



Town of Wellington Collection System Masterplan

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Town of Wellington Collection System Masterplan

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Executive Summary

The Town of Wellington (Town) contracted Jacobs Engineering Group (Jacobs) to perform a master plan study of its wastewater collection system, using growth pattern and population projections furnished through the Town's Comprehensive Plan. These data were used to determine whether there are any significant existing capacity-related constraints, and to determine the capacities and approximate corridors for future interceptor sewers to convey Buildout flow rates for the Town's projected Buildout population of 35,500. This facilitates decisions on interceptor segments that may need to be strategically constructed within new developments as part of the initial infrastructure. The study also examined how well the system fares in the nearer term using the Town's current Year 2040 population projection of approximately 25,000. This is a high-level master plan that provides planning-level flow rate information, which can be used by the Town when performing subsequent detailed design of the collection system.

Jacobs developed an InfoSWMM hydraulic model of the existing collection system based on Town data for trunk sewers and manholes, used flow metering provided to the Town by Hach to validate the model, and then examined how the Town's growth projections impact the collection system through to Buildout. Jacobs used the Town's DRAFT Comprehensive Plan to develop recommendations for improvement projects and a general layout for potential future interceptors.

On the basis of the growth locations and growth rates provided by the Town, the main backbone of the existing collection system can accommodate growth through approximately Year 2040, aside from the existing View Pointe Lift Station which will require improvements in the next few years. The new interceptor sewers will be required to service areas outside the current service area, and will need to be extended to the wastewater treatment facility by approximately Year 2040, as the existing trunk sewers will not have sufficient capacity to convey all of the flows. In most cases the interceptor sewers will work in conjunction with the existing trunk sewers. A summary of the recommendations is provided in Table E-1.

Table E-1. Capital Improvement Plan Recommendations for Buildout

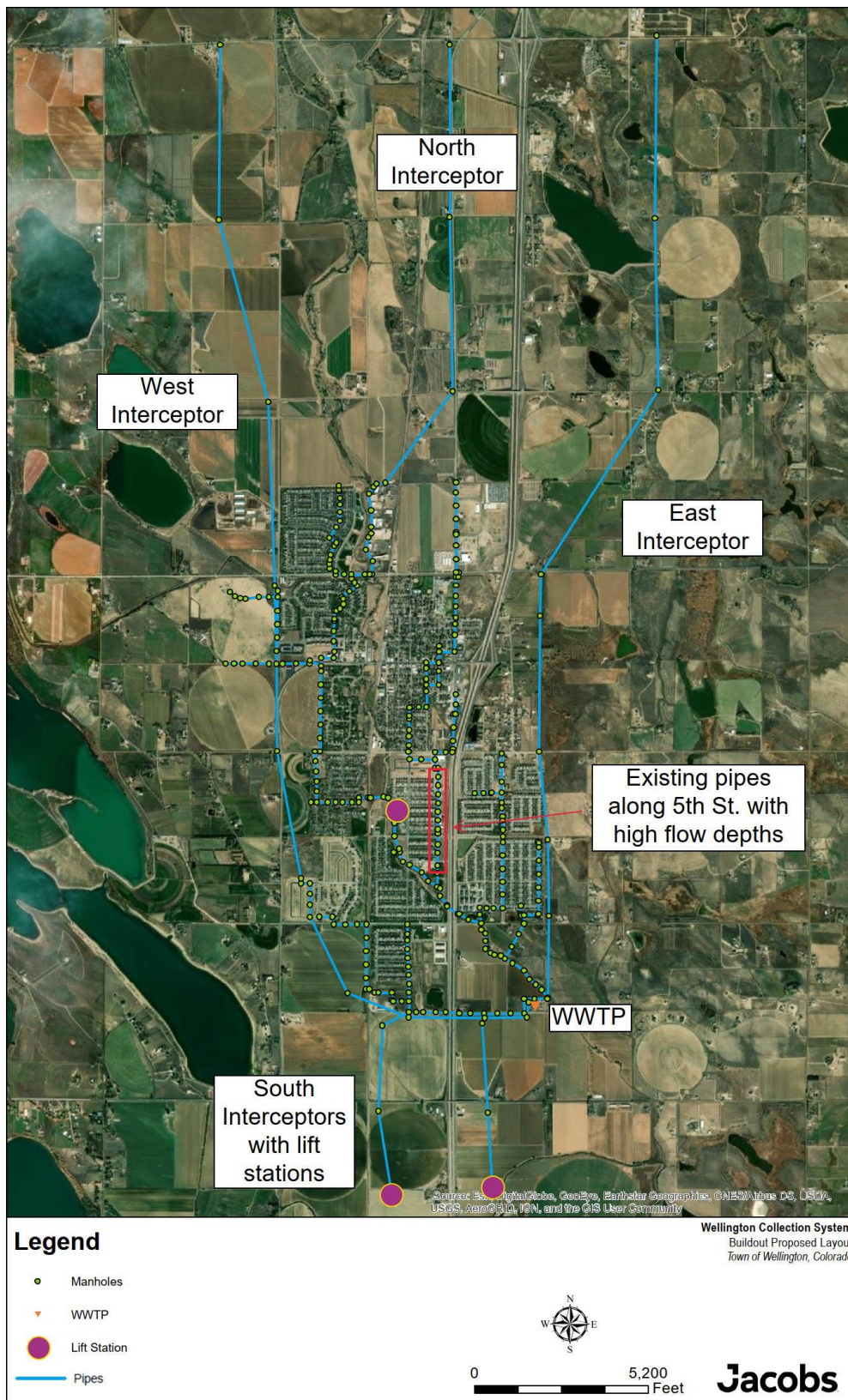
Project	Description	Estimated Cost [#]	Time Frame
East Interceptor	An interceptor along the east side of I-25, which ties into the main interceptor upstream of the plant	\$18M*	2035-2040
North Interceptor	An interceptor east of Boxelder Creek and west of I-25, which ties into the existing collection system in the northwest	\$8M*	As development occurs
West Interceptor	An interceptor to the west of Boxelder Creek which utilizes the 30" line in Sage Meadows before paralleling the existing west interceptor underneath the railroad tracks and discharges directly upstream of the wastewater treatment plant (WWTP)	\$28M*	2035-2040
South Interceptors	Two lines: one west of I-25 which discharges into the existing collection system, and one east of I-25 which discharges into the new West Interceptor	\$9M*	As development occurs

Project	Description	Estimated Cost [#]	Time Frame
South Interceptor Lift Stations	Lift stations to pump flow from the south end of the Buildout area	\$0.5M*	As development occurs
5th Street Upsizing	Improvements to the existing section of pipe south of the downtown area which may experience excessive depths of flow with future growth	\$3.5M	By about 2040 if monitoring confirms need and concerns are not mitigated by peak flow management
View Pointe Lift Station Improvements	Replacement of the existing lift station equipment to address safety, maintenance, and capacity concerns	\$0.7M	Within 2 years
Rehabilitation and/or replacement	Updates to address aging infrastructure and I/I if flow monitoring indicates the need	\$100,000 per year allowance	As needed
Flow Monitoring	Confirmation of project needs and planning	\$50,000-\$100,000 per monitoring season	Every 5 years, 3-month duration
2040 Temporary Improvements	Temporary pipes to convey new flows to the existing collection system prior to the need for a new interceptor are included in the interceptors to which they pertain.		As development occurs
n/a = not applicable, I/I = infiltration and inflow * Interceptor sewer and South Interceptor lift station project costs may be funded in whole or part by development, depending upon Town policy. Costs for interceptors reflect estimate for total cost of installed interceptor. # Estimated costs are in 2021 dollars.			

Most of the interceptor projects are expected to occur in areas as they are developed. The requirements for these lines will be to upsize a run of pipe through the new development, so that buildout capacity flows can be conveyed without needing to relay the interceptor through a developed area in the future.

Jacobs developed two future scenarios in the hydraulic model. The first is for full Buildout based on a population of 35,488 people from the DRAFT Comprehensive Plan. The second scenario is for a population of 25,000 people, which the Town expects to reach around Year 2040. A proposed general layout for Buildout is provided in Figure E-1.

Figure E-1. Buildout Proposed Layout



1. Introduction and Background

The Town of Wellington (Town) is experiencing continued growth which will require improvements to the wastewater collection system. In November of 2019, Jacobs began the process of creating a hydraulic computer model of the existing collection system using geographic information system (GIS) data provided by the Town and Innovyze's InfoSWMM Suite v14.7 software. This report documents the development of the existing conditions model, the field data, model validation, Buildout model development, Capital Improvement Plan (CIP) recommendations for the next 20 years, and model limitations and refinement.

The purpose of this planning-level model is to provide guidance on the future CIP needs and anticipated timing of those project needs. This guidance helps determine the following:

- Capacity available within the existing system, and thus where the existing system is adequate or deficient for projected future flows.
- Approximate location recommendations for future interceptors (based on topography and projected capacity needs).
- Lift station requirements for future Buildout areas.
- Existing View Pointe Lift Station upgrade needs.
- Capacity recommendations for future interceptors.
- Tie-in locations for future interceptors.

A primary goal of this master plan is to determine the highest potential flows based on worst-case conditions and determine sewerage conveyance needs based on those conditions. Therefore, Buildout conditions were based on peak hour scenarios, diurnal patterns based on the worst-case day of the week, and a concurrent clean-in-place wash of the Nanofiltration/Reverse Osmosis Plant (Nano/RO Plant) which discharges into the collection system.

This report is developed as part of the overall Wastewater System Masterplan for the Town of Wellington that includes a masterplan for the Town's Wastewater Treatment Plan that was finalized on July 13, 2021; references are made to that document in this report.

2. Model Development

2.1 Model Network

The model network was built with GIS data provided by the Town on November 1, 2019, which included manhole locations, depths, rim elevations (on NAVD88 datum), pipe lengths, pipe materials, diameters, and invert elevations. The model includes primarily pipes with diameters greater than 8 inches, although a few 8-inch pipes were included. The model does not include the myriad of small sewers within established developments, but instead focuses on only those sewers through developments that serve as the main backbone and are subject to increased flow resulting from the projected growth.

Additions to the original base model network include:

- An 8-inch diameter extension of the interceptor upstream of M-L5-24 was added for a half mile per the pipe shapefile provided by the Town in April 2020.
- Additional sections were added in April of 2021 based on updated GIS shapefiles provided by the Town: 8-inch and 10-inch pipes for the upcoming Poudre School District (PSD) high school; a section of 15-inch pipe along 6th St., north of CR64; the section of 30-inch pipe in Sage Meadows; and the 12-inch and 15-inch pipes between Sage Meadows and Columbine Estates.

A summary of the infrastructure included in the InfoSWMM model is provided in Table 2-1 and the modeled network is shown in Figure 2-1. Note that “manholes with inflow” on Figure 2-1 represent locations where flow is added into the model network, not where there is “inflow” from a rainfall event.

Table 2-1. Summary of Existing Infrastructure in InfoSWMM Model

Category	Value
Number of Pipes	289
Number of Manholes	289
Materials	PVC (281 pipes) VC (8 pipes)
Diameter (in)	8 (6 pipes) 10 (29 pipes) 12 (115 pipes) 15 (73 pipes) 18 (61 pipes) 30 (5 pipes)
Lift Station	2 pumps (1 duty + 1 standby) 1 storage node
Outfall	1 node at WWTP

PVC = polyvinylchloride, VC = vitrified clay

2.1.1 Manholes

Modeled manholes for the existing system were given a manhole identifier (MH ID) which matches the MH ID in the GIS layer, allowing for quick data comparison and interchange between the model and GIS. For Buildout, manhole IDs for future areas were named based on their general location in the DRAFT Comprehensive Plan. Manholes representing the PSD high school flows were named PSD# with general sequential numbering used. Manhole data included are the invert elevation, the rim elevation, and the max depth, which is typically the depth from the rim to the lowest pipe invert.

2.1.2 Pipes

The pipes were modeled using the upstream and downstream invert elevations, pipe diameter, roughness (Manning's n), and upstream and downstream manhole IDs. A Manning's n value of 0.014, which is rougher than what would typically be used for PVC (0.009-0.011), was used to provide slight conservatism in flow depths and to provide for minor losses in manholes.

2.1.3 Pump

A pump was added to the model to represent the existing View Pointe Lift Station. It was modeled as an "ideal pump," so that the outflow is equal to the inflow. This provides a suitable model representation for a variable frequency drive (VFD) pump and ensures all flow entering the lift station is routed on to the downstream gravity sewer segments. It is anticipated that the actual pumping capacity at this lift station will need to be increased in the future.

2.1.4 Outfall

An outfall was modeled at the location of the WWTP with a 30-inch diameter pipe with invert at 5120.33 ft. This is based on a planning-level inlet pipe diameter and the proposed elevation of the inlet to the future headworks influent pump station, which is currently under design. The existing conditions model runs were checked with the existing inlet configuration of an 18-inch pipe and inlet elevation of 5122.83 ft, per the "Town of Wellington Wastewater Treatment Plant Expansion" 2016 Record Drawings, as a sensitivity analysis. Modeling the existing facilities in this manner was determined not to have a significant impact on the validation results. Therefore, for simplicity, the future diameter and inlet elevation were used for all model runs. There are no other special structures in the system.

2.2 Flows

This section describes the process for determining flows in the existing collection system to apply to the InfoSWMM model for validation. Initially, the wastewater flows were based on the number of residential houses throughout the collection system extents using aerial imagery. A conservative flow per dwelling unit (DU) was initially assumed throughout the system. Then, flow per capita was adjusted to match the average influent flow at the wastewater treatment plant (WWTP). The Town provided additional GIS information in April of 2020 which showed the locations where laterals connected to the main trunk lines. This information was used to select model locations for wastewater flows to the collection system. Changes to the assumptions were driven by the DRAFT Comprehensive Plan provided by the Town in January of 2021. The Town assumes a dwelling unit size of 3.2 people per DU. The flow per capita was then calculated using the WWTP influent flow as described in 2.2.4 below.

2.2.1 Wastewater Collection System Flow Monitoring

Validation of a wastewater collection system model requires flow data over an extended period that captures dry and wet weather conditions. In 2020, the Town contracted Hach to install flow meters at seven locations in the collection system to determine flow in strategic locations along the various trunk sewers. The flow data from the meters were used by Jacobs for the period between April and July 2020 for model validation. The Town also installed three rain gauges during the monitoring period: at the WWTP, the View Pointe Lift Station, and the Buffalo Creek Booster Station. The locations of the flow meters and rain gauges are shown in Figure 2-2.

Figure 2-1. Model of Existing Collection System

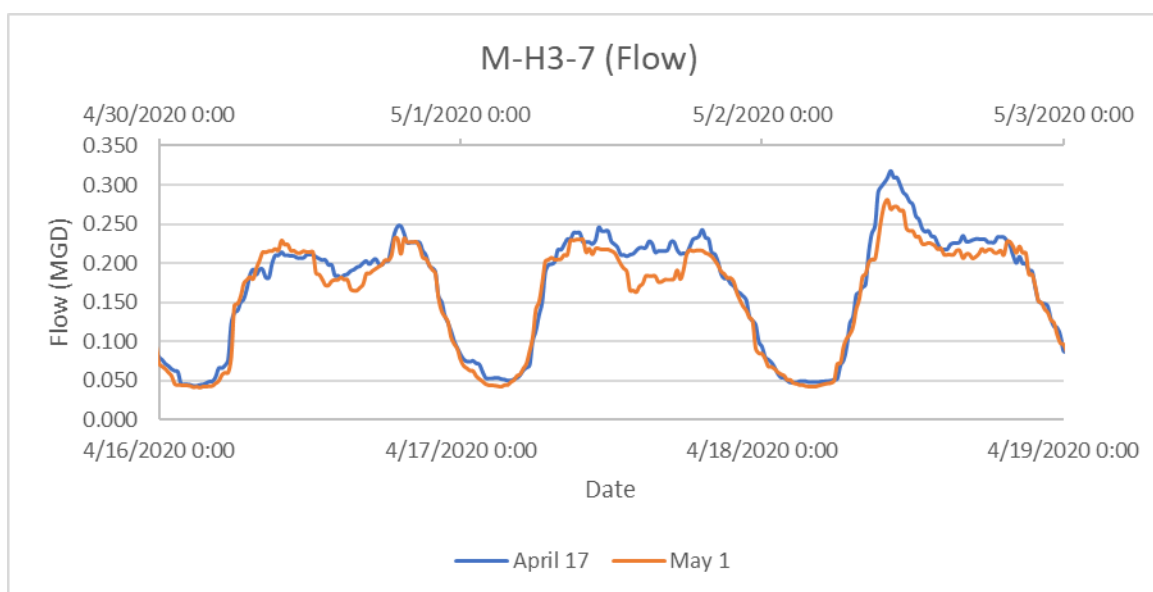


Figure 2-2. Flow Meter and Rain Gauge Locations



Based on long-term historical data from the Colorado State University's main campus weather station in nearby Fort Collins¹, rainfall during the first six months of 2020 was very similar to the long-term average monthly values, although the latter half of the year was below average rainfall. There were several rain events during the months of April and May. In general, the collection system is relatively new, and most of the system is constructed of PVC pipe. Based on a general comparison of the flows during wet and dry periods, the impact of rain on the collection system was observed to be minimal, and no infiltration or inflow was included in the model used for validation. Meter data for two groups of dates for each flow meter are provided in Figures 2-3 to 2-9 to show the similarity in flow data between a period with significant rainfall and one without rain. On Friday, April 17, 2020, there was 0.42 inches of rain recorded at the lift station. The rain began at 9:00 AM, prior to the normal peak flow of the day. On Friday, May 1, 2020, there was a trivial 0.01 inches of rain, with no rain for 3 days prior. Flow meter data for these two dates, along with one day prior to and one day following each date of interest, are plotted below for each of the seven flow meters. They show very little wet weather impact on the metered flows.

