Dewberry

Dewberry Engineers Inc.
4805 Lake Brook Drive, Suite 200
Glen Allen, VA 23060-9278

804.290.7957
804.290.7928 fax
www.dewberry.com

January 6, 2022

Mr. Rob Murphy
Deputy Town Manager
Town of Colonial Beach
315 Douglas Avenue
Colonial Beach, Virginia 22443

RE: Town of Colonial Beach
Overall Sanitary Sewer System I&I Evaluation Letter Report

Dear Mr. Murphy:

Dewberry Engineering Inc. (Dewberry), under contract with the Town of Colonial Beach (Town), has completed an inflow and infiltration (I&I) analysis of the overall sanitary sewer system in the Town to identify the areas indicating the highest potential for I&I. This letter report outlines the analysis, findings, and recommended next steps.

Introduction

The Town has been addressing sewer defects in their sanitary sewer system for many years to combat the increased influent flows to their wastewater treatment plant (WWTP) during rain events. The most recent Sanitary Sewer Improvements Phase 3 project focused improvements in the 3rd Street Pump Station drainage area and a small portion of the Bancroft and Lafayette Pump Station drainage area. Reduction in I&I was witnessed with the completion of that project, particularly in the 3rd Street Pump Station basin, through observation of a reduction in pump station run time and reduction of sand observed at the pump station. While this indicated a successful targeting of I&I, the remainder of the aging sewer system continued to deteriorate, causing I&I to continue to be a priority for the Town to address.

In recent years, the influent flow at the WWTP has exceeded its permitted capacity during some significant rain events and there have been repeated instances of a manhole overflowing. Given an overall I&I analysis of the system has not been completed since 2004, the Town and Dewberry agreed it was worth completing a similar analysis and letter report to identify the areas with substantial I&I. This would allow the Town to determine where to focus and prioritize their I&I reduction efforts to get the “most bang for their buck” when completing sewer system repairs.

Dewberry was contracted to complete an I&I analysis for the overall sanitary sewer system in order to identify the pump station drainage areas that demonstrate the highest potential for I&I and to provide recommended next steps to allow identification of the improvements needed to continue the efforts to reduce system I&I. The evaluation and recommendations are presented in this report in the following sections:

- Introduction
- Sanitary Sewer System Background
- Data Collection and Processing
- Inflow and Infiltration Analysis
- Conclusions and Recommended Next Steps

Sanitary Sewer System Background

The Town's existing sanitary sewer collection and conveyance system within its service area consists of approximately 150,000 linear feet (LF) of gravity sanitary sewer, approximately five hundred (500) manhole (MH) structures, twenty (20) pump stations, and approximately 49,500 LF of force main. The Town boundary, gravity sewer, manholes, pump stations, force mains, and more are shown in **Figure 1**.

The Town's WWTP is rated for two (2) million gallons per day (MGD); however, during wet weather events, the WWTP has received as much as four (4) MGD of influent flow. This excessive I&I can be an environmental issue given the WWTP's resulting inundation and inability to adequately treat flows before discharging, as well as can be an economic issue impacting the Town's ability to expand the current sanitary sewer service area.

The overall sanitary sewer service area was previously divided into four (4) drainage areas, including the Riverside Meadow / Point Bluff Drainage Area, the Classic Shores Drainage Area, the Central Drainage Area, and the Point Drainage Area. The Monroe Point townhome development has since been built and has been added to the Central Drainage Area. The Shellfield Shores pump station is also a new addition to the analysis and has been added to the Riverside Meadow / Point Bluff Drainage Area. Each drainage area includes one or more pump stations, which further divides each drainage area into sub-drainage areas. Each sub-drainage area includes all of the gravity piping that flows to individual pump stations. The Town's drainage areas and sub-drainage areas for each pump station are shown in **Figure 2**. From these drainage areas, all wastewater is eventually pumped to the WWTP for treatment. Effluent from the WWTP is then released into Monroe Bay.

The Town has dealt with the impacts of I&I over many years. In the late 1990s and early 2000s, approximately 12,800 LF of sewer main, 370 vertical feet (VF) of manholes, and twelve (12) pump stations were rehabilitated or replaced, based on defects identified by the Town and by a CCTV study completed in 1992 by Patton, Harris, Rust & Associates. The sewer main and manhole improvements were focused in the Central Drainage Area. Only marginal improvements to I&I reduction were observed after these improvements.

In 2004, Dewberry recommended and completed an I&I analysis for the overall sanitary sewer system (See **Appendix A**) to better target the areas contributing substantial I&I. Dewberry identified two (2) drainage areas (the Classic Shores Drainage Area and the Point Drainage Area) and specific pump stations within those drainage areas (PS5 – 3rd Street and PS13 – Bancroft and Lafayette) that demonstrated the highest potential for I&I. The Town completed further investigation of these areas with flow monitoring and CCTV inspection in the PS5 – 3rd Street sub-drainage area and smoke testing in the PS13 – Bancroft and Lafayette sub-drainage area to identify necessary improvements. These improvements were constructed as part of the Sanitary Sewer Improvements Phase 3 project in 2013, which included trenchless rehabilitation of sewer main and manholes, replacement of sewer main by pipe bursting, replacement of sewer main by open-cut excavation, and installation of new manholes. The project resulted in sewer improvements to approximately 24,000 LF of sewer main ranging in size from 6-inch diameter to 18-inch diameter, rehabilitation of 73 existing manholes, and installation of 19 new manholes. Noticeable reduction to I&I was observed upon the completion of the Phase 3 improvements; however, those improvements were primarily focused in the 3rd Street Pump Station sub-drainage area.

In recent years, the continued realization of influent flows exceeding the WWTP's permitted capacity during wet weather events and repeat instances of a manhole overflowing in the Central Drainage Basin has reinforced the Town's need to prioritize efforts to reduce I&I. The Town and Dewberry agreed an overall I&I analysis was needed again to identify where the Town should focus its next sewer improvement efforts.

