipe, and lateral inverts were calculated through interpolation of sewer inverts.

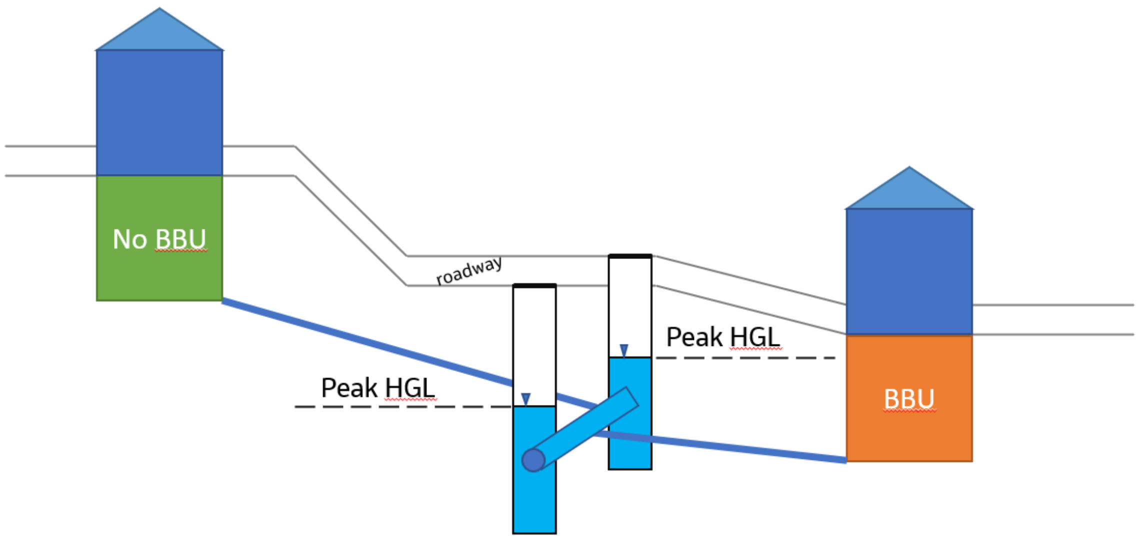


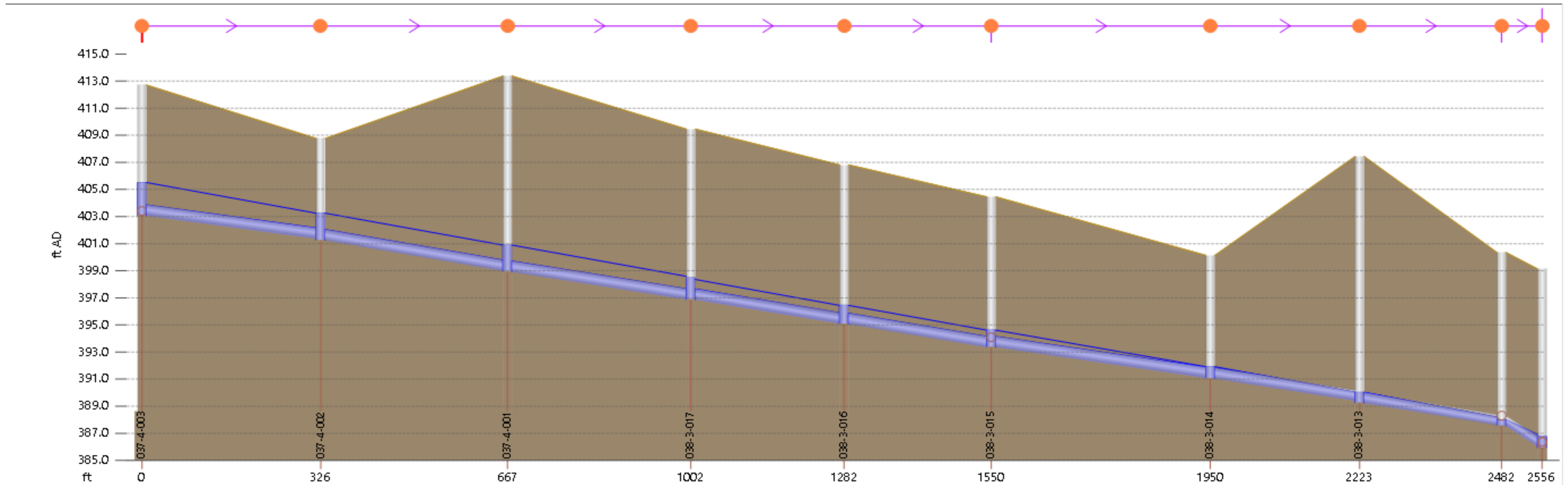
Figure B-6: BBU projection approach based on modeled peak HGL and building basement elevations