Figure 2-3. M-H3-7 Rain (Apr 17) vs Dry (May 1) Meter Flow



¹ ccc.atmos.colostate.edu/~autowx/

Figure 2-4. M-H5-29 Rain (Apr 17) vs Dry (May 1) Meter Flow

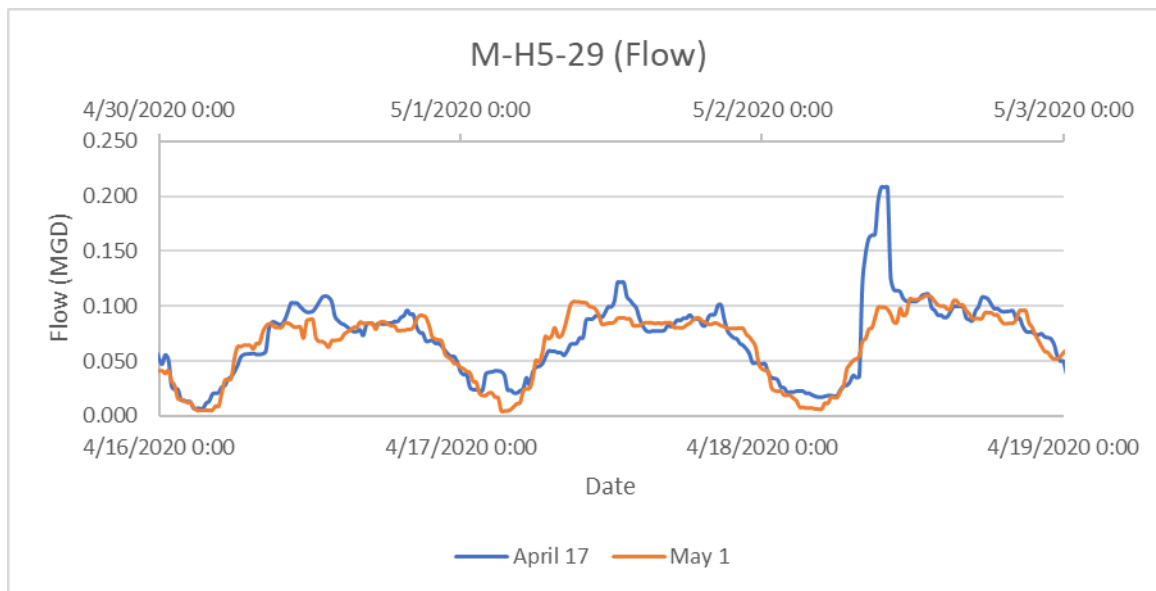


Figure 2-5. M-K5-6 Rain (Apr 17) vs Dry (May 1) Meter Flow

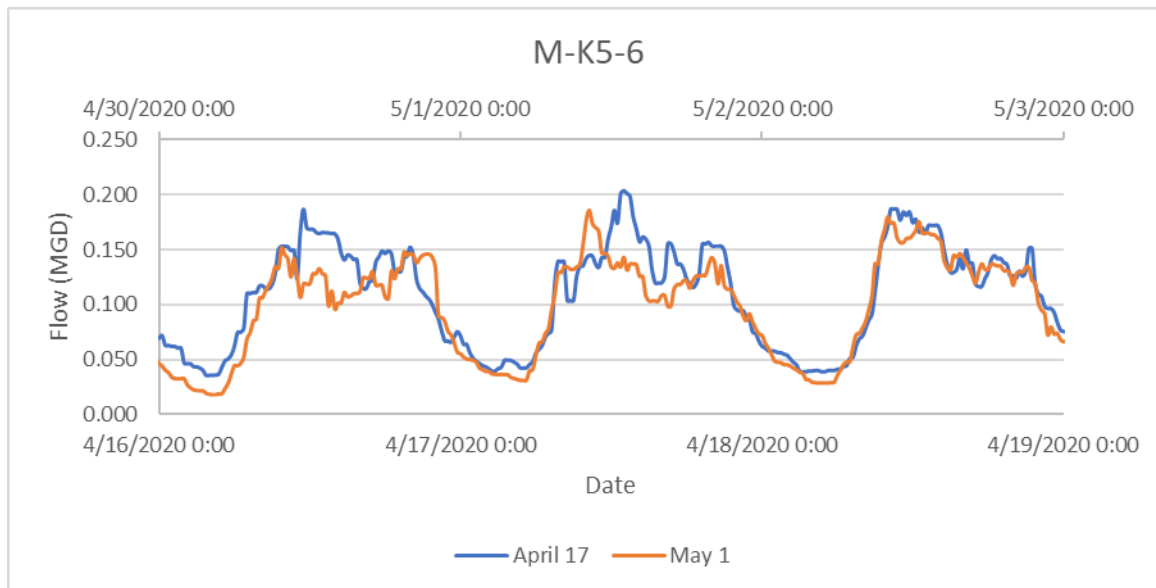


Figure 2-6. M-J4-35 Rain (Apr 17) vs Dry (May 1) Meter Flow

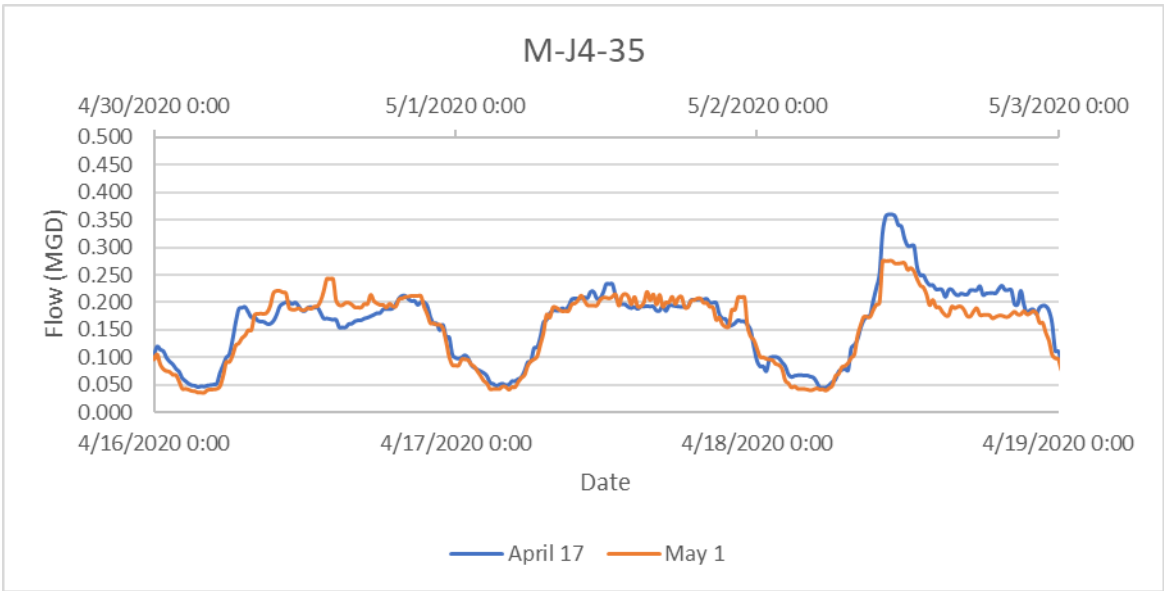


Figure 2-7. M-L5-24 Rain (Apr 17) vs Dry (May 1) Meter Flow

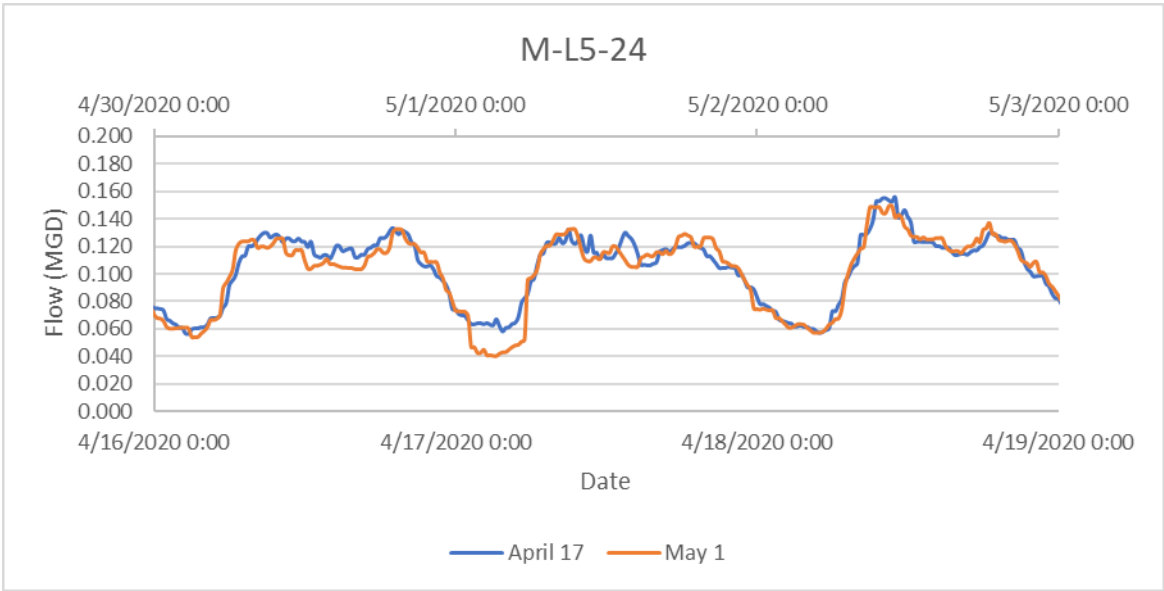


Figure 2-8. M-L6-13 Rain (Apr 17) vs Dry (May 1) Meter Flow

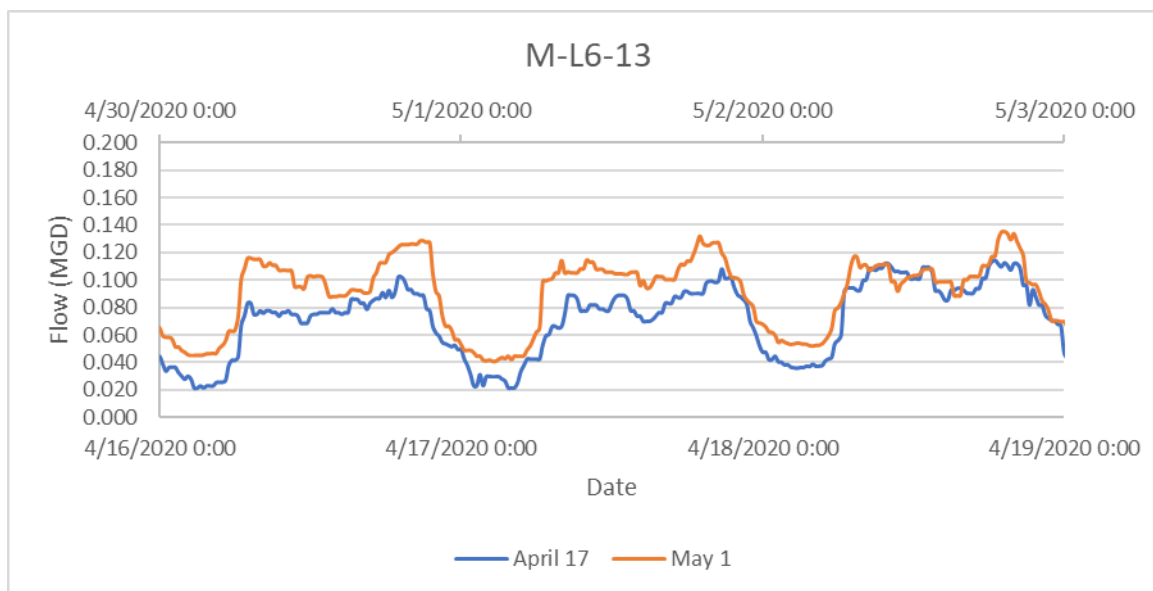
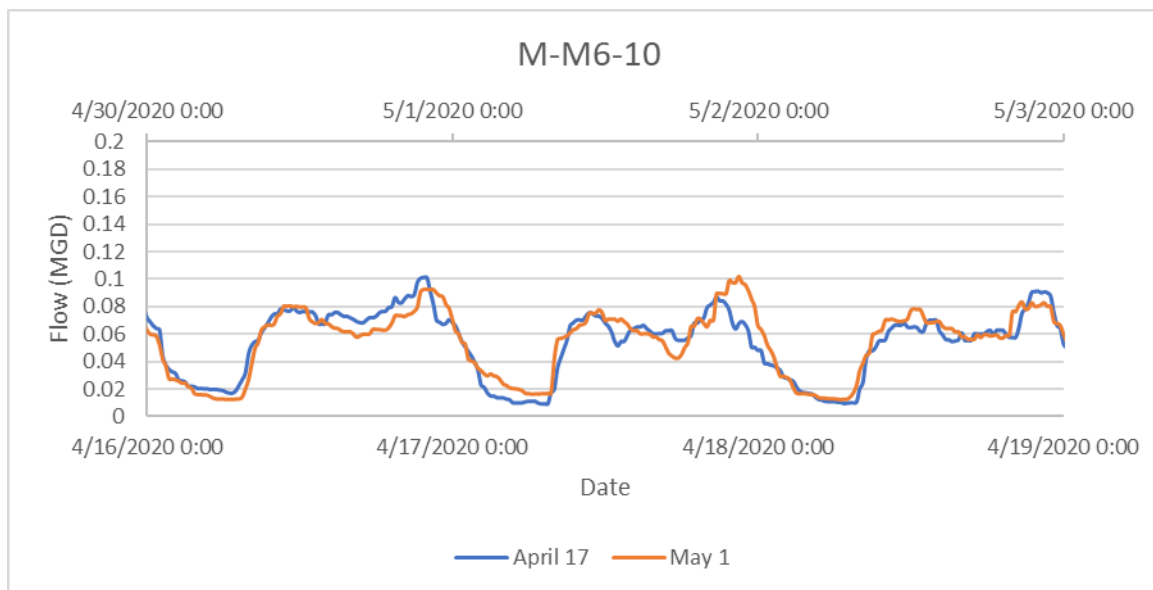


Figure 2-9. M-M6-10 Rain (Apr 17) vs Dry (May 1) Meter Flow



2.2.2 Wastewater Treatment Plant and Nanofiltration Plant Flow Data

The influent flow data from the WWTP were used to estimate the average daily flow and peak hour flow in the collection system based on a calculated peaking factor at the WWTP. Based on the flow data, from January 1 to June 8, 2020 (which overlaps the flow monitoring period used for model validation), the daily peak hour to average flow ratio averaged 1.5, with a maximum of 1.8. The validation model showed a peaking factor at the WWTP of 1.9, similar to that calculated for this period from the data, and therefore validating these model results.