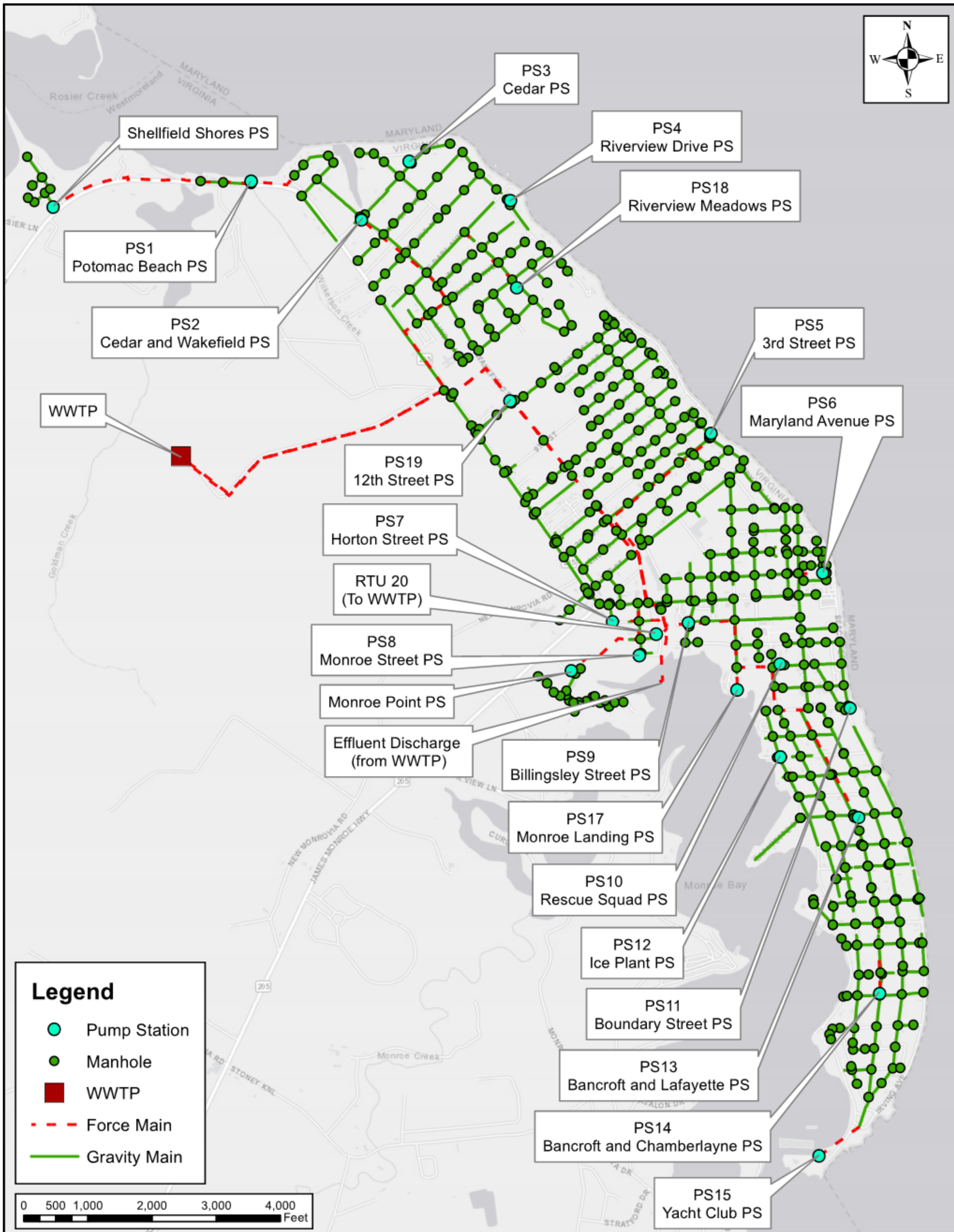

Figure 1: Town of Colonial Beach Existing Sanitary Sewer System

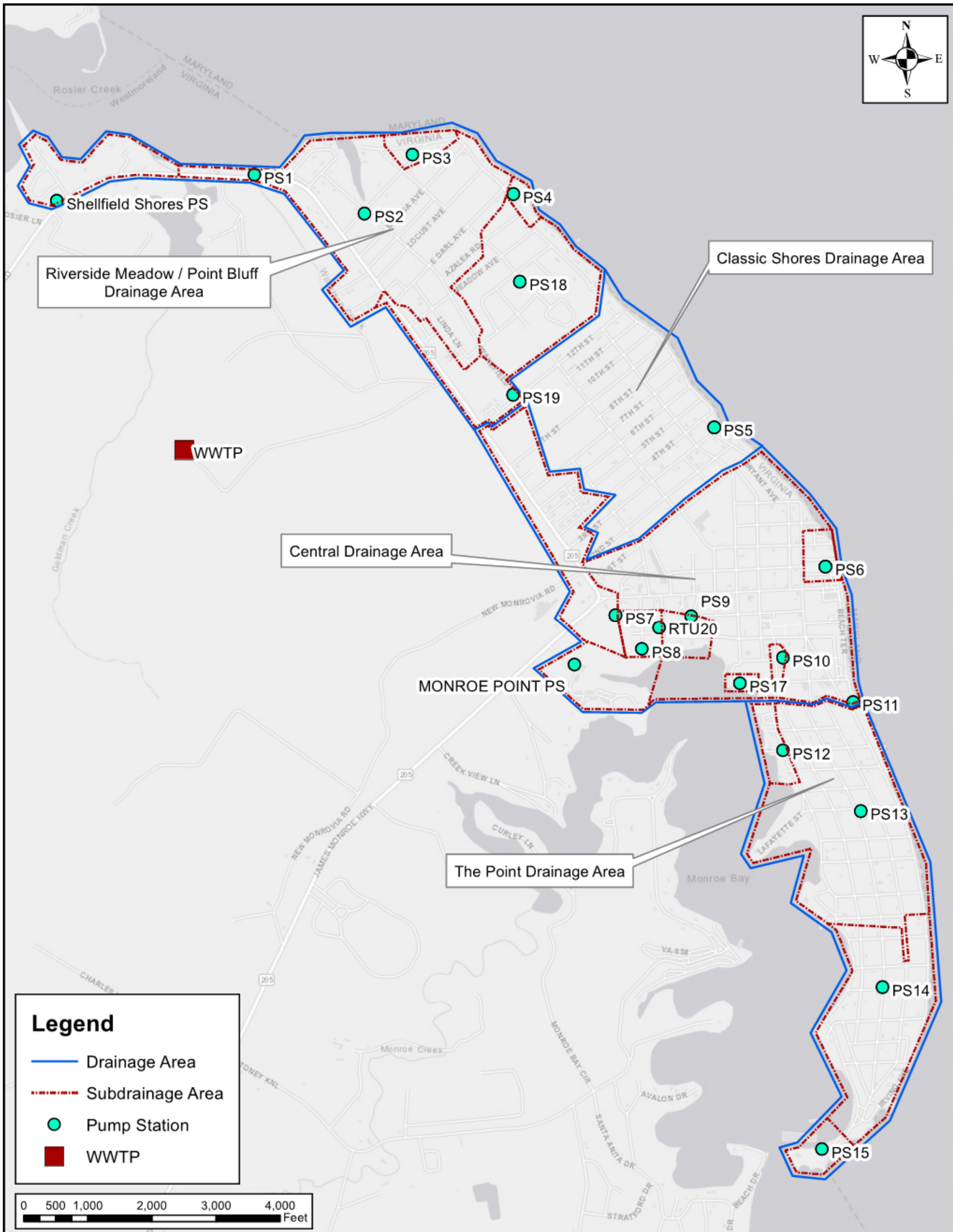

Figure 2: Town of Colonial Beach Drainage Areas

Data Collection and Processing

For the 2004 I&I analysis, the Town provided a schematic layout of the sewer system on 1979 tax maps and instructed Dewberry to assume 8-inch diameter for all sewer main. In addition, flow monitoring activities were completed by utilizing the continuous pump run-time meters on each pump combined with the known flow rate for the pumps at each station. Run-time meters were recorded each day by Town personnel at the same time for 30 days and compared with rain data for the same 30-day period. In developing the scope of this new analysis, it was assumed similar data would be provided by the Town. The Town provided the following information to allow Dewberry to complete the new I&I analysis (See **Appendix B** for data collected):

- GIS mapping of the Town's sanitary sewer system
- Pump station drawdown test data to identify current flow rates of the pumps
- Daily pump station run-times (excluding weekends)
- Influent flow data for the WWTP
- Rain gauge data collected at the WWTP

With the GIS mapping, the Town provided known sizes of the gravity sewer system piping and confirmed force main connection points in the gravity sewer system for each pump station. Where data was missing, Dewberry requested further input and clarification from Town personnel. **Table 1** summarizes the gravity sewer infrastructure within each pump station sub-drainage area.