The results of the August 13th, 2021 power outage event, using the pipe parameters in the B&V model did not produce a sufficiently high HGL in the main sewer along Newton St downstream of the pump station to produce BBUs at the homes with reported flooding incidents, as shown in Figure D-7. However, when updates to the lined pipe diameters and model losses were accounted for in the model, the projected peak HGL showed greater surcharge and flooding was predicted in 4 of the 5 of the homes with reported flooding incidents. Estimated basement elevation for the fifth house is within 6 inches of the modeled HGL, which is well within the expected accuracy of the model and basement elevation estimates. The peak HGL for the updated model conditions is shown in Figure B-8; the reported flooding incidents for the August 13th, 2021 power outage event are located along the most upstream sewer in the profile. Buildings further downstream along Newton St were checked for potential flood risk and, based on the estimated basement elevations and projected peak HGL several houses are projected to flood along these sewers. The County agreed this was an acceptable outcome because the houses may not have basements, may not have bathrooms in the basements, or the basement elevation estimates may be off. A summary of results for the houses evaluated as part of this analysis is shown in Table D-3.

To confirm flooding is not projected during events with lower peak flows, the model was also run over the September 1st, 2021 rain event and the peak HGL was checked against nearby basement elevations. As shown in Figure D-9, there is very little surcharge projected by the model in the pipes along Newton St during this event. This corresponds to the fact that no houses reported flooding during this rain event.

A final model run was conducted to confirm the model would predict street flooding during the July 2nd, 2019 event. A constant discharge of 785 gpm was entered into the model to represent conditions from that day, when one pump with a 12-inch impeller was running with an upstream water level in the wet well that had surcharged to the ground surface. Under this condition, the model predicted flooding at manhole 037-4-003 after 30 minutes of pumped flow. The peak HGL profile for this scenario is shown in Figure D-10.

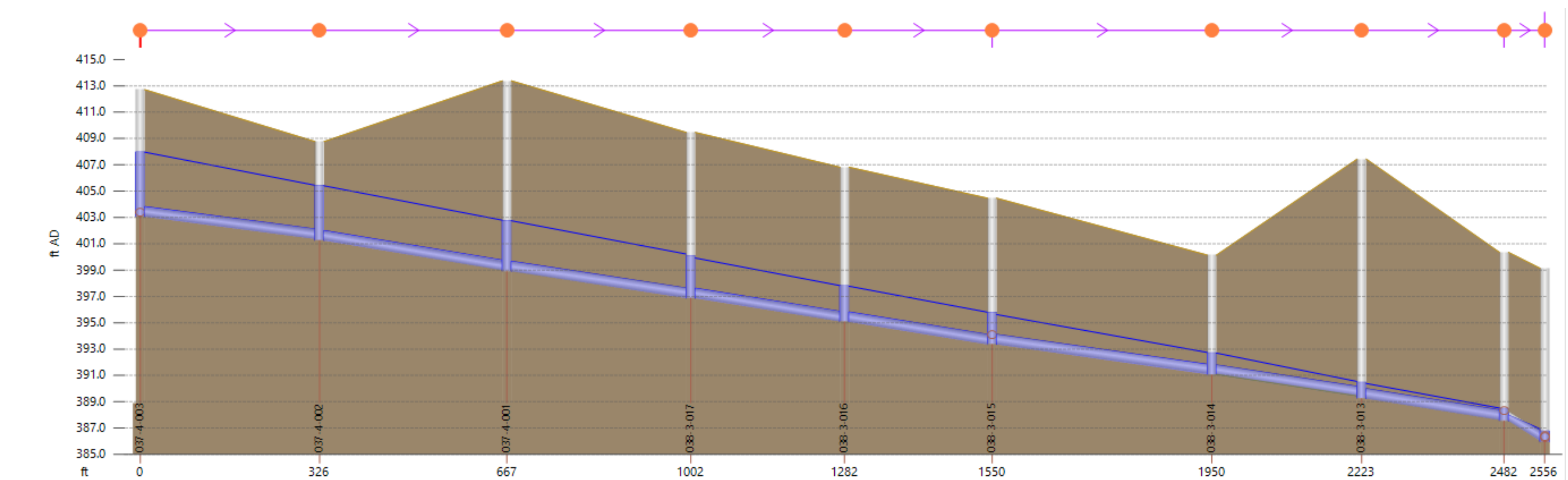
Once calibrated, the model was used to estimate the capacity of the sewer downstream of the pump station and will be used in evaluation of alternatives as the project moves into the PER phase.