Over a longer time period, using influent flow data at the WWTP, the historical maximum peak hour to average annual flow ratio (PH:AA) at the WWTP was determined to be 3.4. However, Jacobs and the Town performed a

review of historical flows from 2017 through 2019 to determine if the data indicating that peaking factor of 3.4 were accurate. After identifying unusual causes for several of the instances of high flow, Jacobs and the Town elected to adopt a peaking factor of 3.1 (PH:AA) for design at the WWTP, as shown in the WWTP Masterplan. Note that this equates to a peaking factor at the WWTP of approximately 2.8 for peak hour to max month (PH:MM). Due to attenuation through the collection system, it is necessary to load a higher peaking curve into the inflow nodes of the collection system model. Accordingly, the collection system model uses a PH:MM factor of 3.1 for residential flows for all planning scenarios, and slightly higher factors for commercial, industrial, and civic sources. (Notice that for planning purposes it is necessary to anticipate the highest peaks that are reasonably expected in the collection system, whereas the validation process is geared to assessing how the model corresponds to the limited period of flow monitoring in the existing collection system.)

The Nano/RO Plant releases its brine into the collection system when in operation, with even higher flows being fed into the sewer system during clean-in-place events. Flows from the Nano/RO Plant in the model were based on flow meter data and confirmed by model validation and the Town. A clean-in-place wash occurred 4 times between the end of May and the end of June 2020. The flow from the wash was determined to be around 0.45 MGD. Typically, the wash occurred right before the Nano/RO Plant would release a consistent stream of brine between 0.14 and 0.16 MGD to the collection system. After the fourth wash, the brine discharge continued for the remainder of the flow monitoring period through July.

2.2.3 Diurnal Patterns

Diurnal patterns representing a 24-hour period were assigned to each flow input to the system. For the existing model validation, residential, commercial, and industrial flows were applied to the validation weekend days (one with the Nano/RO Plant and one without the Nano/RO Plant), as the weekend demonstrated the worst-case scenario. A description and figure of each diurnal pattern are provided below. The first hour represents the time from 12:00 am to 1:00 am.

2.2.3.1 Residential Diurnal Patterns

For validation, the day of the week with the highest peak (which was not a holiday) was chosen for each flow monitoring location to create a general diurnal pattern to apply to residential influent flows. In general, weekends showed higher peaks than weekdays, with two peaks occurring in a day and the first peak as the higher of the two peaks; therefore, a weekend day was used as the basis for the residential diurnal pattern. The data compared for each meter are shown in Figure 2-10, which illustrates the similarities of the diurnal cycles among the meter locations. A peaking factor of 2.0 was selected for the residential pattern. While the peaking factor was lower than 2.0 for some of the meters, those meters in general had lower flows, and the more conservative peak of 2.0 was applied for these branches of the system as well. This diurnal pattern was applied to all influent nodes to model average residential flow for the existing collection system. The residential diurnal pattern used in the model is shown in Figure 2-11.

Figure 2-10. Diurnal Patterns at each Flow Meter

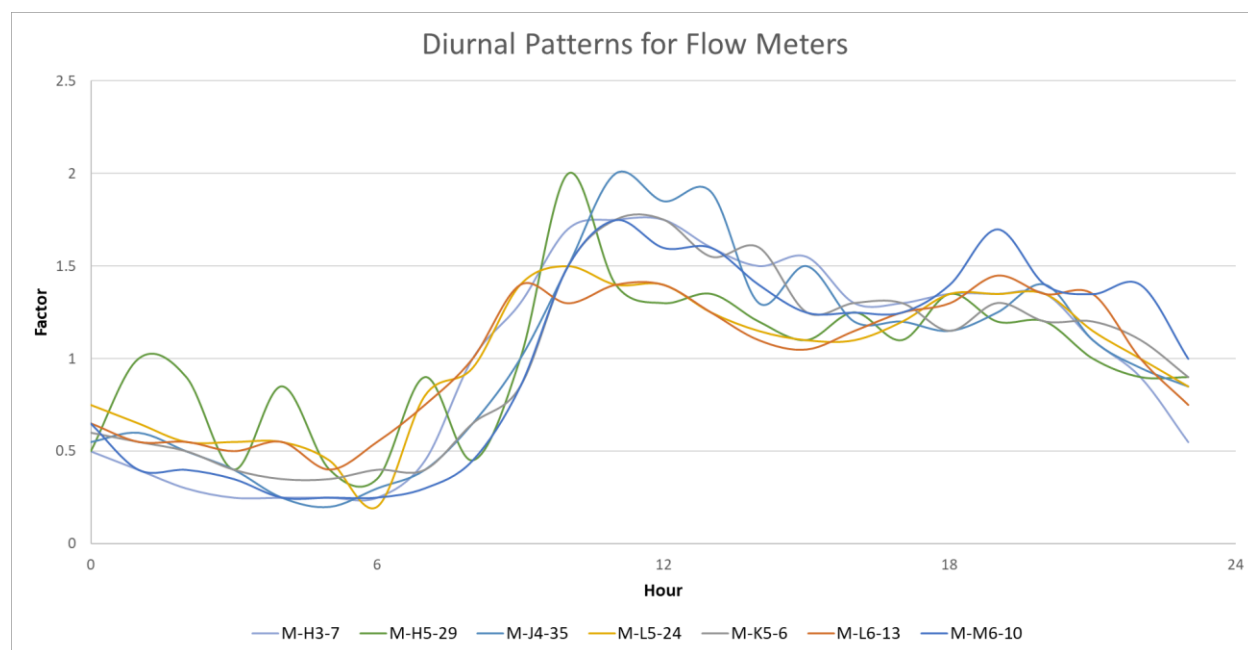
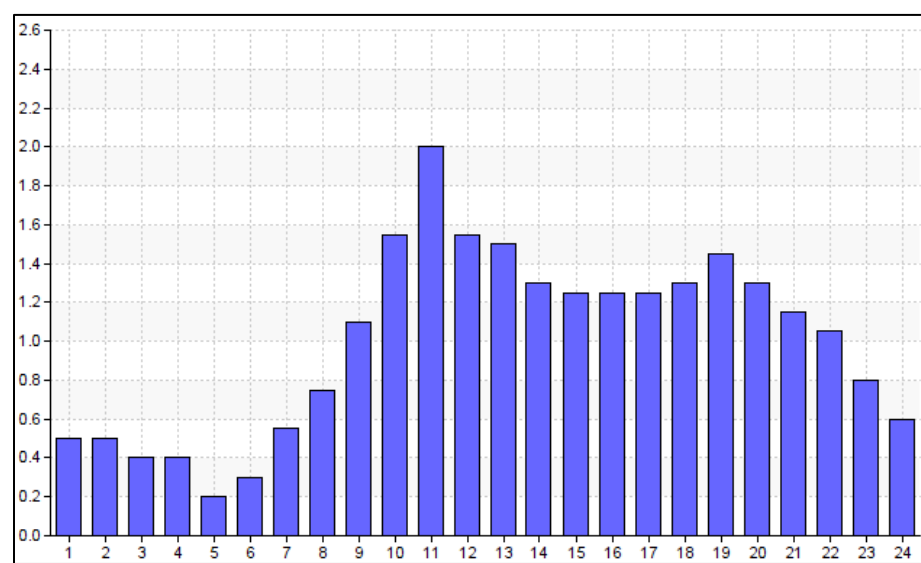


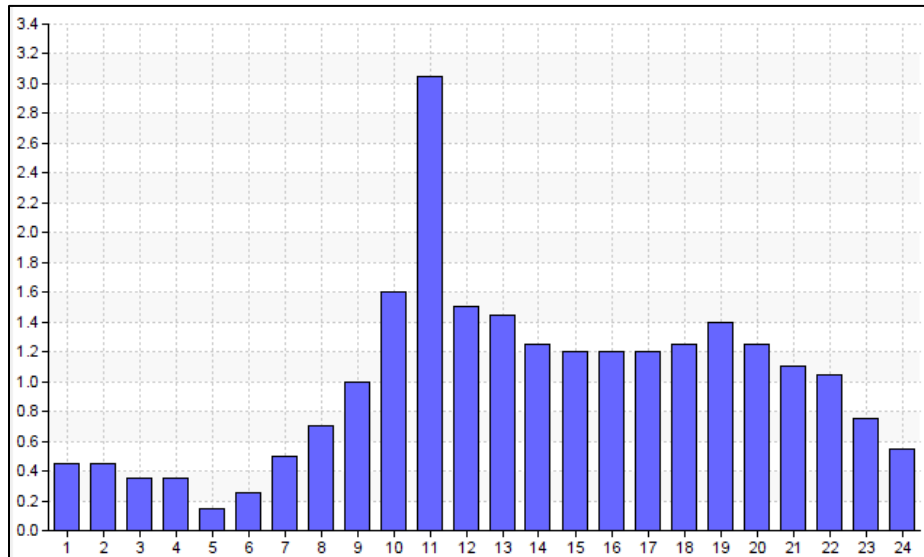
Figure 2-11. Residential Diurnal Pattern



For assessing the longer period record of flow data at the WWTP, a peak flow model was developed for the existing system using the design maximum PH:AA factor at the WWTP of 3.1 discussed above and in the WWTP Masterplan. The peak hour flow at the plant with a PH:AA factor of 3.1 is about 1.95 MGD (based on average annual flow of 0.63 MGD). To simulate existing condition flows at the plant for the design maximum peak hour, a model scenario was developed using maximum month inflows with a new, peak diurnal pattern that uses a 3.1 peaking factor for residential flows. This model simulation results in a peak hour flow at the WWTP of 1.81 MGD, which is within 10% of the 1.95 MGD value adopted in the design of the WWTP, suggesting that there is a small

but reasonable safety margin in the WWTP capacity, based on the review of flows over the past 5 years at the WWTP. The residential diurnal pattern for the peak hour scenario is provided in Figure 2-12.

Figure 2-12. Residential Peak Diurnal Pattern



2.2.3.2 Commercial and Industrial Diurnal Patterns

A peaking factor (PH:MM) of 4.0 was assumed for commercial and industrial flows. In general, it is expected that commercial flows occur mostly during the day when businesses are open. Industrial flows are more likely to be consistent throughout the day, but a peaking factor of 4.0 was applied around 11 am to simulate potential worst-case scenarios during a wash of the Nano/RO Plant system at the same time as the residential peak flow. The diurnal patterns for commercial and industrial flows are provided in Figures 2-13 and 2-14, respectively. The pattern used for the Nano/RO Plant is discussed in Section 2.2.4. (Civic peaks from facilities like schools were assumed to have a peaking factor of 6.0 (PH:MM) – see Section 5).

Figure 2-13. Commercial Diurnal Pattern

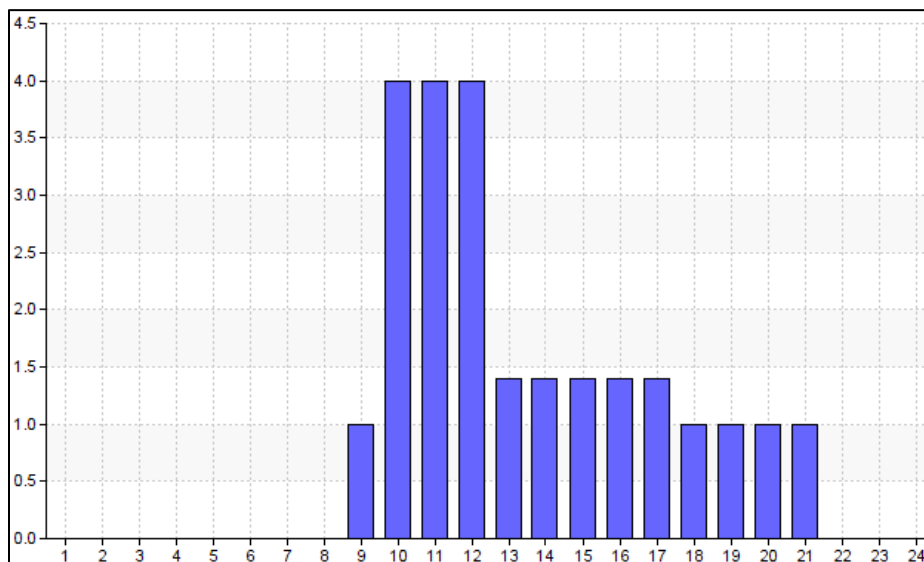
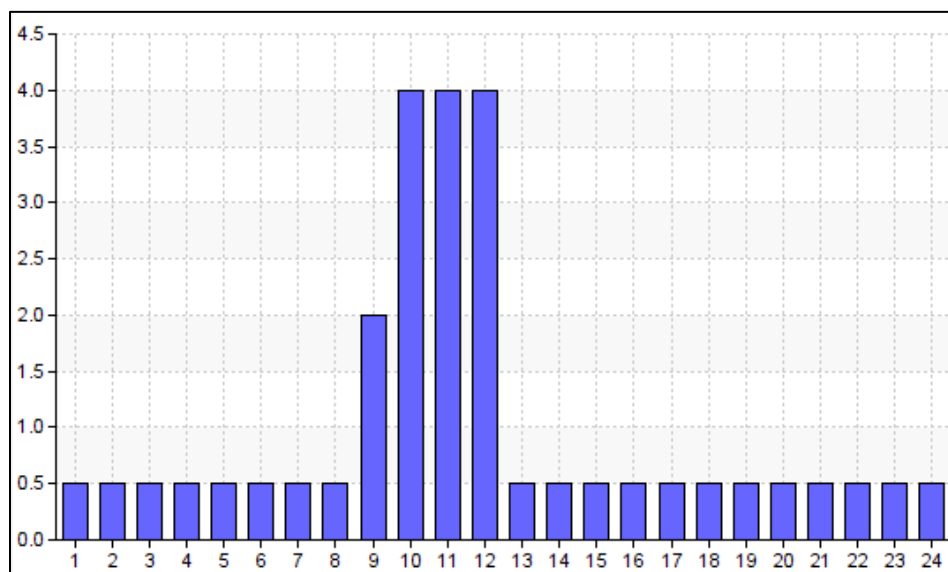


Figure 2-14. Industrial Diurnal Pattern



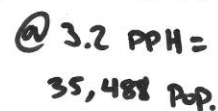
The Town does not currently have significant industrial discharges into the collection system; however, there are businesses in the area of future industrial expansion according to the DRAFT Comprehensive Plan. The industrial diurnal pattern was used for both existing and future flows in this area.

2.2.4 Influent Flow

Flows were calculated in a spreadsheet and then input into InfoSWMM as inflows to specific interceptor manholes (shown previously on Figure 2-1). Input locations were selected where pipes from neighborhoods connect to the interceptors. The following list summarizes how flow was estimated and then validated with the flow meter and WWTP data.

- Per the DRAFT Comprehensive Plan information provided to Jacobs in January 2021 (Figure 2-15), the residential dwelling units are assumed to hold 3.2 people.
- In the WWTP Masterplan, the annual average flow was determined to be 60.6 gallons per capita per day (gpcd) based on historical analysis of flows at the WWTP in 2019 and the 2019 population. This number included the Nano/RO Plant discharging for approximately two-thirds of the year; however, it was preferable to model the Nano/RO Plant discharge individually, so the per-capita flow rate needed to be amended. When in operation, the average discharge of the Nano/RO Plant is 150,000 gallons per day (gpd) per information provided by the Town; the average annual flow rate would be 100,000 gpd if the plant is assumed to discharge for only two-thirds of the year. The population in 2019 was approximately 10,400 people. At 60.6 gpcd for 10,400 people, the average annual flow is 630,000 gpd. If the average annual Nano/RO Plant flow is removed, this equates to 530,000 gpd, or 51 gpcd for equivalent annual average flow from all sources except the Nano/RO.
- For the existing model comparison with existing flows recorded during the flow monitoring: approximately 51 gpcd was applied for the 2020 flows for residential customers using the number of dwelling units shown in the Town's DRAFT Comprehensive Plan (see Figure 2-15) to account for population distribution in each ½ mile by ½ mile block in the existing developed area. However, the number of dwelling units was varied as needed based on aerial views of development extents and the flow meter data. A total population of about 10,000 people at 51 gpcd was modeled; a few commercial and industrial areas were included in the existing model, accounting for approximately 30,000 gpd total.