Table 1: Gravity Sewer Infrastructure Summary

Pump Station Drainage Basin	Gravity Pipe Length (feet)								# of MHs
	4-inch Dia.	6-inch Dia.	8-inch Dia.	10-inch Dia.	12-inch Dia.	15-inch Dia.	16-inch Dia.	18-inch Dia.	
Shellfield Shores	—	—	1,531	—	—	—	—	—	8
PS1: Potomac Beach	—	—	824	—	—	—	—	—	3
PS2: Cedar & Wakefield	—	—	17,228	2,117	—	—	—	—	49
PS3: Cedar	—	—	1,048	—	—	—	—	—	3
PS4: Riverview Dr	—	—	604	—	—	—	—	—	2
PS5: 3rd St	—	2,042	20,271	1,051	543	261	—	2,100	97
PS6: Maryland Ave	—	—	1,690	—	—	—	—	—	14
PS7: Horton St	57	—	3,856	4,325	—	—	—	—	28
PS8: Monroe St	—	—	1,053	344	—	—	—	—	3
PS9: Billingsley St	—	—	551	—	—	—	—	—	4
PS10: Rescue Squad	—	—	650	—	—	—	—	—	4
PS11: Boundary St	135	656	24,995	2,993	50	—	837	1,355	93
PS12: Ice Plant	—	129	1,494	—	—	—	—	—	5
PS13: Bancroft & Laf.	—	—	22,434	44	3,674	398	—	711	88
PS14: Bancroft & Cham.	—	—	12,444	247	—	—	—	—	40
PS15: Yacht Club	*No Gravity Main Data*								0
PS17: Monroe Landing	*No Gravity Main Data*								0
PS18 - Riverview Meadows (Stratford)	—	—	8,970	—	—	—	—	—	27
PS19: 12th St	—	—	787	2,527	—	—	—	40	10
Monroe Point	—	—	2,697	—	—	—	—	—	22
Total:	**192**	**2,827**	**123,125**	**13,646**	**4,267**	**659**	**837**	**4,207**	**500**

Table 2 summarizes where each PS discharges, along with its associated force main length. All of the data summarized in Table 1 and Table 2 is from GIS shapefiles provided by the Town.

Table 2: Force Main Infrastructure Summary

Pump Station	Force Main Discharge Location	Force Main Length (feet)
Shellfield Shores	PS1 - Potomac Beach	3,237
PS1: Potomac Beach	PS2 - Cedar and Wakefield	748
PS2: Cedar and Wakefield	PS19 - 12th Street	3,917
PS3: Cedar	PS2 - Cedar and Wakefield	273
PS4: Riverview Drive	PS2 - Cedar and Wakefield	275
PS5: 3rd Street	RTU 20	4,338
PS6: Maryland Avenue	PS11- Boundary Street	375
PS7: Horton Street	RTU 20	913
PS8: Monroe Street	PS11- Boundary Street	822
PS9: Billingsley Street	PS11- Boundary Street	357
PS10: Rescue Squad	PS11- Boundary Street	69
PS11: Boundary Street	RTU 20	4,210
PS12: Ice Plant	PS13 - Bancroft and Lafayette	317
PS13: Bancroft and Lafayette	RTU 20 (manifolds into PS11 – Boundary Street force main)	1,886
PS14: Bancroft and Chamberlayne	PS13 - Bancroft and Lafayette	1,127
PS15: Yacht Club	PS14 - Bancroft and Chamberlayne	774
PS17: Monroe Landing	PS11- Boundary Street	349
PS18: Riverview Meadows (Stratford)	PS2 - Cedar and Wakefield	1,229
PS19: 12th Street	WWTP (manifolds into RTU 20 force main)	25
Monroe Point	RTU 20	1,557
RTU 20	WWTP	10,933
Final Effluent (from WWTP)	Monroe Bay	11,760
	Total:	**49,491**

During the kickoff meeting for this I&I analysis, it was discussed that review of sewer system flows over a longer period would be beneficial to observe I&I patterns during summer, as well as off-season. Pump run-times, WWTP flows, and rain gauge data were provided by the Town for two time periods: February through March 2021 and May through June 2021.

In reviewing each pump station's raw pump run-time data and the pump flow rate data provided by the Town, a number of data gaps and inconsistencies were identified that required clarifications and/or assumptions to proceed with the analysis. The following is a summary of data gaps and the clarifications or assumptions discussed with the Town:

- Each pump station's run-time data was provided in an excel file. Per discussion with Town personnel, run-time numbers for each pump represented the number of hours a pump operated.
- Each pump's run-time meter data was manually recorded and manually transferred into an excel file for Dewberry's use. There were several instances where the run-time numbers did not make sense because a succeeding number or set of numbers would be less than the preceding numbers, which resulted in calculating a negative amount of time for a pump's run-time. Sometimes it appeared a number was shortened, such as instead of writing 1459.3, 1459.9,

1460.3 the recorder wrote 459.3, 459.9, 460.3. Other times, it appeared numbers may have been inverted (ex – recorder wrote 547.2 instead of 457.2). Therefore, modifications to the run-time data were necessary to proceed with calculations for the analysis. Modified data was discussed with Town personnel for approval and is included in **Appendix B**.

- Pump run-time data was recorded on weekdays only, so each pump's run-time over the weekend had to be averaged over Saturday and Sunday when calculating daily flow. There were also other dates where pump run-times were not recorded, so total run-time had to be averaged over the period of days without data.
- The pumping rate for PS4 – Riverview Drive was verbally provided by Town personnel as 38 gallons per minute (GPM) for each pump (70 GPM for both pumps). Run-times were not provided for Pump #1, as the pump was offline for the four months analyzed.
- For PS7 – Horton Street, PS10 – Rescue Squad, and PS17 – Monroe Landing, the pumping rate for Pump #2 was assumed to match the pumping rate provided for Pump #1.
- For most of the pump stations, Pump #1 and Pump #2 alternate operating; however, the Town clarified that at some of the pump stations, Pump #2 only runs for high flow conditions. This is evident in the data when Pump #2 operates less than Pump #1; for example - Pump #2 for PS7 – Horton Street.
- Pump #2 for PS14 – Bancroft and Chamberlayne was out of service in February and March 2021, so no run-times were provided for that time period. Furthermore, it was assumed the following pumps were also out of service for repairs due to long periods of zero (0) run-times in the data:
    - Pump #2 for PS10 – Rescue Squad for February and March 2021
    - Pump #2 for PS17 – Monroe Landing from February 2021 through May 24, 2021.
    - Pump #2 for PS18 – Riverview Meadows from May 3, 2021, through May 25, 2021.