Link	037-4-003.1	037-4-002.1	037-4-001.1	038-3-017.1	038-3-016.1	038-3-015.1	038-3-014.1	038-3-013.1	-
width (in)	9.5	9.5	9.5	9.5	9.5	9.5	9.5	9.5	9.5
pfc (MGD)	0.89	1.00	0.96	0.98	0.99	0.93	0.96	0.98	1.77
US flow (MGD)	1.0255	1.0170	1.0135	1.0108	1.0087	1.0019	1.0013	0.9997	0.9993
DS flow (MGD)	1.0202	1.0157	1.0125	1.0101	1.0082	1.0016	0.9998	0.9993	0.9993
DS velocity (ft/s)	4.081	3.940	3.024	3.765	3.327	3.738	4.052	4.082	3.187

Node	037-4-003	037-4-002	037-4-001	038-3-017	038-3-016	038-3-015	038-3-014	038-3-013	038-3-012
level (ft AD)	405.513	403.248	400.893	398.478	396.447	394.619	391.920	390.028	387.990
flood dep (ft)	-7.197	-5.522	-12.487	-10.982	-10.373	-9.841	-8.220	-17.412	-12.360
vol.lost (US Mgal)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Figure B-7: Model-projected peak HGL in sewer along Newton St during August 13th, 2021 power outage using received model data for lined pipes (9.5″ diameter, roughness = 0.013, little to no minor losses).



Link	037-4-003.1	037-4-002.1	037-4-001.1	038-3-017.1	038-3-016.1	038-3-015.1	038-3-014.1	038-3-013.1	-
width (in)	9.0	9.0	9.0	9.0	9.0	9.0	9.0	9.0	9.0
pfc (MGD)	0.84	0.94	0.90	0.92	0.92	0.87	0.87	0.92	1.66
US flow (MGD)	1.0183	1.0086	1.0010	0.9945	0.9895	0.9706	0.9696	0.9694	0.9694
DS flow (MGD)	1.0150	1.0061	0.9988	0.9930	0.9883	0.9701	0.9695	0.9694	0.9693
DS velocity (ft/s)	4.132	3.941	3.215	3.666	3.183	3.470	4.018	3.264	3.436

Node	037-4-003	037-4-002	037-4-001	038-3-017	038-3-016	038-3-015	038-3-014	038-3-013	038-3-012
level (ft AD)	407.979	405.425	402.773	400.070	397.803	395.778	392.721	390.463	388.497
flood dep (ft)	-4.731	-3.345	-10.607	-9.390	-9.017	-8.682	-7.419	-16.977	-11.853
vol.lost (US Mgal)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Figure B-8: Model-projected peak HGL in sewer along Newton St during August 13th, 2021 power outage using updated model data for lined pipes (9″ diameter, roughness = 0.012, additional minor losses)

Table D-3: Summary of peak depth results for depth-calibrated model runs against observed flood reports for August 13th, 2021 power outage event