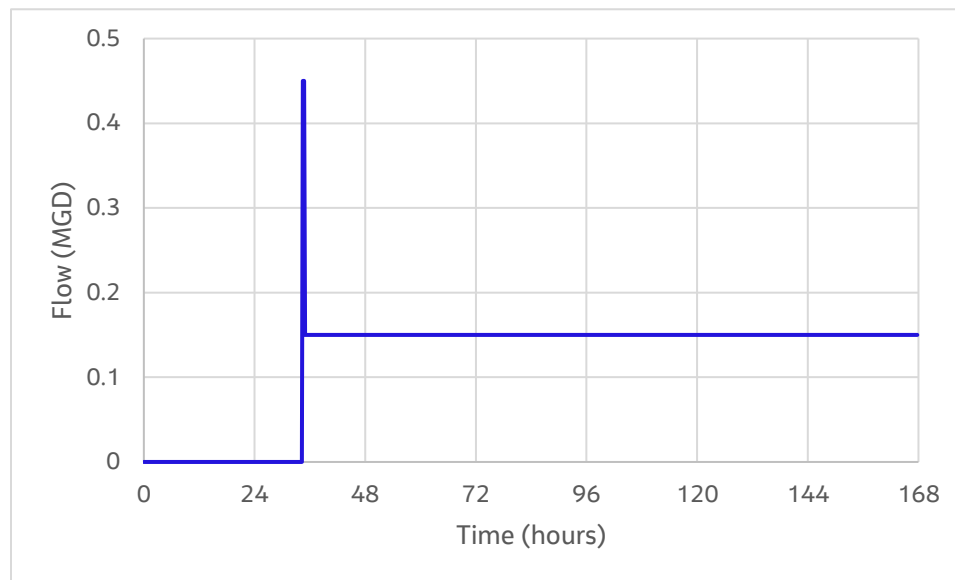
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17

- This collection system model is used for sizing both collection system and WWTP infrastructure, and the WWTP sizing is based on a maximum monthly flow rate. Thus, for consistency, the collection system model adopts base values of flows in a maximum month scenario: existing system-wide per-capita flow rate is increased from annual average of 51 to a maximum month of 54 gpcd from all sources, excluding the Nano/RO. (This value coordinated well with the model results from the actual collection system analysis and aligns with typical ratios of maximum month to annual average observed at the WWTP. The ratio selected in the WWTP Master Plan is slightly higher at 1.12, as that value is reflective of additional considerations related to WWTP sizing for flow and loading capacity. Note also that the future growth values of maximum month system-wide flow per capita increases from 54 to 66 gpcd, as discussed in Section 5.)
- During the flow metering period, the Nano/RO Plant discharged a flow of approximately 150,000 gpd (0.15 MGD) of brine with a steady stream and no diurnal pattern and occasionally discharged from a clean-in-place wash of approximately 450,000 gpd (0.45 MGD) for a 30-minute period. The brine discharge will be reduced when the RO is in operation; however, the peak flow resulting from the clean-in-place wash will still occur and should therefore be used to assess sewer sizes. The inflow pattern applied to the model for the Nano/RO Plant is shown in Figure 2-16.

Figure 2-16. Model Inflow for Nanofiltration Plant



3. Field Data Assessment

3.1 Field Survey and Investigation

In the initial model, there were four sections of pipe that were flatter than typically expected in a collection system. Additional survey was performed to verify invert elevations in these four sections. Two of the sections were adjusted slightly as a result of the survey, which increased the slope of the flat sections. One section in the northwest part of the collection system was found to have been previously surveyed with a different datum. The model was updated with the new survey information. The section which goes underneath I-25 in the southern part of the collection system was updated but is still relatively flat (the pipes between M-M5-12 and M-M5-14 have slopes of 0.07%, which is below the recommended minimum slope² for an 18-inch diameter pipe of 0.12%). The pipe sections that were adjusted are shown in Figure 3-1.

3.2 Meter Flow Data Assessment

The following list summarizes the assessment of flow monitoring data and the period of flow:

- The results and evaluations by HACH of the flow monitoring study quality are provided in Appendix A. The Manning's Analysis Curves of velocity versus depth represent well-distributed, reasonable data.
- Data were compared between the rainy and dry days, showing minimal influence from inflow and infiltration (see Section 2.2.1).
- Monitoring started during the COVID pandemic lockdown in Larimer County, which occurred from March 25 to May 8, 2020. Additionally, following the lockdown, typical commuting patterns and commercial uses were affected. Based on this information, it is anticipated that the diurnal patterns may not be typical, but they are expected to be conservative since most of the flow is residential and the residential flows were likely higher during the pandemic due to residents working from home.
- No silt buildup was found in any monitoring location.

3.3 View Pointe Lift Station

3.3.1 Lift Station Evaluations

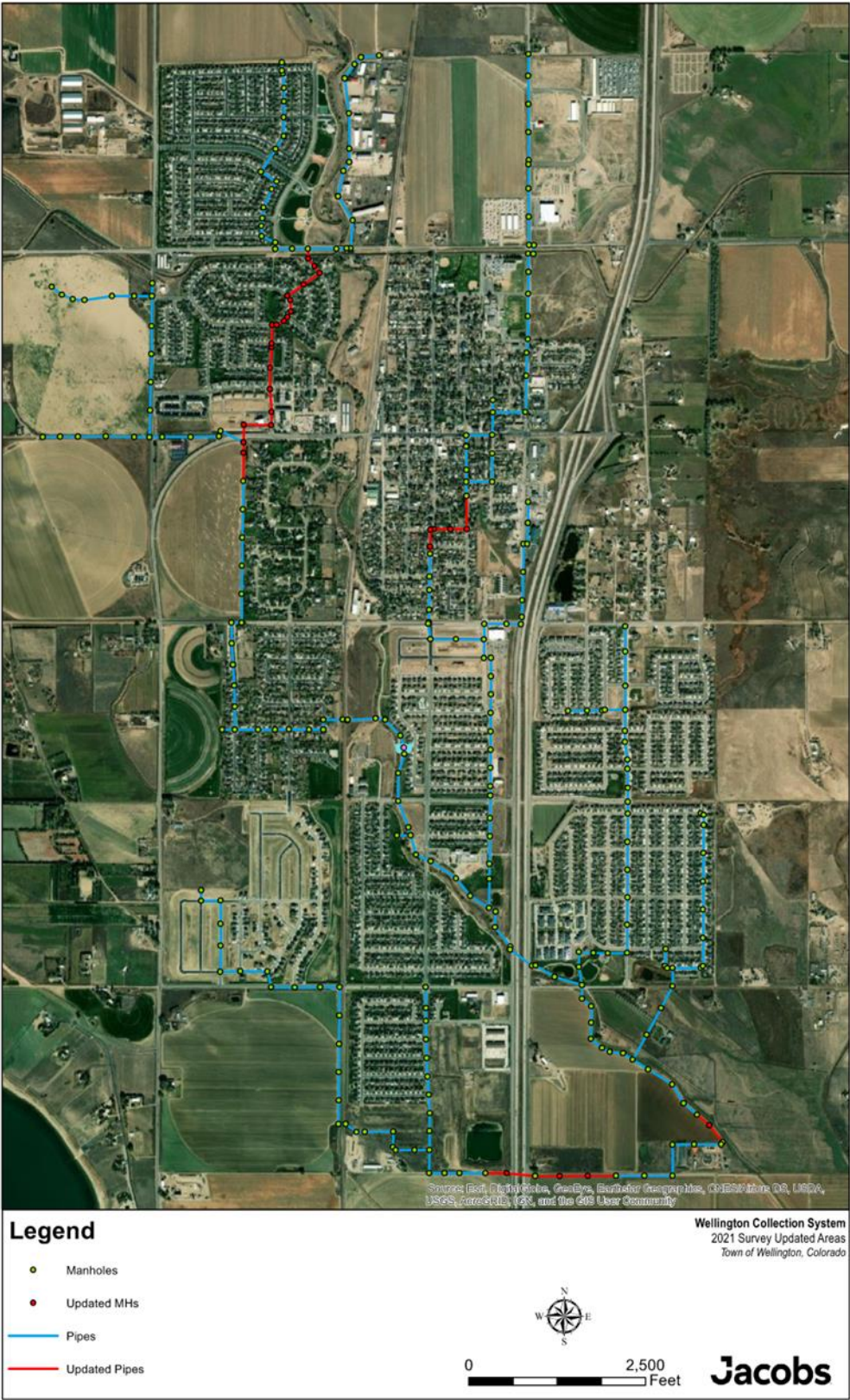
Studies of the condition and remaining spare capacity of the View Pointe Lift Station were conducted for the Town by others just prior to the start of this master plan. Findings of that report (JVA, 2019, *Town of Wellington View Pointe Lift Station Capacity Analysis*) are included in Appendix B.

In summary, this lift station is anticipated to handle the pending additional flows from the high school, but some added capacity will be required as growth occurs upstream of the lift station. In addition, due to the age and condition of the pumps, and the limited access for servicing this facility, work is anticipated to be required in the near term.

Prior to initiating any major repairs to the existing wetwell and pump station, it is recommended that a preliminary engineering level investigation compare the costs and benefits of a replacement lift station against an online upgrade. The advantage of a new wetwell and lift station is that it can be sized for optimum performance for the projected future flows and that it can be constructed offline, whereas upgrades to the existing facility will require bypass pumping operations and the wetwell depth and diameter will be constrained to existing dimensions. For the purpose of this master plan, the cost estimate assumes an allowance for modest upgrades to the existing lift station.

² Health Research, Inc., Health Education Services Division, 2014, *Recommended Standards for Wastewater Facilities*, 2014 Edition, A Report of the Wastewater Committee of the Great Lakes – Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers. Commonly known as the "Ten State Standards."

Figure 3-1. Pipes Updated from 2021 Survey



For the purposes of that preliminary engineering:

- Current firm capacity from the 2019 JVA letter report = 620 to 750 gpm depending whether the pump curve or field draw-down test is used, respectively.
- 2019 JVA estimated peak influent flow prior to the high school coming online = 482 gpm
- 2019 JVA estimated peak influent flow with the school flows added in (excluding any additional system growth) = 508 gpm
- Peak flow recorded during the Hach flow monitoring in 2020 just upstream of the lift station = 260 gpm
- Master Plan estimated current peak influent to the lift station = 410 gpm, not including the pending new high school flows.
- Master Plan estimated peak influent prior to the estimated implementation of the West Interceptor in approximately 2040 = 1,350 gpm
- Master Plan estimated peak influent at Buildout with the West Interceptor in service = 930 gpm

As can be seen from these data points, the peak flow rate through the lift station is anticipated to occur prior to the West Interceptor coming online. The design of lift station improvements/replacement should anticipate the higher value unless there are changes in the sequencing of when the West Interceptor is planned to be placed into service.

3.3.2 Evaluation of Option to Eliminate the View Pointe Lift Station

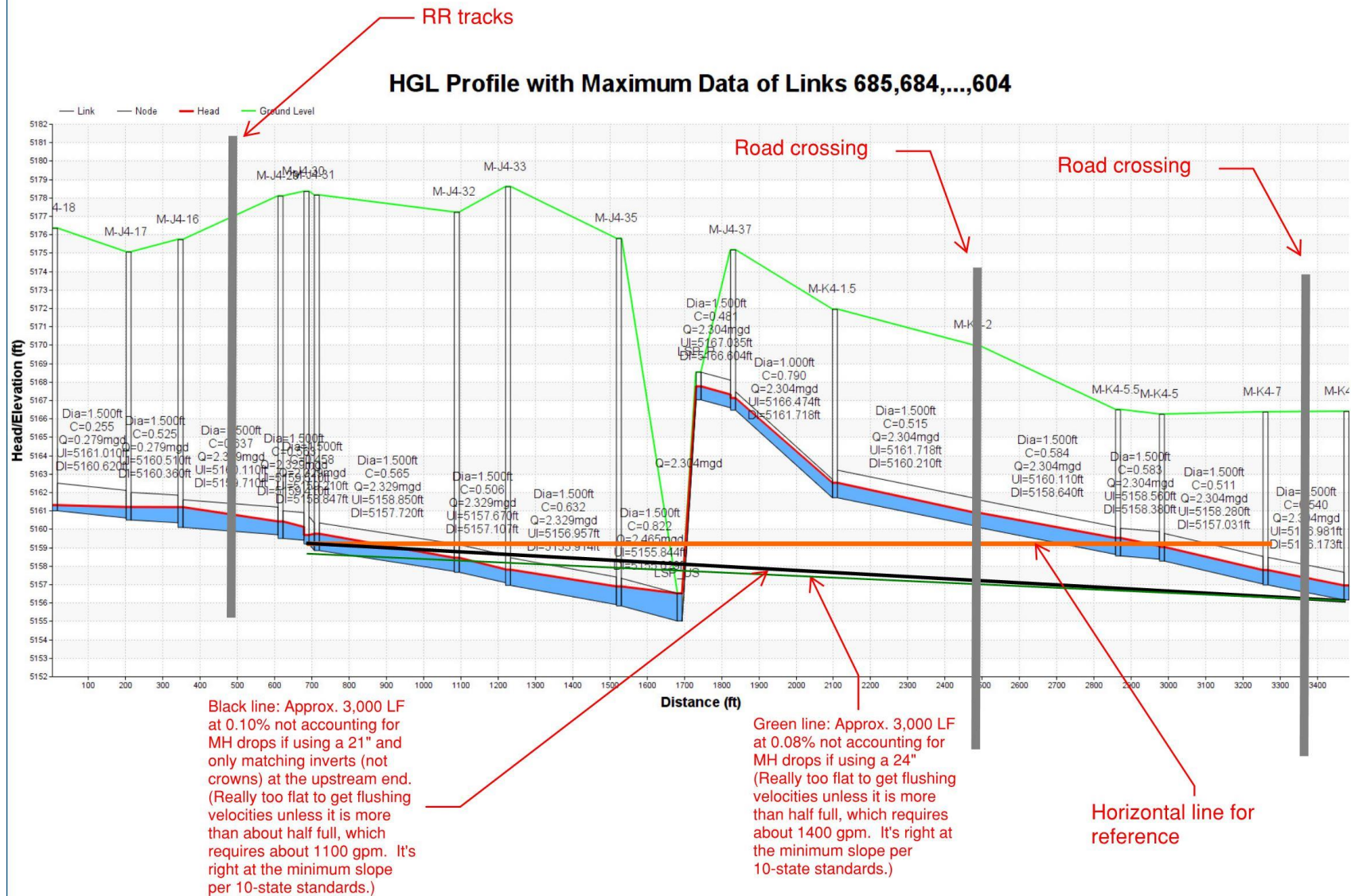
Elimination of lift stations is often a goal, as long term operational and maintenance costs can be eliminated. To eliminate the View Pointe Lift Station, it would be necessary to construct approximately 3,000 linear feet of new sewer from manholes M-J4-30 to M-K4-8, as depicted in Figures 3-2 and 3-3.

The cost of this replacement is expected to be on the order of \$3M, which is considerably higher than the cost to upgrade a lift station. Actual costs would depend upon many factors including, but not limited to: whether the roads could be open cut; the depth of trench rock; the amount of groundwater that would need to be managed; construction and permanent easement availability; and environmental issues, being in close proximity to the creek. Given these concerns, the option for lift station elimination is not pursued further in this master plan.