In addition to the data requested from the Town, Dewberry researched and collected additional precipitation data for the area, as well as tidal data from the National Oceanic and Atmospheric Administration (NOAA). Precipitation data was collected for Stratford, VA, and used as a backcheck on the WWTP rain gauge data. Tide elevations were collected for Lewisetta, VA, and Dahlgren, VA, to see if there was any correlation between tide elevation and increased I&I in the sewer system. This data is included in **Appendix B**.

Tide elevations were also compared to manhole rim elevations. Rim elevations were not known for all manholes, so contour data was used to extrapolate manhole rim elevations. Based on the comparison of water elevations to manhole rim elevations, ten (10) manholes were identified with elevations less than or near the maximum tide levels realized in Lewisetta, VA and Dahlgren, VA as shown in **Figure 3**. Two (2) of the manholes are in the sub-drainage area for PS1 – Potomac Beach.

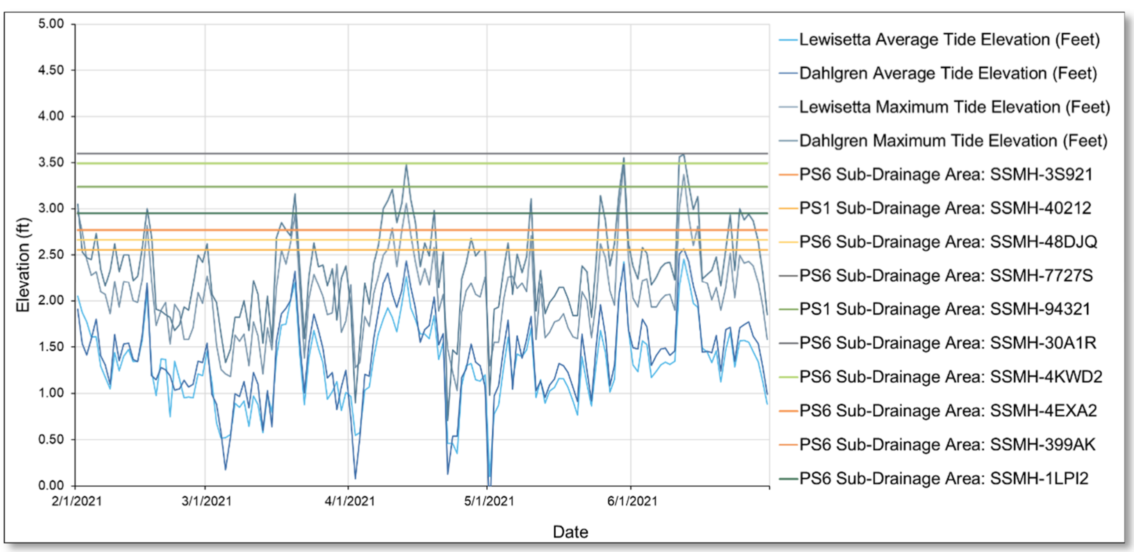

Figure 3: Tide Elevations vs. Manhole Rim Elevations

Eight (8) of the manholes are in the sub-drainage area for PS6 – Maryland Avenue, as shown in **Figure 4**.

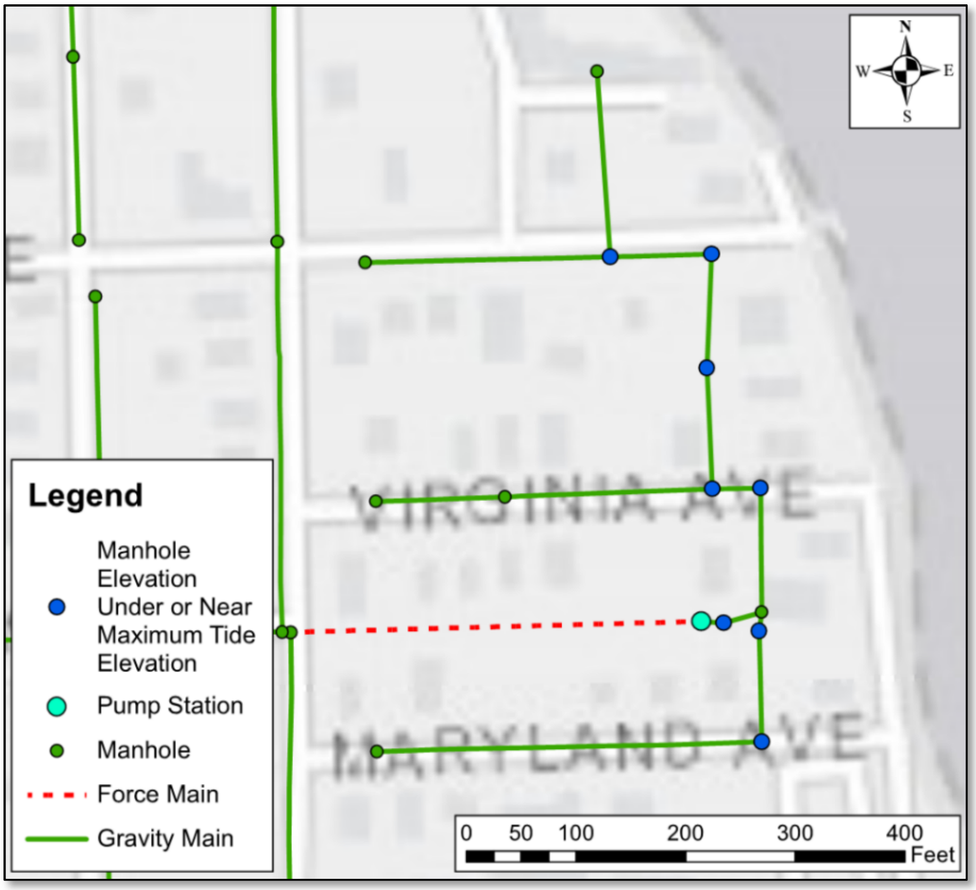

Figure 4: Manholes in PS6 Drainage Area with Elevations Less Than or Near Maximum Tide Elevations

Inflow and Infiltration Analysis

The pump station run-times and pump flow rates were used to calculate total daily flow in gallons (GPD) for each pump station for the months of February, March, May, and June of 2021. Daily flow values for each pump station can be found in **Appendix C**.

Flow data from each pump station was reviewed against precipitation data to identify dry periods and gather flow from several dry weather days. These days were averaged to obtain a baseline for dry weather flow. Separate dry weather dates and wet weather dates were chosen for the February/March time period and the May/June time period. For each two-month time period, three (3) dry weather days were averaged to establish an average dry day flow, and one wet weather day was identified with a significant rain event to establish wet weather flow. Wet weather flow was compared to average dry day flow to determine the increase in flow due to I&I.