House Address	Basement Elevation (ft)	HGL at Lateral (ft)	Depth of Flow Below Basement (ft)	Lateral Surcharged?	Projected Flooding?	Reported Flooding?
2419 Newton St	403.9	407.98	-4.06	Yes	Yes	Yes
2418 Newton St	407.1	407.98	-0.90	Yes	Yes	Yes
2417 Newton St	405.2	407.29	-2.13	Yes	Yes	Yes
2416 Newton St	407.8	407.41	0.34	Yes	No	Yes
2415 Newton St	407.5	406.83	0.67	Yes	No	No
2414 Newton St	407.8	406.61	1.14	Yes	No	No
2413 Newton St	402.3	405.78	-3.53	Yes	Yes	Yes
2412 Newton St	406.1	405.95	0.13	Yes	No	No
2411 Newton St	404.1	404.90	-0.82	Yes	Yes	No
2410 Newton St	406.1	404.85	1.23	Yes	No	No
2409 Newton St	403.5	404.03	-0.53	Yes	Yes	No
2408 Newton St	403.8	404.16	-0.41	Yes	Yes	No
2406 Newton St	408.1	403.23	4.85	Yes	No	No
9933 Newton St	403.8	403.26	0.49	Yes	No	No

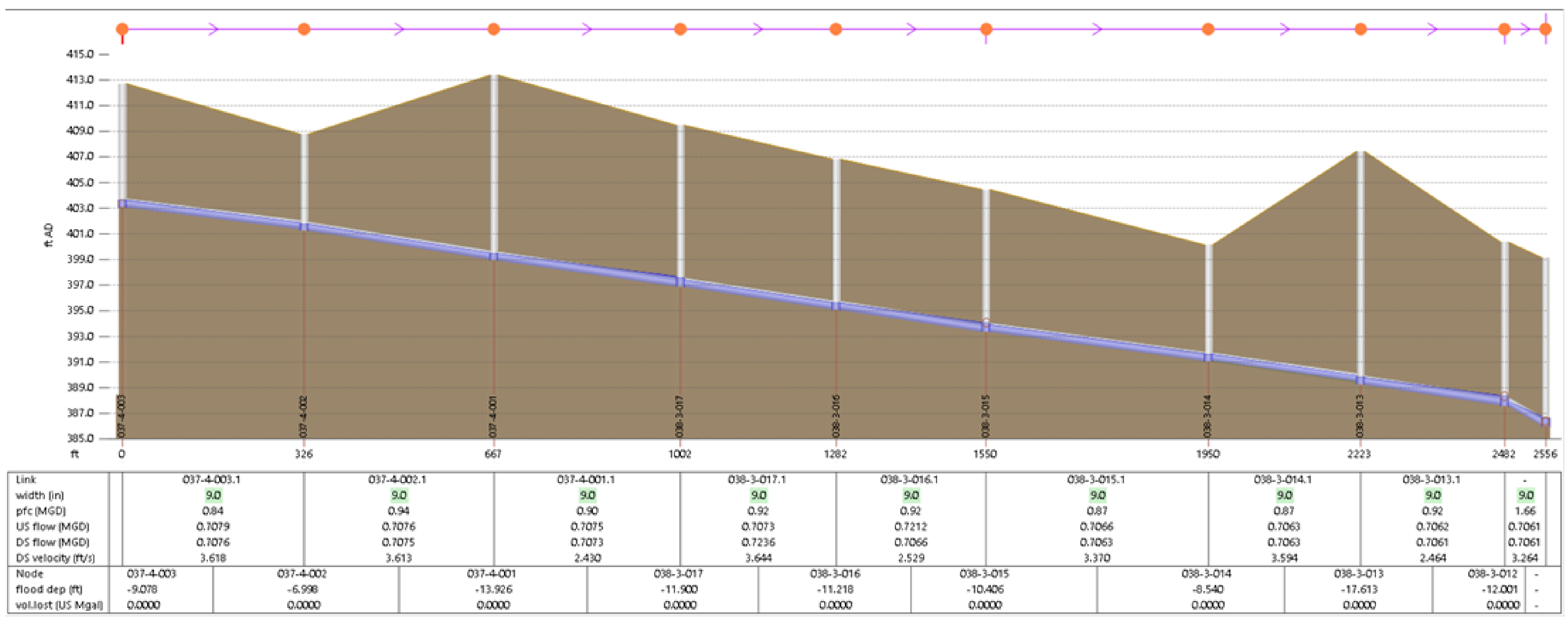
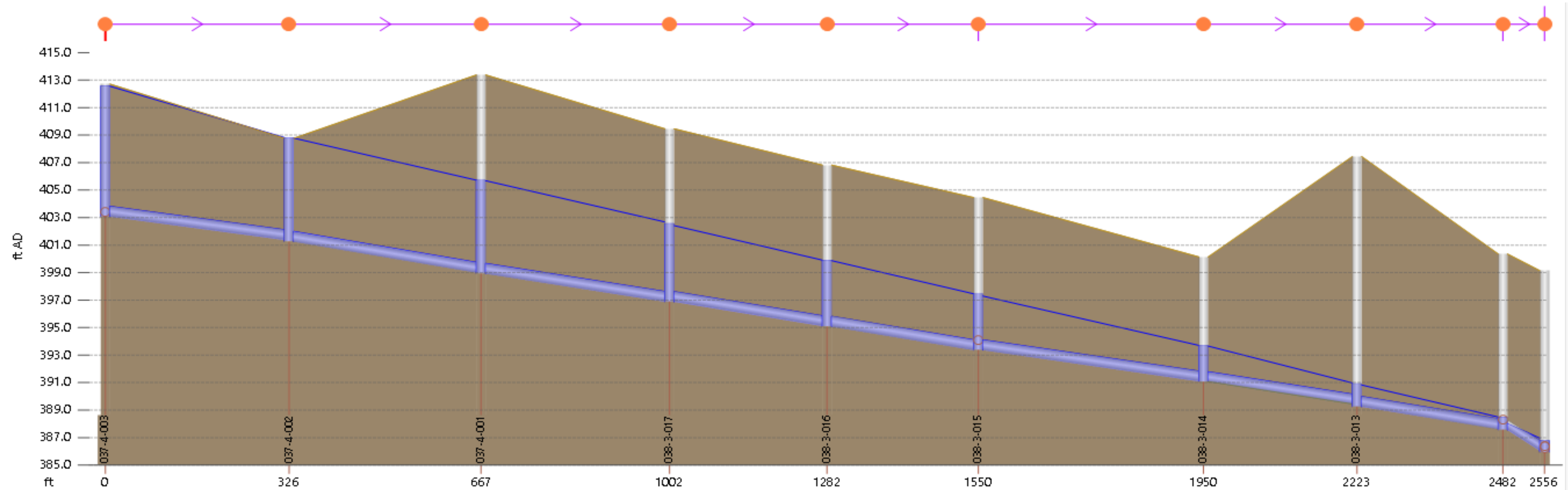