Figure 3-2. Lift Station Elimination Manhole Extents



Figure 3-3. Lift Station Elimination Profile



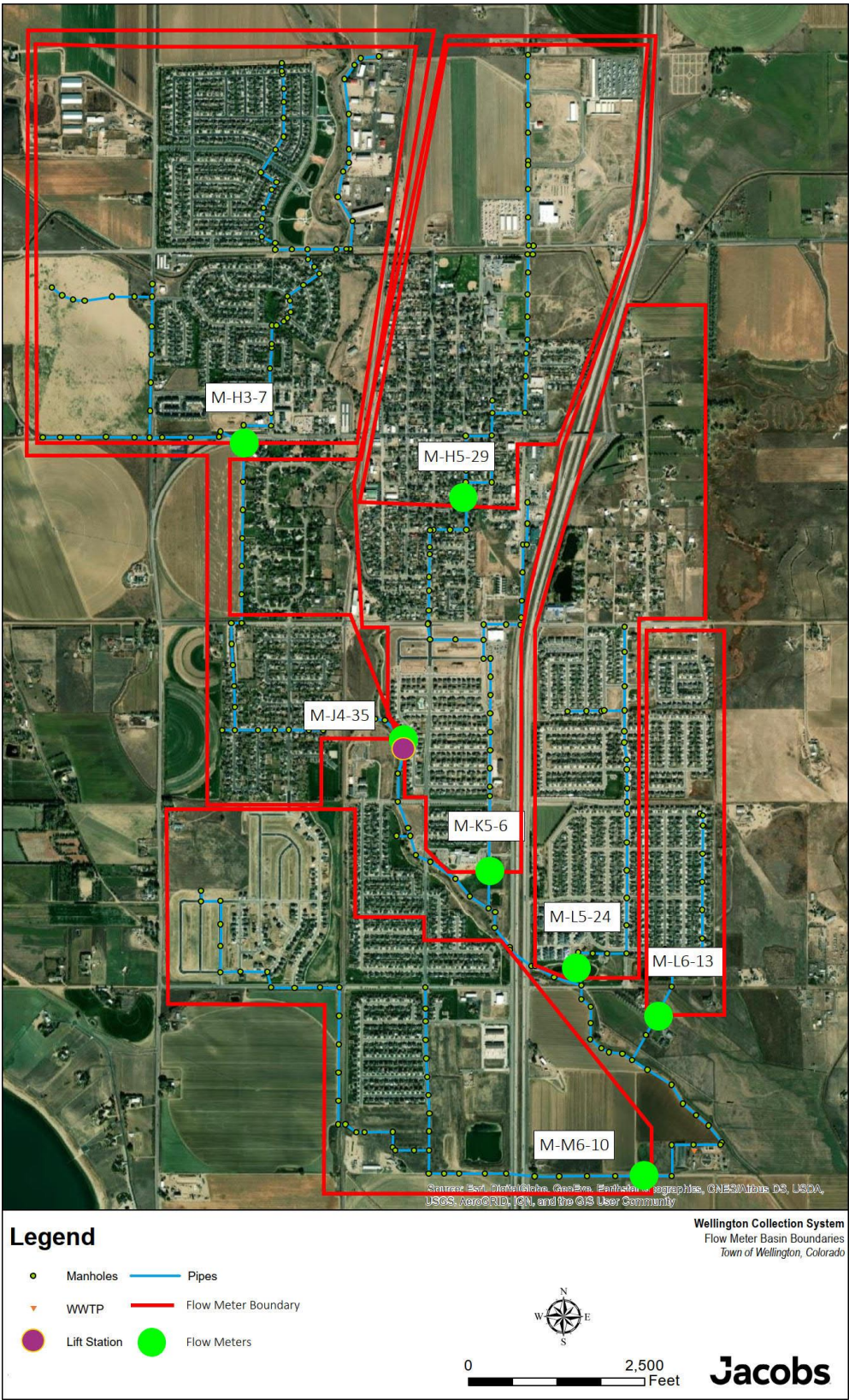
4. Collection System Model Validation

To validate the model, the initial model results, which used the number of dwelling units estimated from aerial photographs and a per-capita flow rate with the diurnal patterns described above, were compared to the flow meter data. Based on the comparison, the model was low in some areas and high in others. Flows were redistributed to match the flow peaks and patterns in the flow meters while maintaining historical flows at the WWTP. Following the redistribution, flows were then validated based on matching existing dwelling units from the DRAFT Comprehensive Plan, aerial development estimates, and monitored flow rates.

4.1 Meter and Validated Model Comparison

Model data were compared to meter data, both with and without discharges from the Nano/RO Plant. Saturday, April 18 and Sunday, April 19, 2020, were selected as comparison dates without Nano/RO Plant discharges for most meter locations. Saturday, May 2 and Sunday, May 3, 2020 were used for two meter sites due to anomalous meter data on April 18. A clean-in-place wash of the Nano/RO Plant occurred on Saturday, June 27, 2020, and it remained on afterwards, releasing a steady discharge to the collection system. The two flow meters on the trunk with the Nano/RO Plant discharge were compared to model data with the Nano/RO Plant running for Saturday, June 27 and Sunday, June 28, 2020. A map which outlines the location of each flow meter with the boundaries of the areas that discharge to each flow meter is provided in Figure 4-1.

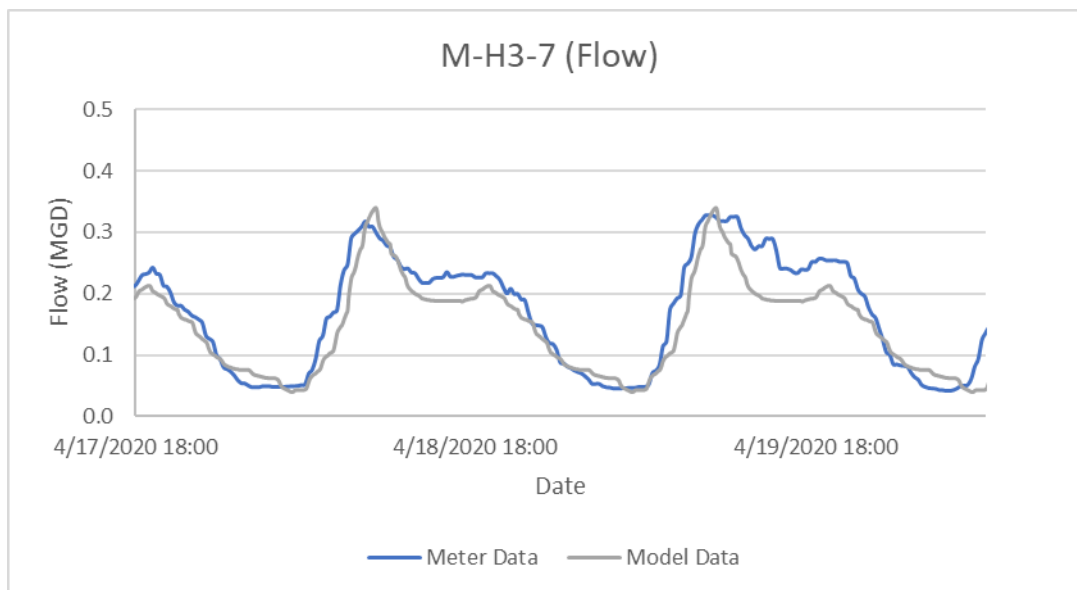
Figure 4-1. Flow Meter Basin Boundaries



The following describes the location of each flow meter and provides a graph of the comparison of the meter data to the validated model data.

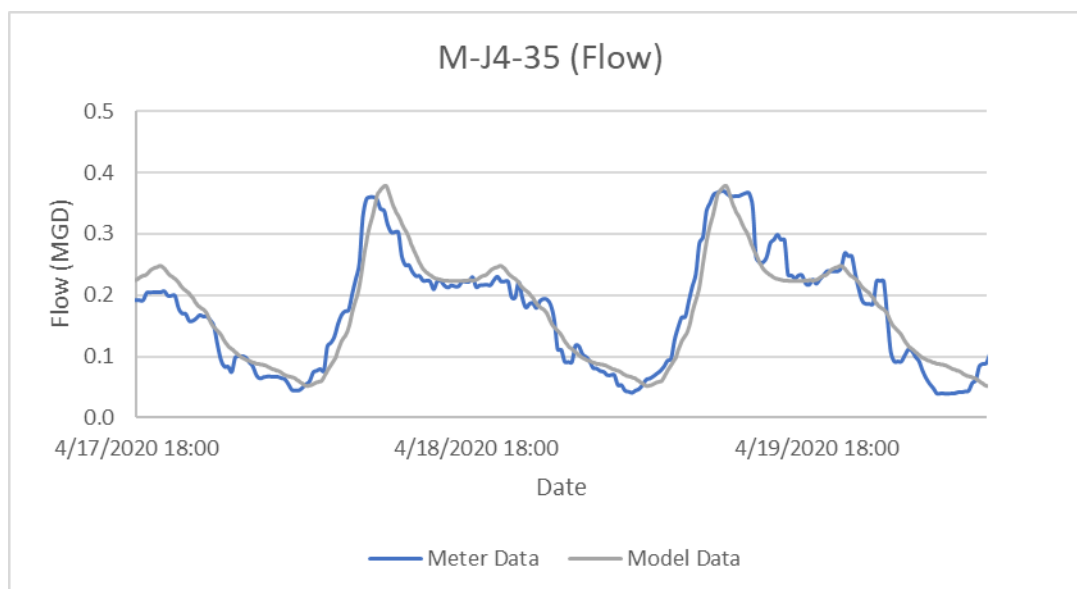
The flow meter located in Manhole M-H3-7 represents an area approximately one mile long by a half mile wide in the northwest area of the existing Town limits. The comparison of the meter flow to the model flow data is provided in Figure 4-2.

Figure 4-2. M-H3-7 Meter vs Model Flow Comparison



The meter located in manhole M-J4-35 represents flow directly before the View Pointe Lift Station (VPLS) and includes the flow from the upstream meter in Manhole M-H3-7. The comparison of the meter and model flow is provided in Figure 4-3.

Figure 4-3. M-J4-35 Meter vs Model Flow Comparison



The meter located in Manhole M-H5-29 represents flow for 0.75 miles of trunk length and is located south of the Nano/RO Plant. Figure 4-4 represents the flow comparison with the Nano/RO Plant brine discharge off and Figure 4-5 represents flow with the Nano/RO Plant brine discharge on with a clean-in-place wash. This wash and brine discharge are still expected to occur in the future and are therefore included in the future flow model scenarios.

Figure 4-4. M-H5-29 Meter vs Model Flow Comparison (without Nano/RO Plant Discharge)

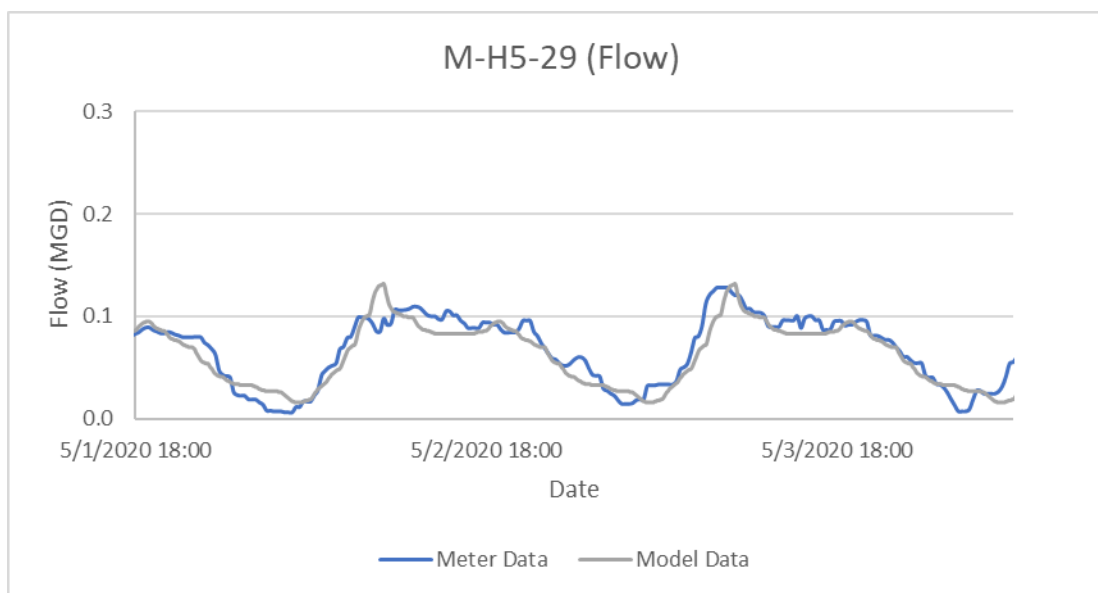
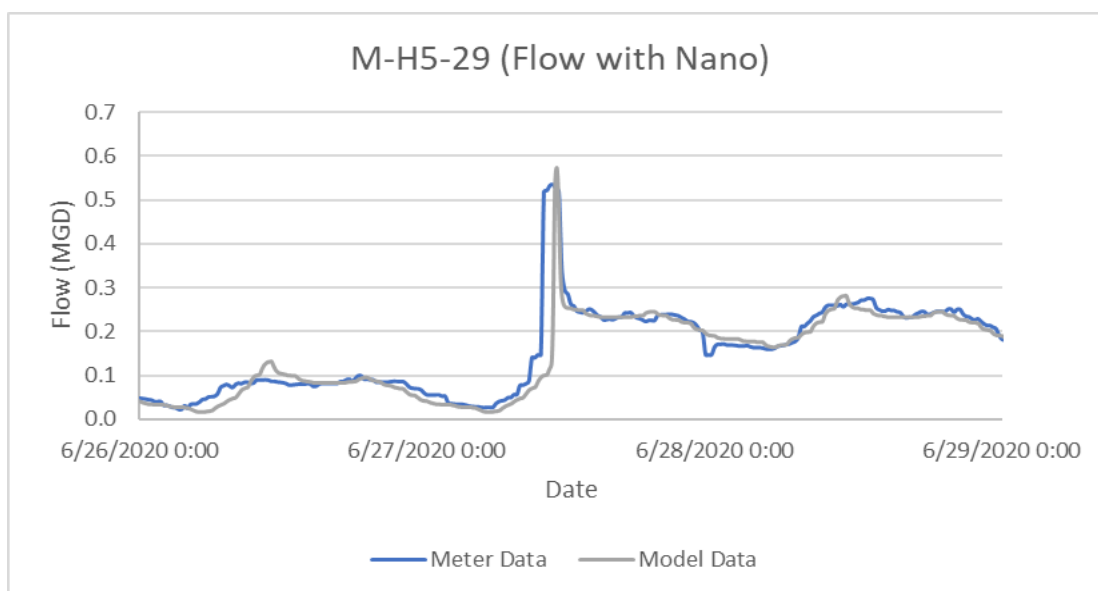


Figure 4-5. M-H5-29 Meter vs Model Flow Comparison (with Nano/RO Plant Discharge)



The meter located in manhole M-K5-6 was about 2 miles downstream of the previous meter. Note, this manhole is labelled as M-L5-25 in Appendix A; however, it was confirmed that the actual manhole location is M-K5-6. Figure 4-6 represents the flow comparison with the Nano/RO Plant brine discharge off, and Figure 4-7 represents flow with the Nano/RO Plant discharge on.

Figure 4-6. M-K5-6 Meter vs Model Flow Comparison (without Nano/RO Plant Discharge)

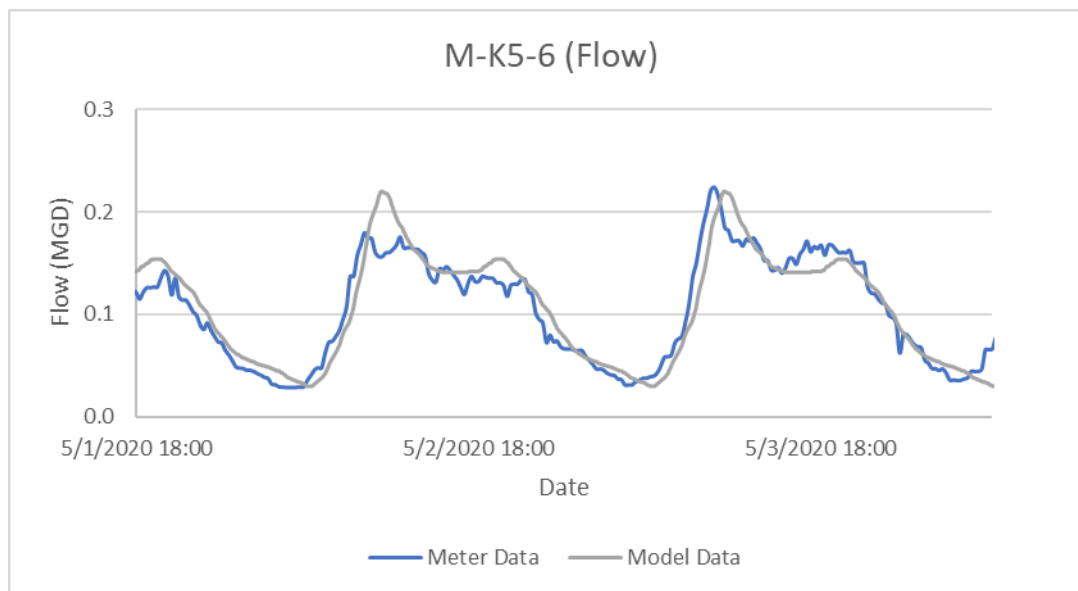
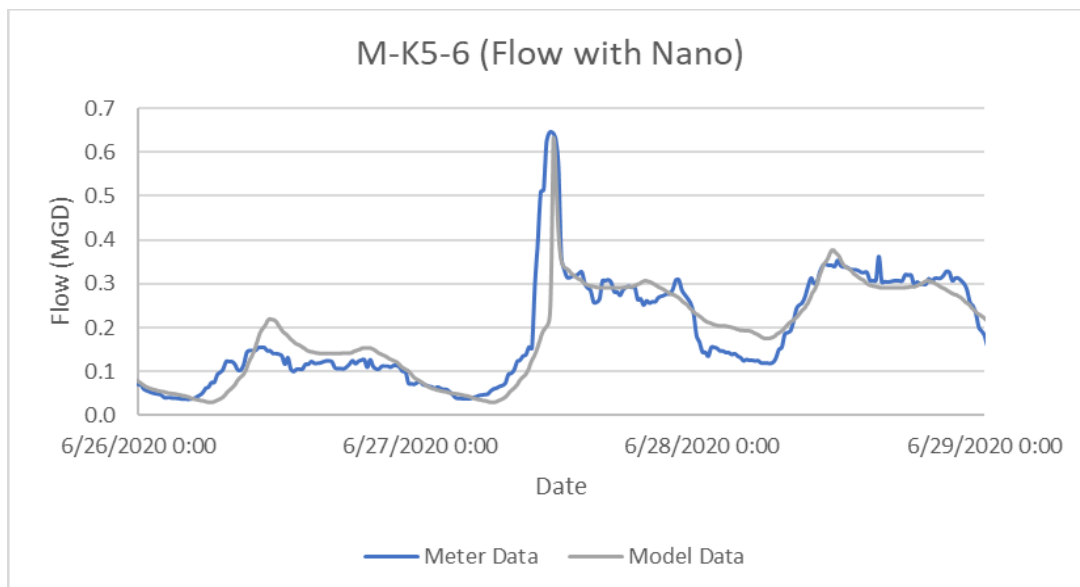
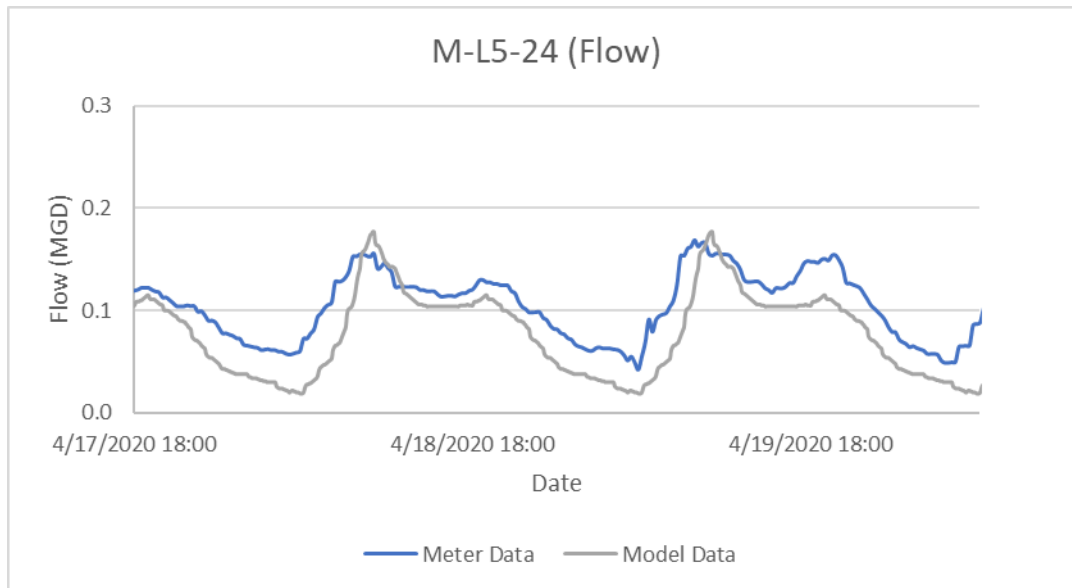