The dry day flows averaged for the February/March 2021 data were 3/10, 3/11, and 3/12. While a large rain event occurred over 2/12 (4 inches) and 2/13 (3.25 inches), the actual impact to pump run-times was not observed because run-times were not recorded over weekends. As the data was analyzed, it was realized this lack of daily data on weekends buffered the actual reaction of the system by averaging the total run-times over 2 or 3 days (depending on the time the run-time data was recorded). Therefore, a wet weather event chosen to calculate I&I had to be chosen in the middle of the week. The wet day flow identified for the February/March 2021 data was 3/24 (1.875 inches). The dry day flows averaged for the May/June 2021 data were 5/19, 5/20, and 5/21. The wet day flow identified for the May/June 2021 data was 6/10 (2.5 inches).

Flows for each pump station were plotted against the WWTP rain gauge data to evaluate flows during both wet and dry periods. As I&I was calculated for February/March, it was observed that even though there was no rain for over a week leading up to the dry days used for the average dry weather flow, the system flows were still being impacted by significant rainfall during February 2021 (the Town recorded rainfall on 16 out of 28 days in February 2021). A review of the graphs created for each pump station illustrates that the sewer system in March was still being impacted by February rainfall, meaning I&I couldn't be calculated because a true baseline for dry weather flow did not exist in the February/March time period. This issue and the fact that multiple pumps were out of service eliminated the February/March data from the I&I analysis, and the remaining evaluation was completed with the May/June 2021 data.

The pump station graphs of Daily Flow vs Precipitation for May/June 2021 were analyzed to determine the amount of I&I that was affecting each station, as well as determine if the increase in flow was caused by Inflow or Infiltration. Inflow can be caused by a cross-connection between a storm sewer and a sanitary sewer or a broken pipe exposed to surface runoff. Inflow will typically be indicated by an immediate increase of flow during a rain event. Infiltration can be caused by offset joints in the gravity sewer, cracks in the pipe, or deteriorated service connections that allow groundwater to seep into the pipe over a length of time. Infiltration will typically be indicated by a gradual increase of flow and a slow decrease of flow over a longer period of time.

Figure 5, 6, 7, 8, and 9 show graphs of daily flow versus precipitation for five of the pump station sub-drainage areas. In each graph, the flow's response to precipitation indicates evidence of infiltration (shown by the slow incline of flow and/or slow decline of flow after a rain event) and inflow (shown by the immediate spike inflow). Graphs for all the pump stations can be found in **Appendix C**.