Figure B-9: Model-projected peak HGL in sewer along Newton St during September 1st, 2021 rain event using updated model data for lined pipes (9” diameter, roughness = 0.012, additional minor losses)



Link	037-4-003.1	037-4-002.1	037-4-001.1	038-3-017.1	038-3-016.1	038-3-015.1	038-3-014.1	038-3-013.1	-
width (in)	9.0	9.0	9.0	9.0	9.0	9.0	9.0	9.0	9.0
pfc (MGD)	0.77	0.87	0.83	0.85	0.85	0.81	0.80	0.85	1.53
US flow (MGD)	1.1300	1.0864	1.0625	1.0418	1.0259	0.9994	0.9994	0.9994	0.9994
DS flow (MGD)	1.1300	1.0786	1.0555	1.0371	1.0220	0.9994	0.9994	0.9994	0.9994
DS velocity (ft/s)	3.909	3.771	3.206	3.357	3.225	3.187	3.864	3.368	3.363

Node	037-4-003	037-4-002	037-4-001	038-3-017	038-3-016	038-3-015	038-3-014	038-3-013	038-3-012
level (ft AD)	412.576	408.770	405.706	402.554	399.890	397.437	393.682	390.899	388.473
flood dep (ft)	-0.134	-0.000	-7.674	-6.906	-6.930	-7.023	-6.458	-16.541	-11.877
vol.lost (US Mgal)	0.0000	0.1276	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Figure B-10: Model-projected peak HGL in sewer along Newton St during July 2nd, 2019 rain event using updated model data for lined pipes (9" diameter, roughness = 0.012, additional minor losses

Appendix C – Opinion of Probable Construction Cost

OPINION OF PROBABLE CONSTRUCTION COST

Project Name:	Lakevale Estates PS - DS-1- Pipe Replacement	Prepared By:	A. Nemati	
Project Owner:	Fairfax County DPWES	Checked By:	M. Osborne	
Project Location:	Fairfax County, Virginia			
Estimate Class:	Rough Order of Magnitude (-50% /+75%)			

Description	Qty	Unit	Unit Cost	Subtotal Cost
Alternative DS-1 - Pipeline Replacement				$ 2,508,404
Open Cut Excavation	7,705	CY	$ 48.50	$ 373,685
Excavation Support System	69,344	SF	$ 11.50	$ 797,456
Backfill & Compaction	7,705	CY	$ 35.00	$ 269,670
Erosion & Sediment Control	1	LS	$ 10,000.00	$ 10,000
Maintenance of Traffic	1	LS	$ 35,000.00	$ 35,000
Installation of 12-Inch RCP Sanitary Sewer	2,920	LF	$ 129.00	$ 376,680
New 4-Foot Diameter Sewer Manholes (w/coating)	10	EA	$ 4,910.00	$ 49,100
Installation of 4-inch Service Lateral Sewer	2,040	LF	$ 87.00	$ 177,480
Utilities Support/Relocation	1	LS	$ 80,000.00	$ 80,000
Temporary Bypass Pumping (duty/standby)	30	Days	$ 554.00	$ 16,620
Bypass Pumping Inspection (Fairfax County Section 01 51 00)	30	Days	$ 1,080.00	$ 32,400
Temporary HDPE Bypass Piping Installation	2,920	LF	$ 19.55	$ 57,086
Existing Manhole Demo, Flowable Fill & Backfill	120	VF	$ 83.00	$ 9,924
Post Construction CCTV and Acceptance Testing	2,920	LF	$ 7.60	$ 22,192
Bituminous Concrete, Milling & Paving	8,806	SY	$ 20.00	$ 176,111
Site Restoration & Landscaping	1	LS	$ 25,000.00	$ 25,000

Subtotal		**$ 2,508,404**
Mobilization/Demobilization	5%	$ 125,420
General Conditions	12%	$ 316,059
Prime Contractor OH&P	16%	$ 471,981
Bonds & Insurance	3.0%	$ 102,656
Escalation to Midpoint	7%	$ 246,716

Rough Order of Magnitude

Budget Estimate	**$ 3,771,237**
Low Range Estimate (-50%)	**$ 1,885,618**
High Range Estimate (+75%)	**$ 6,599,664**

OPINION OF PROBABLE CONSTRUCTION COST

Project Name:	Lakevale Estates PS - DS-2-Parallel Gravity Sewer	Prepared By:	A. Nemati
Project Owner:	Fairfax County DPWES	Checked By:	M. Osborne
Project Location:	Fairfax County, Virginia		
Estimate Class:	Rough Order of Magnitude (-50% /+75%)		