Figure 4-7. M-K5-6 Meter vs Model Flow Comparison (with Nano/RO Plant Discharge)



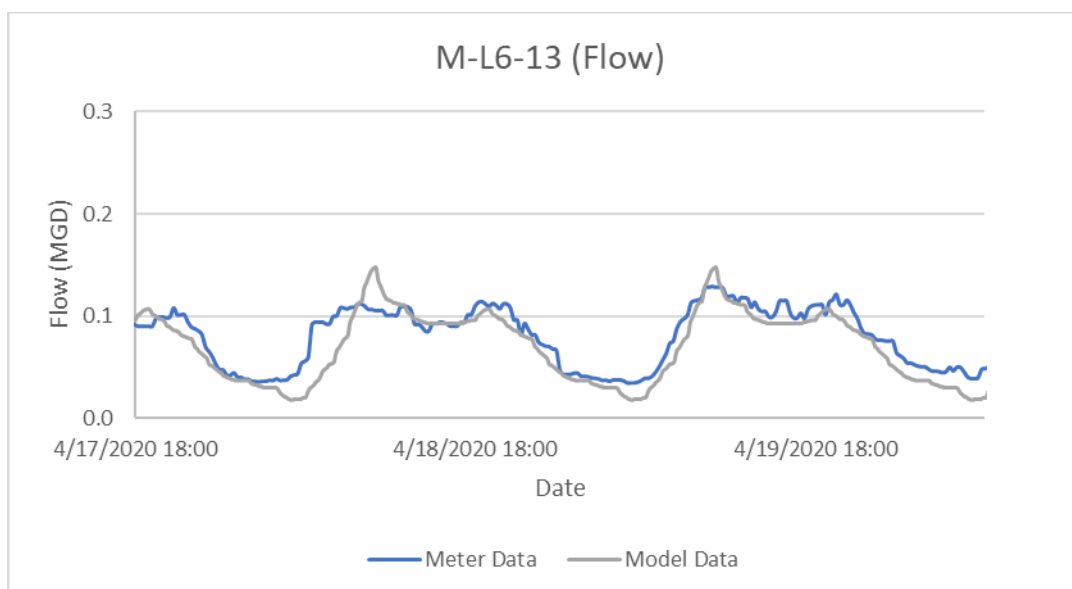
The meter located in manhole M-L5-24 represents flow in the first trunk line to the east of I-25. The comparison of the meter and model is provided in Figure 4-8.

Figure 4-8. M-L5-24 Meter vs Model Flow Comparison



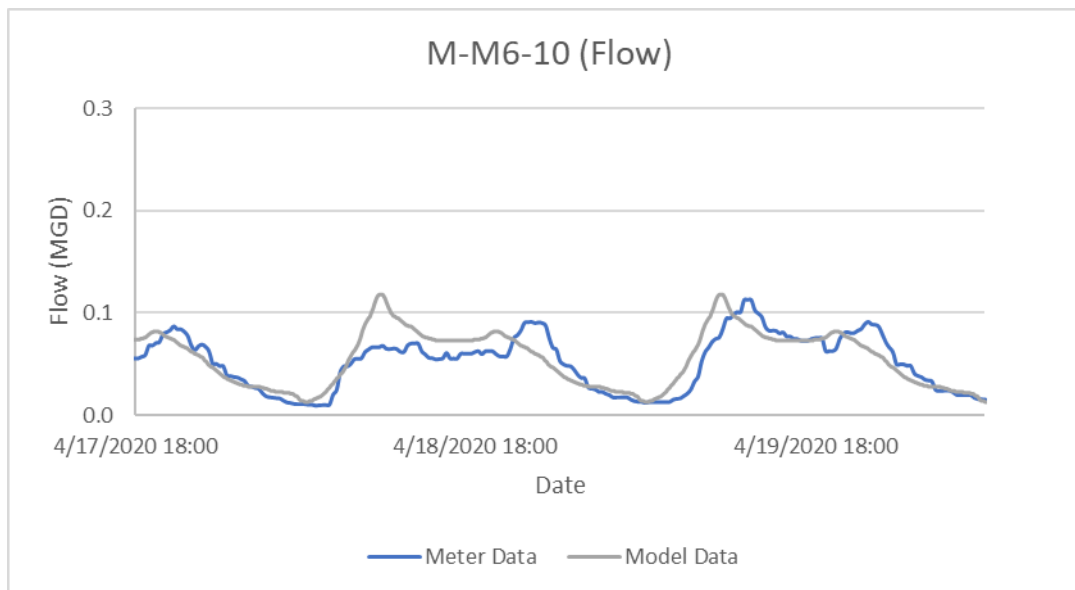
The meter located in manhole M-L6-13 represents flow in the second trunk to the east of I-25. The comparison of the meter and model flow is provided in Figure 4-9.

Figure 4-9. M-L6-13 Meter vs Model Flow Comparison



The meter located in manhole M-M6-10 represents flow for the trunk conveying flow to the WWTP from the west. The comparison of the meter and model flow is provided in Figure 4-10.

Figure 4-10. M-M6-10 Meter vs Model Flow Comparison



Based on the validated model, approximately 10,000 people are represented in the model at 51 gpcd. This is within 10 percent of the expected population of 10,700, which is between the 2019 population and 2020 population as described in the WWTP Master Plan. A summary of the modeled dwelling units per grid is provided in Appendix C.

The model results confirm what Town staff has observed: the existing collection system conveys flow without capacity concerns, based on a review of the modeled depth of flow. The existing View Pointe Lift Station needs updating due to unsafe conditions, as previously discussed in Section 3.

4.2 Model Limitations and Refinement

The InfoSWMM model for existing conditions was successfully validated using data acquired with the flow meters and from the WWTP inflow data. Model results match meter data well at each of the metered locations. The model can be used to support planning-level decisions such as developing and confirming capital improvement plans and prioritizing projects.

Caution should be employed if the model is used to evaluate design-level questions. Because of the relatively small number of meters used, the minimal storm events evaluated, and the fact that it is not an all-pipes model, some model results may not be accurate enough to support design decisions. In such a case, information from the model may be able to be used to supplement other information used to make design decisions, but it is recommended that model results be viewed in light of their potential error for such an application.

The following list outlines some limitations and recommended improvements for future refinement of the model:

- Periodic updating of the model is recommended whenever there are significant changes implemented in the collection system and/or when new growth occurs. It would also be very beneficial to expand the model detail on the proposed future interceptor sewers once preliminary design is performed.

- The lift station was modeled with an “ideal pump” to have outflow equal inflow as it is understood that the lift station will be upgraded as required to properly convey influent flows. Once the new lift station is upgraded, future model scenarios could be modeled with specific pump curves.
- Flow monitoring occurred during 2020 while there was a state-wide pandemic lockdown limiting travel for work and patronizing of businesses. To further validate flow patterns, additional flow monitoring during more typical conditions is recommended.
- Flow monitoring was conducted in a small number of locations. Acquiring data to compare to model results in more locations may facilitate model improvements to increase accuracy and/or increase confidence in model results in areas that have not yet been monitored.
- Currently, InfoSWMM does not model headloss in manholes. To provide some conservatism in flow depths appropriate for a planning-level model, additional headloss was imposed on the system by using a Manning's n value of 0.014 for all pipes, which is higher than what would typically be used for PVC pipes. The flow depths in the validation model were compared to the 2020 meter data. Modeled peak depths were similar to or slightly more conservative than the metered depths. Further refinement may be useful in improving the model by including entry and exit losses on pipes to and from the manholes. Since the Buildout model does not currently show all potential manholes, interceptors could be refined to model the anticipated number of manholes.
- No allowance for I/I was included in the model, based on flow monitoring data that did not show significant wet weather influence. As collection systems age, however, I/I influence can increase, so it is recommended that the need for including such an allowance be revisited from time to time so that future flow projections can include it if appropriate. An alternative approach would be to plan for rehabilitation or replacement of piping where I/I becomes problematic.

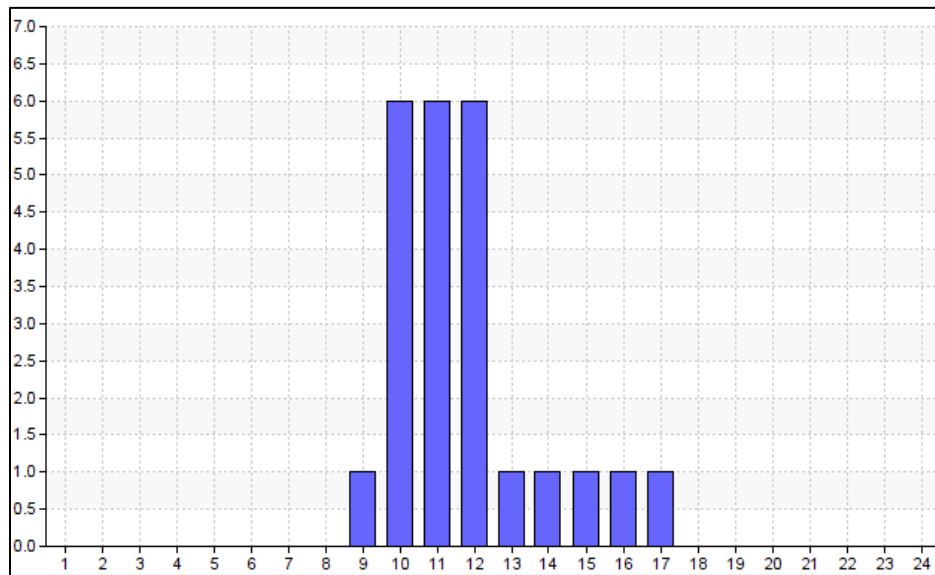
5. Buildout and 2040 Model Development

To evaluate capital improvement needs, a model scenario was developed for the full Buildout of the collection system with a population of approximately 35,500, based on the DRAFT Comprehensive Plan. Once the infrastructure needed for the Buildout scenario was determined, another model scenario was developed that represents a population of about 25,000 people—assumed to occur around the year 2040—with the goal of determining whether significant improvements are required to meet capacity changes over the next 20-year planning period and to explore how much growth may be accommodated by the existing collection system. In both future model scenarios, interceptors were extended in different directions outwards from the existing collection system to capture the anticipated flows from future growth. This section discusses the development of these model scenarios.

The existing residential and commercial/industrial flows were estimated based on the flows at the wastewater treatment plant in 2019 without the Nano/RO Plant online, as well as a review of aerial photographs of the Town and flow meter data, as previously described in Section 2.2.4. For future growth, a slightly higher per-capita flow rate was used for residential flows; per conversations with the Town and their DRAFT Comprehensive Plan, higher rates were requested to represent the Plan's intention to encourage expanded growth in commercial, industrial, and civic flows as the Town develops towards Buildout. For the existing population of approximately 10,000 people, a per-capita flow rate for residential, commercial, civic, and industrial flows of 54 gpcd was used, as described in Section 2.2.4. For all future population (totaling approximately 25,000 additional people by Buildout), a per-capita rate for residential, commercial, industrial, and civic flows of 66 gpcd was used. These per-capita rates of 54 and 66 gpcd were adopted as maximum month flow rates, corresponding to those used in the WWTP Master Plan. For the Buildout population of about 35,500 people, these rates correspond to a future systemwide average maximum month rate of approximately 62 gpcd across all use types (except for the Nano/RO Plant, which is modeled separately). This future systemwide average rate of approximately 62 gpcd for maximum month flow corresponds to an average annual flow rate of 56 gpcd (using the WWTP maximum month to average annual ratio [MM:AA] of 1.12 from the WWTP Master Plan). This average annual per-capita rate is approximately 9% higher than the equivalent average annual per-capita flow rate for existing conditions of 51 gpcd described in Section 2.2.4, consistent with the Town's goals. For all model scenarios, the Nano/RO plant was included per the pattern previously discussed in Section 2.2.4.

A diurnal flow pattern was added to represent civic flows for the 2040 and Buildout scenarios. Civic flows are used to represent buildings such as schools. In general, these flows occur during the day and not at night. It is anticipated that a peak would occur before, during and after the lunch hour based on food preparation and cleanup. The diurnal pattern assumed for future civic flows, including a 6.0 peaking factor to provide some conservatism due to the uncertainty in projected future flows, is provided in Figure 5-1.