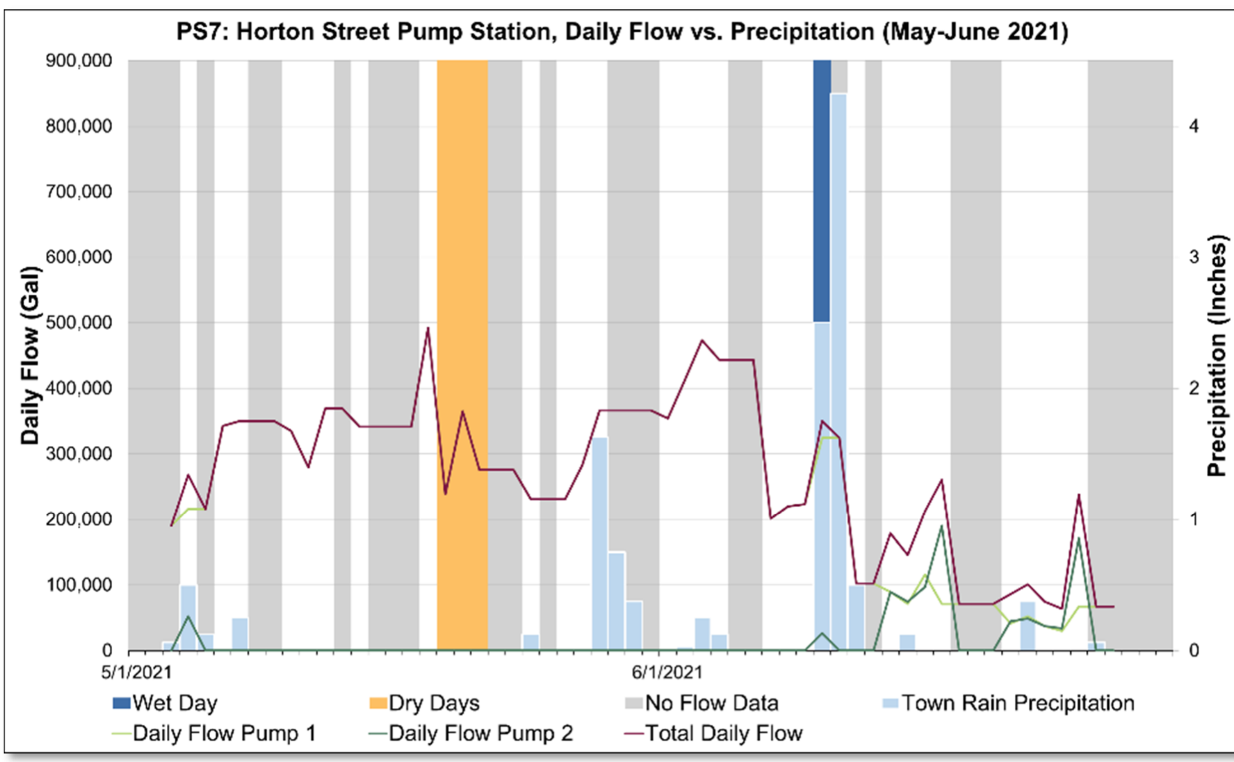

Figure 5: PS7 - Evidence of Infiltration and Inflow

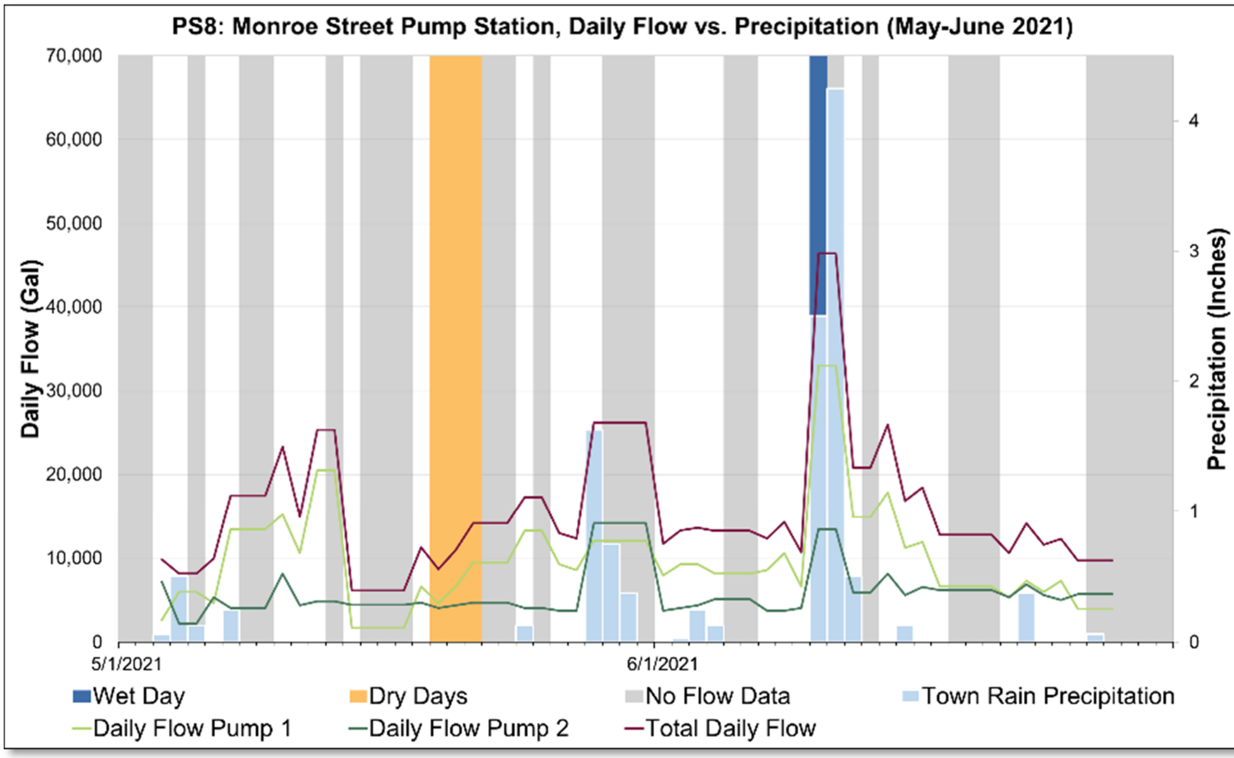

Figure 6: PS8 - Evidence of Infiltration and Inflow

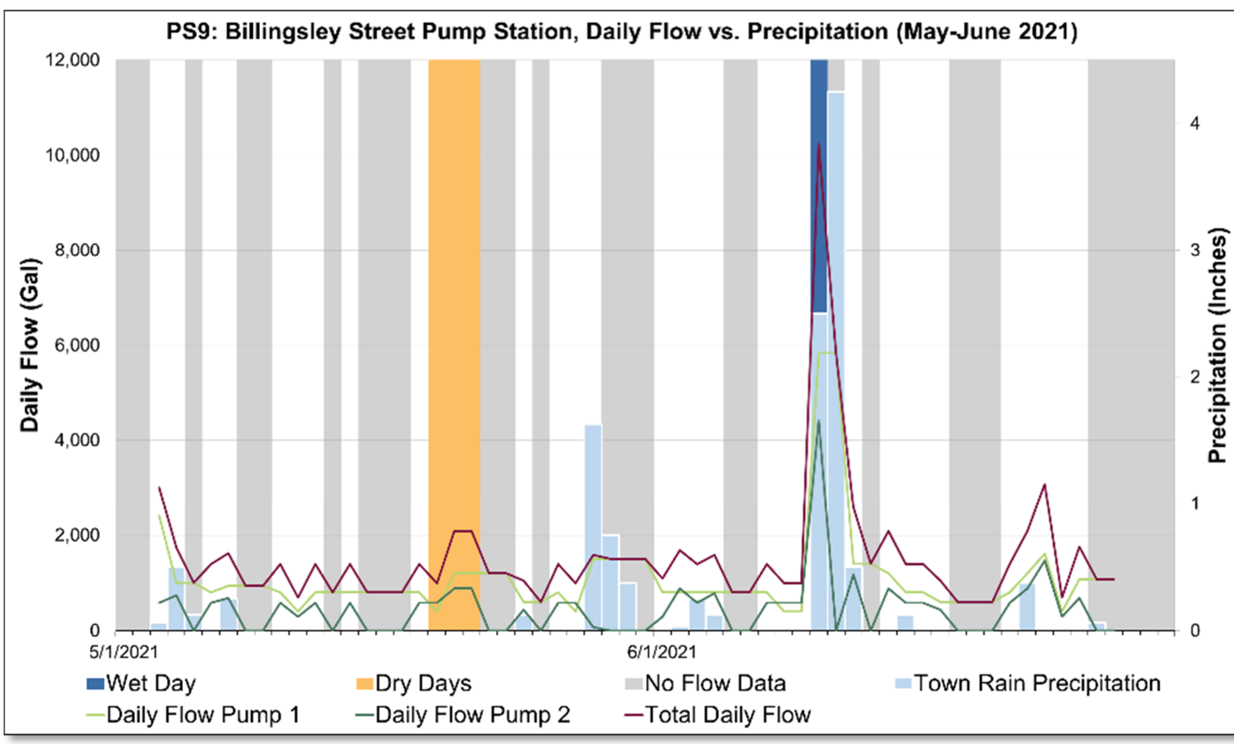

Figure 7: PS9 - Evidence of Infiltration and Inflow

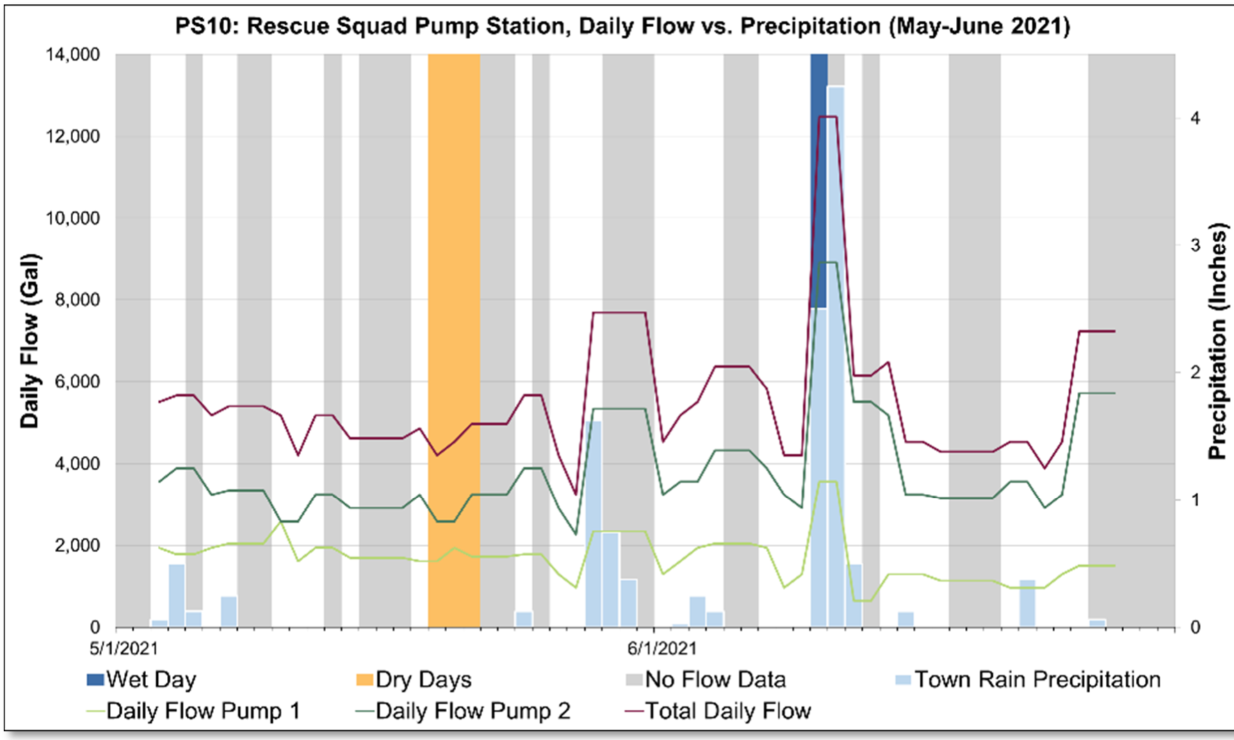

Figure 8: PS10 - Evidence of Infiltration and Inflow

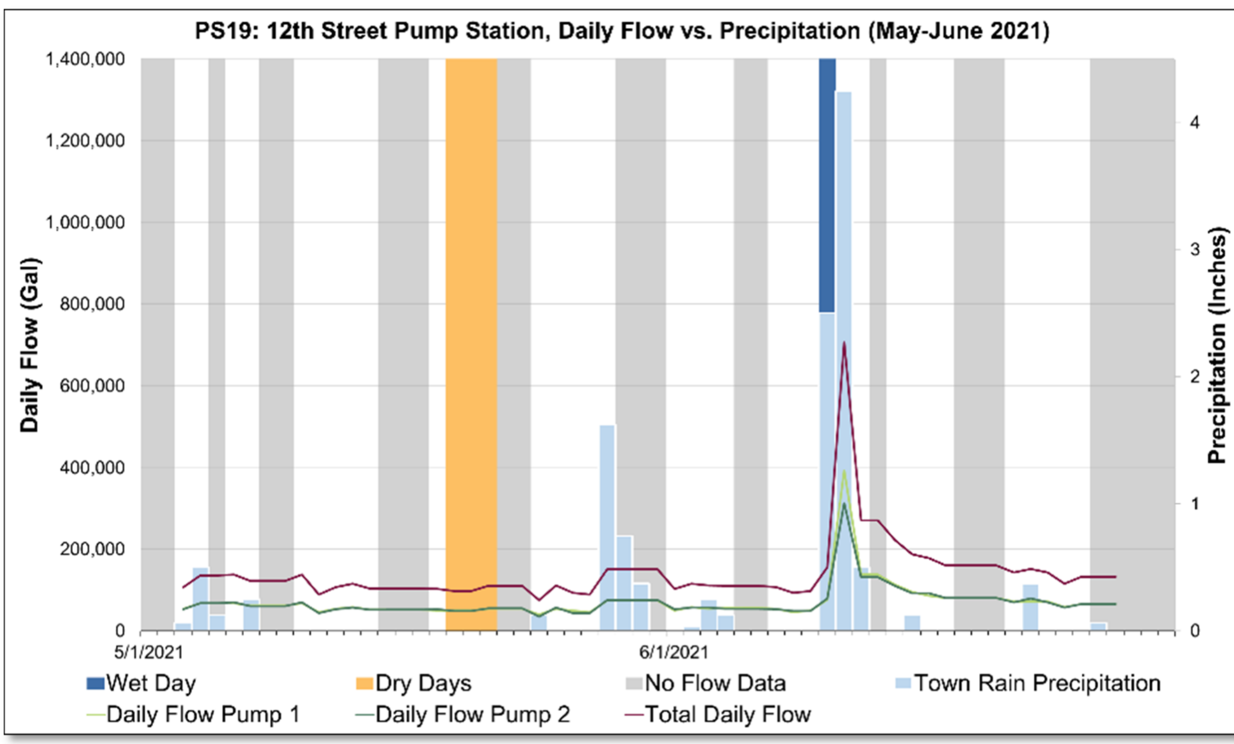

Figure 9: PS19 - Evidence of Infiltration and Inflow

When calculating the I&I for May/June, it was noticed the wet weather flow was less than the average dry day flow for PS3 – Cedar. This may be because the June 10, 2021 pump run time recording occurred before the rain began. A modification was made to the wet weather flow to assume an average of the daily flow calculated on 6/10 (3,468 GPD) and 6/11 (10,386 GPD) to capture the impact of the rain event on the flow.

Once I&I flow was calculated for each pump station sub-drainage area, the total pipe length by pipe diameter was quantified for each sub-drainage area and a weighted factor of gallons per day per diameter-inch-mile was developed in order to compare each area consistently. The weighted factor allows the smallest sub-drainage area to be compared to the largest sub-drainage area based on an equivalency that normalizes the length of the pipe and the diameter of the pipe, thereby allowing an equal comparison of the amount of I&I experienced by individual sub-drainage areas during wet weather events. **Table 3** shows the result of this