Description	Qty	Unit	Unit Cost	Subtotal Cost
Alternative DS-1 - Pipeline Replacement				$ 2,579,569
Shaft Excavation (Pilot Tube Microtunneling)	282	CY	$ 48.50	$ 13,664
Excavation Support System	3,381	SF	$ 11.50	$ 38,879
Backfill & Compaction	282	CY	$ 35.00	$ 9,861
Erosion & Sediment Control	1	LS	$ 5,000.00	$ 5,000
Maintenance of Traffic	1	LS	$ 20,000.00	$ 20,000
PTMT Parallel Sewer Gravity Line Sanitary Sewer including pipe	2,920	LF	$ 750.00	$ 2,190,000
New 4-Foot Diameter Sewer Manholes (w/coating)	10	EA	$ 4,910.00	$ 49,100
Installation of 4-inch Service Lateral Sewer	1,020	LF	$ 87.00	$ 88,740
Utilities Support/Relocation	1	LS	$ 40,000.00	$ 40,000
Temporary Bypass Pumping (duty/standby)	15	Days	$ 554.00	$ 8,310
Bypass Pumping Inspection (Fairfax County Section 01 51 00)	15	Days	$ 1,080.00	$ 16,200
Temporary HDPE Bypass Piping Installation	1	LS	$ 25,000.00	$ 25,000
Existing Manhole Demo, Flowable Fill & Backfill	-	VF	$ 83.00	$ -
Post Construction CCTV and Acceptance Testing	2,920	LF	$ 7.60	$ 22,192
Bituminous Concrete, Subbase, Base Course, Top Course & Coate	636	SY	$ 67.00	$ 42,624
Site Restoration & Landscaping	1	LS	$ 10,000.00	$ 10,000

Subtotal		**$ 2,579,569**
Mobilization/Demobilization	5%	$ 128,978
General Conditions	12%	$ 325,026
Prime Contractor OH&P	16%	$ 485,372
Bonds & Insurance	3.0%	$ 105,568
Escalation to Midpoint	7%	$ 253,716

Rough Order of Magnitude

Budget Estimate	**$ 3,878,229**
Low Range Estimate (- 50%)	**$ 1,939,114**
High Range Estimate (+75%)	**$ 6,786,900**

OPINION OF PROBABLE CONSTRUCTION COST

Project Name:	Lakevale Estates PS - DS-3-Parallel Force Main	Prepared By:	A. Nemati
Project Owner:	Fairfax County DPWES	Checked By:	M. Osborne
Project Location:	Fairfax County, Virginia		
Estimate Class:	Rough Order of Magnitude (-50% /+75%)		

Description	Qty	Unit	Unit Cost	Subtotal Cost
Alternative DS-1 - Pipeline Replacement				$ 2,363,657
Shaft Excavation (Pilot Tube Microtunneling)	282	CY	$ 48.50	$ 13,664
Excavation Support System	3,381	SF	$ 11.50	$ 38,879
Backfill & Compaction	282	CY	$ 35.00	$ 9,861
Erosion & Sediment Control	1	LS	$ 5,000.00	$ 5,000
Maintenance of Traffic	1	LS	$ 10,000.00	$ 10,000
PTMT Parallel Sewer Gravity Line Sanitary Sewer including pipe	2,920	LF	$ 750.00	$ 2,190,000
New 4-Foot Diameter Sewer Manholes (w/coating)	-	EA	$ 4,910.00	$ -
Installation of 4-inch Service Lateral Sewer	-	LF	$ 87.00	$ -
Utilities Support/Relocation	-	LS	$ 5,000.00	$ -
Temporary Bypass Pumping (duty/standby)	7	Days	$ 554.00	$ 3,878
Bypass Pumping Inspection (Fairfax County Section 01 51 00)	7	Days	$ 1,080.00	$ 7,560
Temporary HDPE Bypass Piping Installation	1	LS	$ 10,000.00	$ 10,000
Existing Manhole Demo, Flowable Fill & Backfill	-	VF	$ 83.00	$ -
Post Construction CCTV and Acceptance Testing	2,920	LF	$ 7.60	$ 22,192
Bituminous Concrete, Subbase, Base Course, Top Course & Coate	636	SY	$ 67.00	$ 42,624
Site Restoration & Landscaping	1	LS	$ 10,000.00	$ 10,000

Subtotal		**$ 2,363,657**
Mobilization/Demobilization	5%	$ 118,183
General Conditions	12%	$ 297,821
Prime Contractor OH&P	16%	$ 444,746
Bonds & Insurance	3.0%	$ 96,732
Escalation to Midpoint	7%	$ 232,480

Rough Order of Magnitude

Budget Estimate	**$ 3,553,618**
Low Range Estimate (-50%)	**$ 1,776,809**
High Range Estimate (+75%)	**$ 6,218,831**