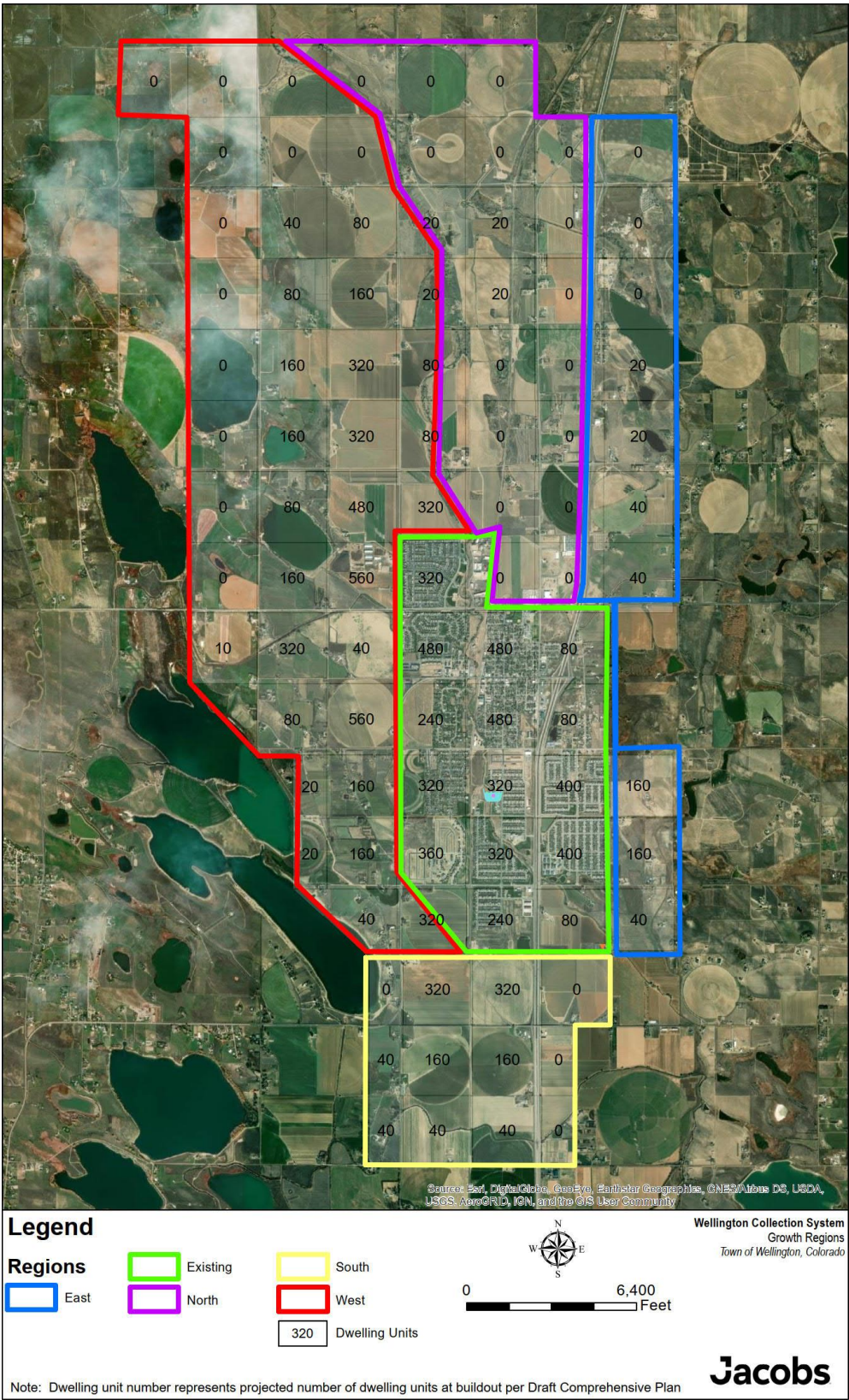
Figure 5-1. Civic Diurnal Pattern



Four general regions for growth outside of existing Town limits were defined by limiting geographic features as depicted in Figure 5-2. The west region is bordered to the east by Boxelder Creek and the existing Town extent. The north region is bordered to the west by Boxelder Creek and to the east by I-25. The east region borders I-25 and the existing Town extent on its west. The south region includes all potential growth areas south of the existing Town extent; this region decreases in elevation with distance from the Town.

Sections 6 and 7 describe model findings and proposed infrastructure for Buildout and 2040 recommendations, respectively.

Figure 5-2. Regional Growth Areas Defined for the Model Scenarios



6. Buildout Recommendations

6.1 Proposed Interceptors

A general layout for future interceptors needed to convey full Buildout flows was developed by geographic region. Manholes and pipes were added for each region, and flows based on numbers of dwelling units from the DRAFT Comprehensive Plan for residential, civic, commercial, and industrial uses were developed and assigned to the manholes in their corresponding regions. Pipe sizes were determined for Buildout using a maximum depth to diameter ratio (d/D) of 0.65 for new pipes (compared to 0.8 for existing pipes) to provide conservatism in anticipated pipe sizes. The layout of the proposed Buildout collection system is provided in Figure 6-1.

Jacobs recommends adding an East Interceptor, a North Interceptor, a West Interceptor, and two South Interceptors based on the physical limitations of existing land and infrastructure, as follows:

- The East Interceptor would run along the east side of I-25.
- The North Interceptor would run between I-25 and the east side of Boxelder Creek.
- The West Interceptor would run from northwest of the Town along the west side of the existing Town extents and capture flows from development west of Boxelder Creek. The line will connect to the existing 30-inch pipe located in Sage Meadows (which is large enough to handle anticipated Buildout flows), continue southeast of the 30-inch pipe, cross the railroad tracks, and then run parallel to the southern side of the existing interceptor, under I-25 and east to the WWTP. The existing 10-inch pipe for the new Poudre School District high school is not sufficient to carry all new flows for Buildout in the vicinity. Under the currently modeled assumptions, the West Interceptor will need to parallel this pipe.
- The South Interceptors would run from south to north on either side of I-25 and require one or more lift stations based on the slope of the ground increasing moving north toward the WWTP.

Google Earth elevation profiles for the proposed East, North, and West Interceptors were reviewed to generally assess the feasibility of gravity-flow interceptors where shown in Figure 6-1. Based on the elevation profiles, it is expected that the pipes can remain between 10 and 20 feet below the ground surface at a reasonable slope. One area of concern for the West Interceptor is the hill located on the west side of the Town, west of CR 9 and north of CR 64, along with the multiple ditch crossings at this intersection. The proposed South Interceptors are expected to require lift stations, and therefore they need more accurate modeling for future detailed design. Accordingly, it is strongly recommended to develop a preliminary design for all the proposed interceptors so that utility conflicts, road and irrigation ditch crossings, sewer depths, and similar design concerns can be understood.

All the proposed pipes for Buildout have maximum d/D values in the model below 0.65, as shown in Figures 6-2 and 6-3.

6.2 Existing System Improvements

6.2.1 5th Street Interceptor Upsizing

The existing interceptor section south of the Nano/RO Plant has several sections of pipe which may be of concern as the population grows, where d/D values are near or slightly over 0.8 (see Figures 6-2 and 6-3). Jacobs recommends periodically monitoring flow depths on this branch of the system for the need to address concerns in the future. Avoiding running the Nano/RO Plant's clean-in-place wash concurrently with peak flows could help lessen concerns. Alternatively, if the Nano/RO Plant were to stop discharging to the collection system,

Figure 6-1. Proposed Buildout Pipe Configuration

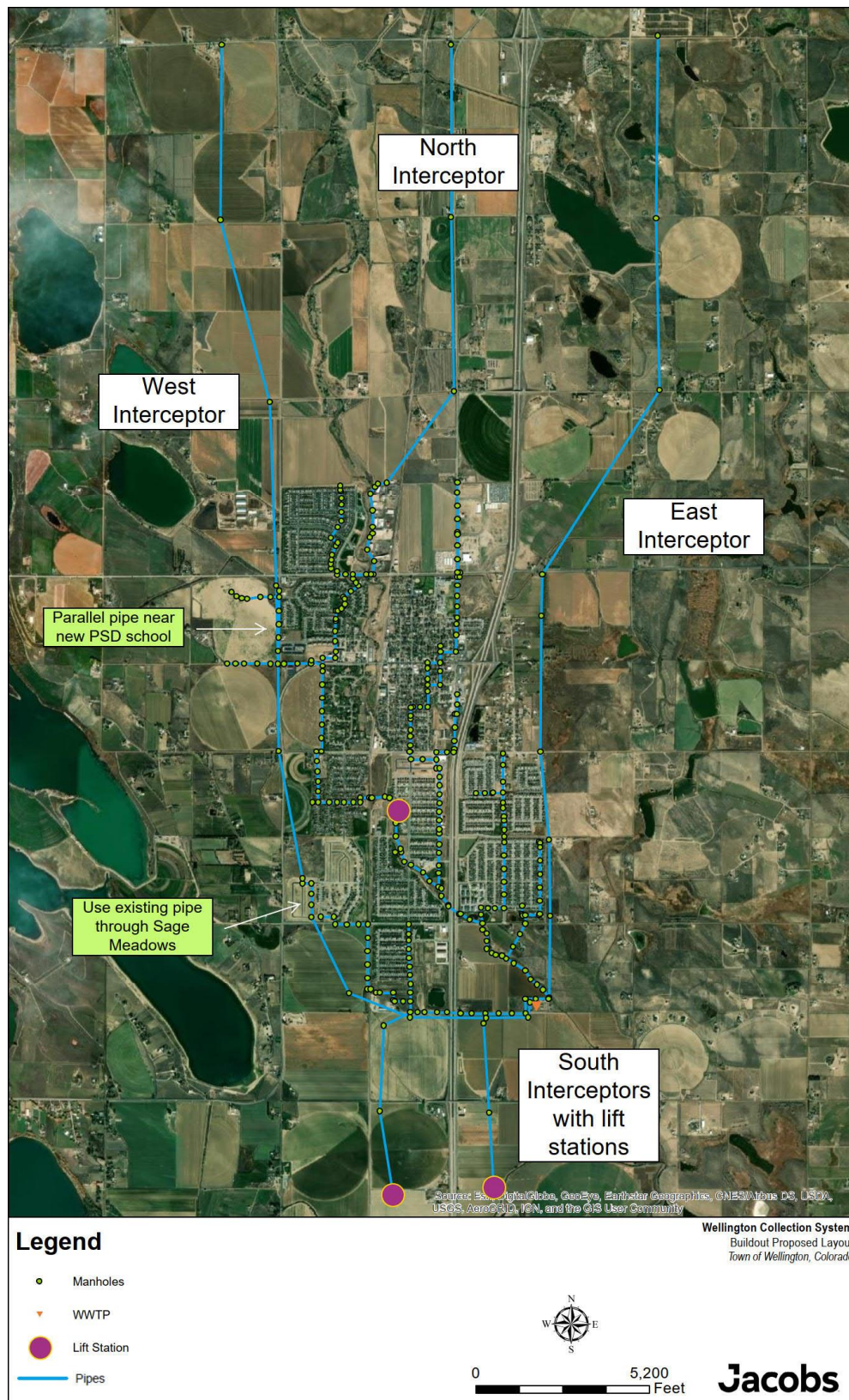


Figure 6-2. Buildout Proposed Layout with Pipes Colored by Maximum d/D

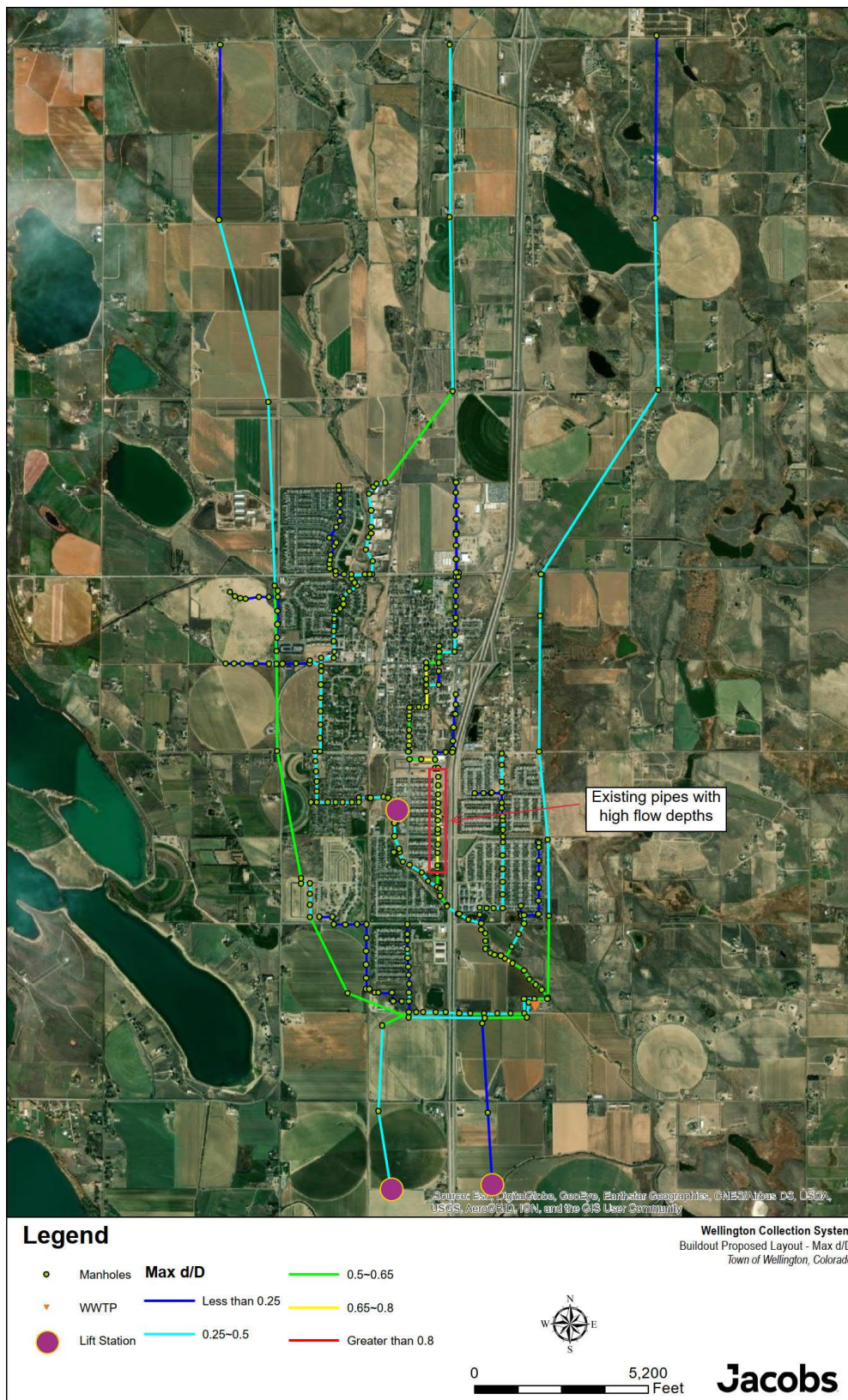


Figure 6-3. Buildout Proposed Layout with Pipes Colored by Maximum d/D, Zoom of Existing Area



it would extend the capacity of this section of pipe for growth. If additional capacity is needed in the future, the pipes between manholes M-J5-3 and M-K5-7 along 5th Street (as shown on Figure 6-3) may need to be upsized from 12-inch to 15-inch diameter, or perhaps the flows could be moved to another line to the east with an additional bore under I-25.

6.2.2 View Pointe Lift Station Replacement

As discussed in Section 3, it is recommended that upgrades be made to the View Pointe Lift Station within the next couple of years due to safety and maintenance concerns, and that the upgrade design account for a way to be able to increase its capacity up to the maximum flow that is expected to occur through to Buildout.

6.2.3 Rehabilitation and Replacement

As collection systems age, pipe conditions can worsen and allow infiltration and inflow (I/I) into the sewers as well as cause structural failures. The flow monitoring data showed very little I/I, suggesting that pipe conditions are generally good at the present time. However, it is recommended to include a program in the capital improvement plan to address rehabilitation and replacement of pipes as needs arise. An allowance of \$100,000 per year is recommended.

6.2.4 Flow Monitoring

Flow monitoring in seven locations was conducted to support model development for this master plan. It is recommended that monitoring be conducted in additional locations to evaluate conditions systemwide and that periodic monitoring be pursued to evaluate flow changes due to growth. These data can help confirm project needs and aid in planning and improving system understanding. Conducting flow monitoring with a larger number of meters less frequently tends to be more cost-effective than conducting small monitoring efforts every year (for example), due to one-time costs such as mobilization.

A summary of the recommended capital improvement projects is provided in Table 6-1, which matches Table E-1 presented in the Executive Summary. A map of the proposed interceptor projects is shown in Figure 6-4, and further details about the projects are provided in Table 6-2.

Table 6-1. Capital Improvement Plan Recommendations for Buildout

Project	Description	Estimated Cost [#]	Time Frame
East Interceptor	An interceptor along the east side of I-25, which ties into the main interceptor upstream of the plant	\$18M*	2035-2040
North Interceptor	An interceptor east of Boxelder Creek and west of I-25, which ties into the existing collection system in the northwest	\$8M*	As development occurs
West Interceptor	An interceptor to the west of Boxelder Creek which utilizes the 30" line in Sage Meadows before paralleling the existing west interceptor underneath the railroad tracks and discharges directly upstream of the wastewater treatment plant (WWTP)	\$28M*	2035-2040
South Interceptors	Two lines: one west of I-25 which discharges into the existing collection system, and one east of I-25 which discharges into the new West Interceptor	\$9M*	As development occurs
South Interceptor Lift Stations	Lift stations to pump flow from the south end of the Buildout area	\$0.5M*	As development occurs
5th Street Upsizing	Improvements to the existing section of pipe south of the downtown area which may experience excessive depths of flow with future growth	\$3.5M	By about 2040 if monitoring confirms need and concerns are not mitigated by peak flow management
View Pointe Lift Station Replacement	Replacement of the existing lift station to address safety, maintenance, and capacity concerns	\$0.5M	Within 2 years
Rehabilitation and/or replacement	Updates to address aging infrastructure and I/I if flow monitoring indicates the need	\$100,000 per year allowance	As needed
Flow Monitoring	Confirmation of project needs and planning	\$50,000-\$100,000 per monitoring season	Every 5 years, 3-month duration
2040 Temporary Improvements	Temporary pipes to convey new flows to the existing collection system prior to the need for a new interceptor are included in the interceptors to which they pertain.		As development occurs

n/a = not applicable, I/I = infiltration and inflow

* Interceptor sewer and South Interceptor lift station project costs may be funded in whole or part by development, depending upon Town policy. Costs for interceptors reflect estimate for total cost of installed interceptor.