comparison. It is important to note that when one pump station was discharging into the collection system for another pump station, the calculated I&I for the upstream pump station was subtracted from the calculated I&I for the downstream pump station to allow for an adjusted and accurate comparison of the I&I being experienced in each sub-drainage area.

Table 3: Pump Station I&I Comparison

Pump Station	Average Dry Day Flow (GPD)	Wet Day Flow (GPD)	Calculated I&I (GPD)	Pipe Length (LF)	Weighted Units (dia-in-mi)	Comparison Factor (GPD / dia-in-mi)	Adjusted Comparison Factor (GPD / dia-in-mi)
Shellfield Shores PS	2,500	5,000	2,500	1,531	2.32	1,078	
PS1	14,274	20,232	5,958	824	1.25	4,770	2,769*
PS2	128,471	156,918	28,447	19,344	30.11	945	606**
PS3	5,385	6,927	1,542	1,048	1.59	970	
PS4	1,393	1,596	203	604	0.91	222	
PS5	63,358	77,580	14,222	26,267	44.16	322	
PS6	5,100	10,476	5,376	1,690	2.56	2,100	
PS7	293,112	350,244	57,132	8,239	14.08	**4,058**	
PS8	11,261	46,416	35,155	1,397	2.25	**15,650**	
PS9	1,722	10,239	8,517	551	0.82	**10,360**	
PS10	4,572	12,474	7,902	650	0.98	**8,024**	
PS11	248,269	464,850	216,581	31,021	51.66	4,193	3,053***
PS12	1,809	4,654	2,844	1,623	2.41	1,180	
PS13	93,933	264,900	170,967	27,262	45.98	3,718	3,312****
PS14	8,676	24,486	15,810	12,690	19.32	818	812*****
PS15	960	1,080.0	120	*--No Data--*	--	--	--
PS17	1,633	3,570	1,937	*--No Data--*	--	--	--
PS18	14,467	17,400	2,933	8,970	13.59	216	
PS19	102,231	156,282	54,051	3,353	6.11	8,841	**4,188******
Monroe Point PS	2,920	5,346	2,426	2,697	4.09	594	

** - Calculated I/I for Shellfield Shores pump station was subtracted from Calculated I/I for pump station 1.*
*** - Calculated I/I for pump stations 1, 3, 4, and 18 was subtracted from Calculated I/I for pump station 2.*
**** - Calculated I/I for pump stations 6, 8, 9, 10, and 17 was subtracted from Calculated I/I for pump station 11.*
***** - Calculated I/I for pump stations 12 and 14 was subtracted from Calculated I/I for pump station 13.*
****** - Calculated I/I for pump station 15 was subtracted from Calculated I/I for pump station 14.*
******* - Calculated I/I for pump stations 2 was subtracted from Calculated I/I for pump station 19.*

Based on the I&I comparison factors in Table 4, it appears the sub-drainage areas of PS8 – Monroe Street, PS9 – Billingsley Street, and PS10 – Rescue Squad (all in the Central Drainage Area) demonstrate the highest potential for I&I. Given the overall pipe length is relatively small in each of those sub-drainage areas, the sub-drainage areas with the next two highest I&I comparison factors, PS 7 – Horton Street and PS19 – 12th Street, are also added to the list of areas with the highest potential for I&I.

Conclusions and Recommended Next Steps

It's encouraging to see that the I&I comparison factor for PS5 – 3rd Street is relatively low when compared to the other pump station sub-drainage areas. This shows the Phase 3 improvements completed in 2013 decreased the I&I in that drainage area.

Based on the I&I analysis included in this report, the five (5) pump station sub-drainage areas indicating the highest potential for I&I are PS7 – Horton Street, PS8 – Monroe Street, PS9 – Billingsley Street, PS10 – Rescue Squad, and PS19 – 12th Street. These sub-drainage areas were the largest contributors to the I&I experienced during the time period analyzed (May/June 2021). It is recommended the Town focus its sewer improvement efforts in these areas.

Upon acceptance of this letter report by the Town, Dewberry recommends the following next steps for the Town in proceeding with the identification of improvements and acquisition of funding for a project:

1. Complete field activities in the five (5) sub-drainage areas to investigate sewer defects.
    a. Complete smoke testing to identify probable areas of inflow. Smoke testing is effective in identifying cross-connections between storm sewer and sanitary sewer, connections of roof drains to the sanitary sewer, or broken pipe open to surface water run-off.
    b. Complete cleaning and CCTV inspection to identify probable areas of infiltration. Cleaning and CCTV inspection of sewer main is effective to identifying defects in the sewer.
    c. Complete manhole inspections to identify probable areas of inflow and/or infiltration.

2. Proceed with the completion of a Preliminary Engineering Report (PER) in order to:
    a. Summarize the field activities, identify sewer defects, and recommend improvements.
    b. Assist in the acquisition of grant/loan money from funding agencies, including USDA-RD.

3. Proceed with the completion of a funding source application to be submitted for grant/loan money.

4. Upon acquisition of grant/loan money and completion of PER, prepare contract documents for a construction project(s).

5. Advertise for bid and award construction contract for the design project(s).

6. Begin and complete construction.

A preliminary schedule with approximate durations for the recommended next steps is provided in **Table 4**.

Table 4: Preliminary Schedule

Description	Approximate Time
Field Activities (Smoke Testing, CCTV, MH Inspection)	45-60 days
Preliminary Engineering Report	90-120 days
Funding Source Application	90-120 days
Prepare Contract Documents for Construction Project(s)	180-210 days
Advertise for Bid and Award Contract	30-45 days
Construction	300-360 days

Upon the completion of the preliminary analysis of the data to identify the five (5) pump station sub-drainage areas indicating the highest potential for I&I, the Town requested an estimate of the costs to complete the repairs in the five (5) areas to assist with a funding request. It is important to note that without completing the field investigation to determine actual sewer defects, the ability to determine the level of repair and associated costs is highly speculative. Dewberry reviewed the recent Phase III Sewer Improvements breakdown of rehabilitation versus replacement to estimate 50% sewer rehabilitation and 50% sewer replacement for the future improvements. However, given the relatively small amount of pipe in the basins for PS#8, PS#9, and PS#10, we assumed 100% replacement for those basins.

Table 5 outlines a budget project cost estimate for the assumed future improvements. The quantities are estimates based on GIS mapping and can therefore change significantly when verified in the field with survey, therefore a contingency of 35% was added. It should also be noted that there is high volatility in the construction market currently due to the global pandemic and construction prices are extremely elevated and fluctuating greatly due to supply chain issues and labor shortages.

Table 5: Budgetary Project Cost Estimate

Quantity	Unit	Description	Unit Price ($)	Cost ($)
1	LS	Traffic Control (Max. 5%)	$ 150,000	$ 150,000
4604	LF	Remove Ex. Sewer and Install 8" Gravity Sewer	$ 105	$ 483,420
3770	LF	Remove Ex. Sewer and Install 10" Gravity Sewer	$ 115	$ 433,550
40	LF	Remove Ex. Sewer and Install 18" Gravity Sewer	$ 135	$ 5,400
116	EA	Replace Ex. Lateral and Install Cleanout	$ 4,000	$ 464,000
5776	LF	Cleaning and Pre/Post CCTV Inspection	$ 15	$ 86,640
2350	LF	Rehab 8" Gravity Sewer w/ CIPP	$ 45	$ 105,750
3426	LF	Rehab 10" Gravity Sewer w/ CIPP	$ 50	$ 171,300
72	EA	Reinstate Lateral w/ Watertight Seal	$ 1,000	$ 72,000
49	EA	Rehabilitate 48" Diameter Manhole	$ 5,000	$ 245,000
49	EA	Replace Manhole Frame and Cover	$ 500	$ 24,500
4674	SY	Asphalt Repair	$ 100	$ 467,444
360	DAY	Bypass Pumping	$ 2,500	$ 900,000
1	LS	Mobilization (Max. 5%)	$ 150,000	$ 150,000
			Subtotal	**$ 3,759,004**
			Contingency (35%)	$ 1,315,652
			Construction Subtotal	**$ 5,074,656**
			Engineering (15%)	$ 761,198
			Total Project Cost	**$ 5,835,854**

More refined costs can be estimated during the PER phase, as field investigation should be completed as part of the PER.

Should you have any questions or concerns regarding this letter report, please feel free to contact us.

Sincerely,
Dewberry Engineers, Inc.

Heather Campbell, PE, PACP, MACP, LACP, ITCP
Senior Project Manager

Richard Kincheloe, PE
Principal Engineer

Appendix A: 2004 I&I Analysis Letter Report
Appendix B: Data Collected
Appendix C: Analysis Tables and Graphs
Appendix D: 11/29/21 Letter with Anticipated Sewer Repair Costs