[#] Estimated costs are in 2021 dollars.

Figure 6-4. Buildout Proposed Interceptor Projects

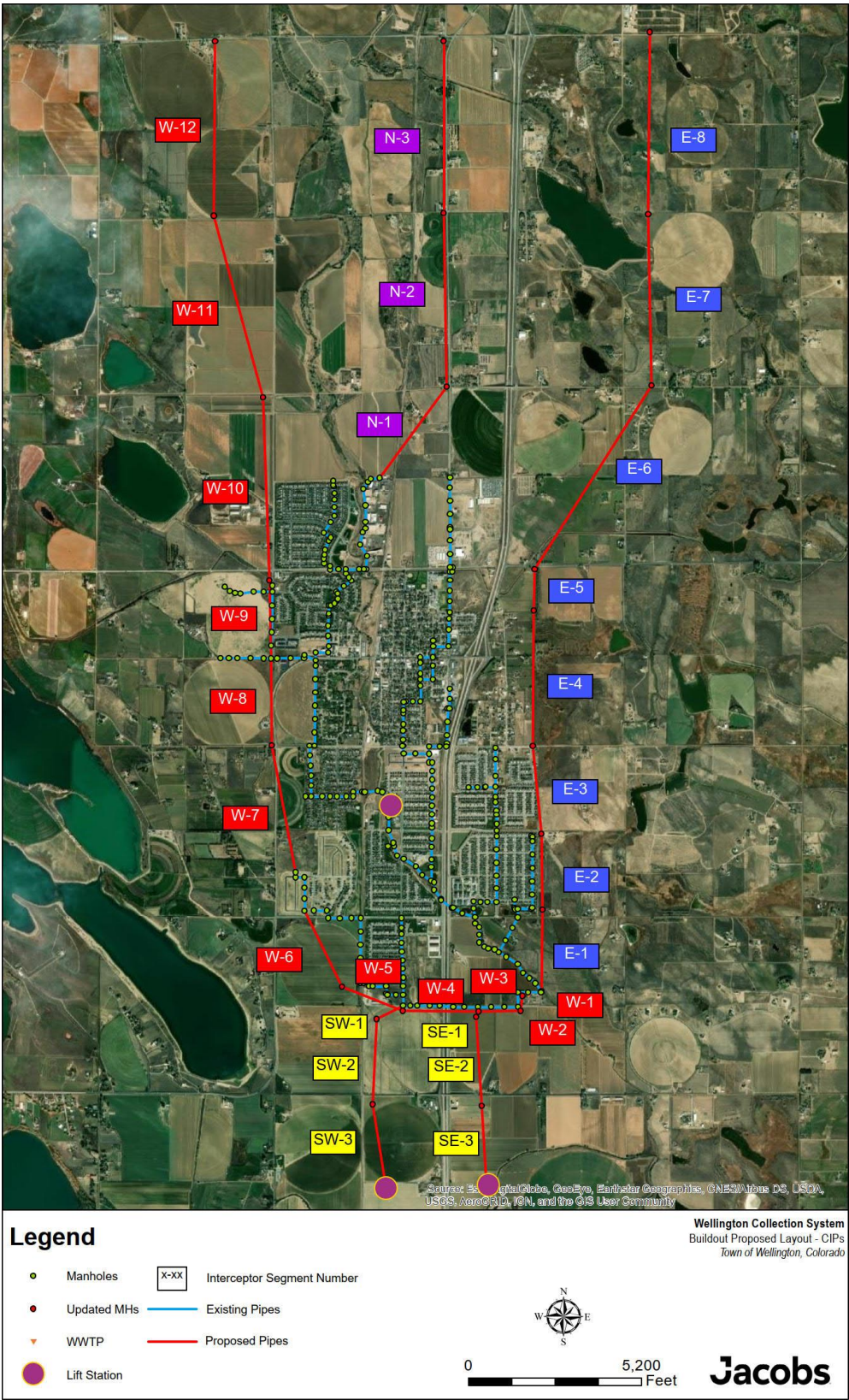


Table 6-2. Buildout CIP Proposed Interceptor Details

CIP	Description	Estimated Length (ft)	Diameter (in) ^a	Projected Buildout Flow per Segment (MGD)
E-1	East Interceptor Segment 1	2500	12	0.3
E-2	East Interceptor Segment 2	2300	12	0.3
E-3	East Interceptor Segment 3	2700	10	0.2
E-4	East Interceptor Segment 4	4100	10	0.2
E-5	East Interceptor Segment 5	1300	10	0.2
E-6	East Interceptor Segment 6	5600	8	0.1
E-7	East Interceptor Segment 7	5400	8	0.1
E-8	East Interceptor Segment 8	5100	8	0.1
N-1	North Interceptor Segment 1	3400	10	0.5
N-2	North Interceptor Segment 2	5500	10	0.3
N-3	North Interceptor Segment 3	5200	8	0.1
W-1	West Interceptor Segment 1	200	24	2.8
W-2	West Interceptor Segment 2	500	24	2.8
W-3	West Interceptor Segment 3	1300	24	2.8
W-4	West Interceptor Segment 4	2300	24	2.7
W-5	West Interceptor Segment 5	2000	18	2.7
W-6	West Interceptor Segment 6	2600	18	2.6
W-7	West Interceptor Segment 7	3900	18	2.3
W-8	West Interceptor Segment 8	2700	18	1.8
W-9	West Interceptor Segment 9	2400	15	1.7
W-10	West Interceptor Segment 10	5500	18	0.9
W-11	West Interceptor Segment 11	5700	12	0.3
W-12	West Interceptor Segment 12	5300	10	0.01
SW-1	South Interceptor/West Segment 1	900	12	0.7
SW-2	South Interceptor/West Segment 2	2600	12	0.3
SW-3	South Interceptor/West Segment 3	2600	10	0.1
SE-1	South Interceptor/East Segment 1	200	12	0.1
SE-2	South Interceptor/East Segment 2	2700	10	0.1
SE-3	South Interceptor/East Segment 3	2400	10	0.01

7. 2040 Recommendations

The distribution of growth selected by the Town's planners (see Figure 7-1) fortuitously minimizes improvements required prior to approximately 2040 (with a population of about 25,000). From now until 2040, this planned growth is more pronounced in the southern and eastern portions of the planned service area. From 2040 through to Buildout, most of the residential growth is expected to occur in the west region; the north region is expected to be predominantly industrial growth. Civic and commercial growth is expected to occur in multiple areas.

Once the pipe sizes needed for Buildout were determined (see Section 6), the 2040 model was developed to determine how much growth the existing collection system could accommodate before significant improvement projects and major railroad and/or interstate crossings are required. Overall, the growth modeled in each area by 2040, expressed as a percentage of full Buildout growth, is as follows:

- Existing Town areas – 100%
- South region – 90%
- East region – 80%
- North region – 60%
- West region – 35%

These percentages represent one scenario for growth which can be accommodated by the existing collection system while maintaining a flow depth to diameter (d/D) ratio below 0.8 in the existing system and below 0.65 in the proposed sections of interceptor outside of the existing system. Initial growth occurring predominantly in the existing service area, the south region, and the east region would postpone the need for a West Interceptor, which would cross both a railroad and I-25, to parallel the existing piping. With limited initial growth in the north, the existing system likely can receive flows from each area and not exceed a d/D of 0.8. However, if there is significant industrial growth over the next 20 years, flow will need to be carried across I-25 in a new section of sewer prior to flowing to the WWTP, or other interceptors will need to be completed to relieve the capacity through the existing sewer system. New interceptors will only be required in specific areas if development occurs there. Some temporary piping to connect new interceptors to the existing system is expected to be needed until the rest of the proposed interceptors for Buildout are constructed. The layout of the interceptors modeled for this scenario is shown in Figure 7-2 and summarized in Table 7-1. Portions of the proposed interceptors already discussed in Section 6 are shown, but they are only needed to meet the requirements of development as it occurs. Temporary connections to the existing system are shown with orange labels in Figure 7-2, in areas that will be served by the remaining portions of the East and West Interceptors once they are constructed. Figure 7-3 shows the existing and proposed pipes for the 2040 scenario colored by the d/D values, with a close-up of the existing area in Figure 7-4.

In the area of the future Poudre School District high school, one section of pipe is shown as having a maximum d/D greater than 0.8 (Figures 7-3 and 7-4). Upon review of other model metrics, as well as the depth over time modeled in this pipe, this result appears to be an inconsistency in the model results that is not believed to be accurate. InfoSWMM calculates three metrics that are of use when evaluating capacity. The metric that has been used on all the figures herein showing modeled capacity, and which was recommended by Innovyze for this purpose³, is based on the maximum depth in the middle of the pipe (by length). Another metric (referred to as "Surcharged d/D ") is based on the average of the depths at the upstream and downstream ends of the pipe. A third metric (called "Capacity") is based on the cross-sectional area of the flow compared to the cross-sectional area of the pipe. It can be useful to review all these metrics when assessing capacity, and the three metrics generally result in numbers that are slightly different but close enough to one another that the conclusions are consistent. However, in a couple of cases there is a discrepancy in one of the metrics. For this reason, data for

³ Personal communication from Robert Dickinson (Innovyze) to Perrin Niemann (Jacobs) on February 4, 2020.

the second and third metrics were exported from model results to determine whether any other pipes had results that varied significantly from the data used for the figures. There were no other pipes that showed significant differences between the three metrics.

In the section of pipe through and south of downtown, there were several pipes which were borderline for having a d/D above 0.8, in the same area as that discussed already in Section 6. Based on the model results, these pipes are not expected to be of concern prior to buildout of the existing downtown area (which occurs by 2040 in the modeled scenarios) if the Nano/RO Plant does not perform a clean-in-place wash during peak hour flows, and growth occurs in a manner similar to the assumptions use in the model scenarios. Depending on the sensitivity of connections to this pipe and the Town's risk tolerance, it may be acceptable to allow this stretch of pipe to run fuller on a regular basis. It is recommended that future metering be conducted periodically along this stretch of pipe to track how significantly growth is impacting the area.

Table 7-1. 2040 CIP Proposed Interceptor Details

CIP	Description	Estimated Length (ft)	Diameter (in) ^a	Projected Flow per Segment in 2040 (MGD)
E-T-1	East Interceptor Temporary Connection 1	400	8	0.1
E-T-2	East Interceptor Temporary Connection 2	400	10	0.1
E-T-3	East Interceptor Temporary Connection 3	1200	10	0.1
E-4	East Interceptor Segment 4	4100	10	0.1
E-5	East Interceptor Segment 5	1300	10	0.1
E-6	East Interceptor Segment 6	5600	8	0.1
E-7	East Interceptor Segment 7	5400	8	0.05
E-8	East Interceptor Segment 8	5100	8	0.03
N-1	North Interceptor Segment 1	3400	10	0.3
N-2	North Interceptor Segment 2	5500	10	0.2
N-3	North Interceptor Segment 3	5200	8	0.02
W-T-1	West Interceptor Temporary Connection 1	600	10	0.1
W-T-2	West Interceptor Temporary Connection 2	1200	10	0.2
W-T-3	West Interceptor Temporary Connection 3	200	18	0.3
W-T-4	West Interceptor Temporary Connection 4	5300	12	0.3
W-11	West Interceptor Segment 11	5700	12	0.1
W-12	West Interceptor Segment 12	5300	10	0.01
SW-1	South Interceptor/West Segment 1	900	12	0.6
SW-2	South Interceptor/West Segment 2	2600	12	0.2
SW-3	South Interceptor/West Segment 3	2600	10	0.1
SE-T-1	South Interceptor/East Temporary Connection 1	300	10	0.04
SE-2	South Interceptor/East Segment 2	2700	10	0.03
SE-3	South Interceptor/East Segment 3	2400	10	0.01

Figure 7-1. Portion of Buildout Growth Modeled for 2040

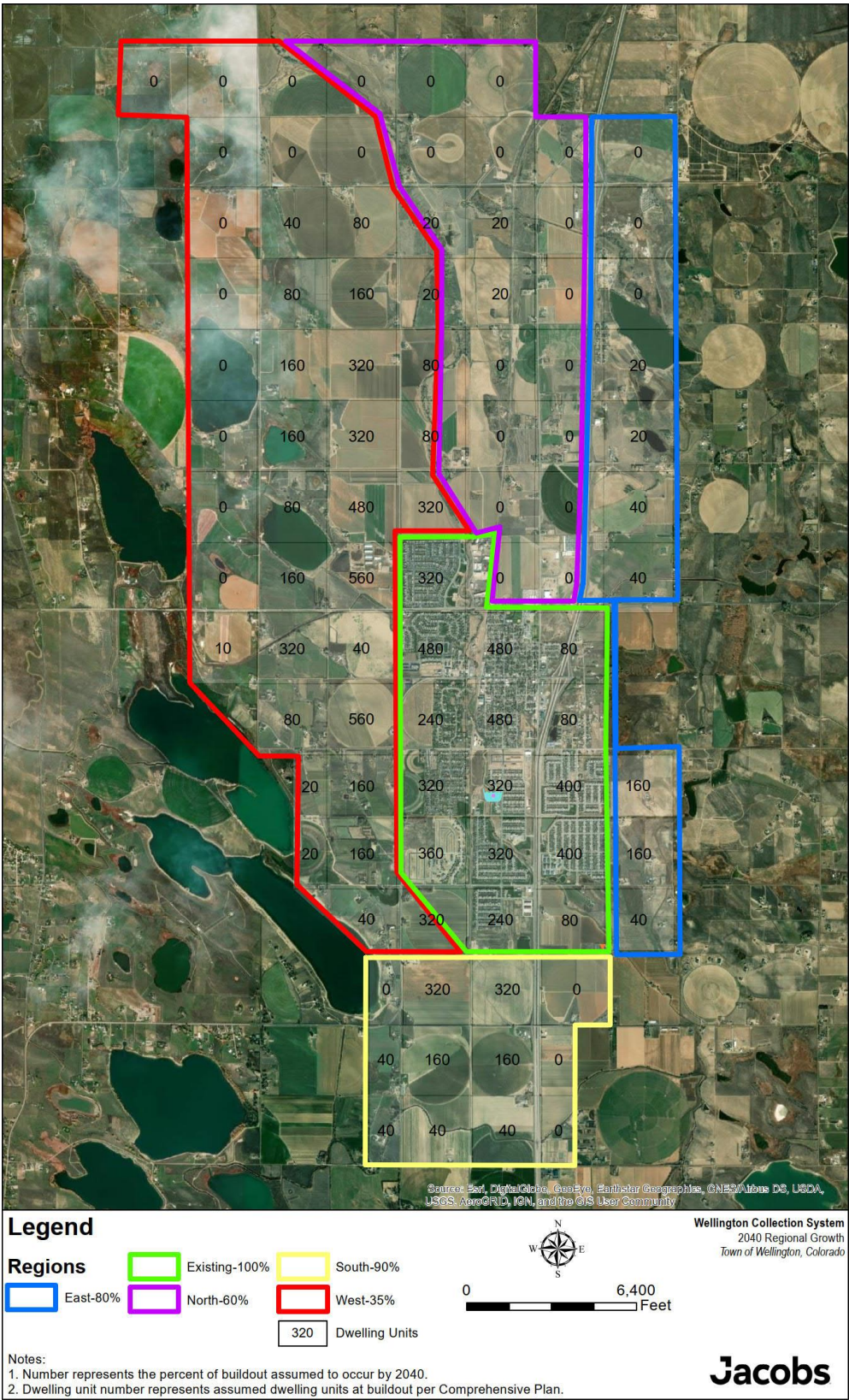


Figure 7-2. 2040 Proposed Collection System Layout, with Temporary Connections

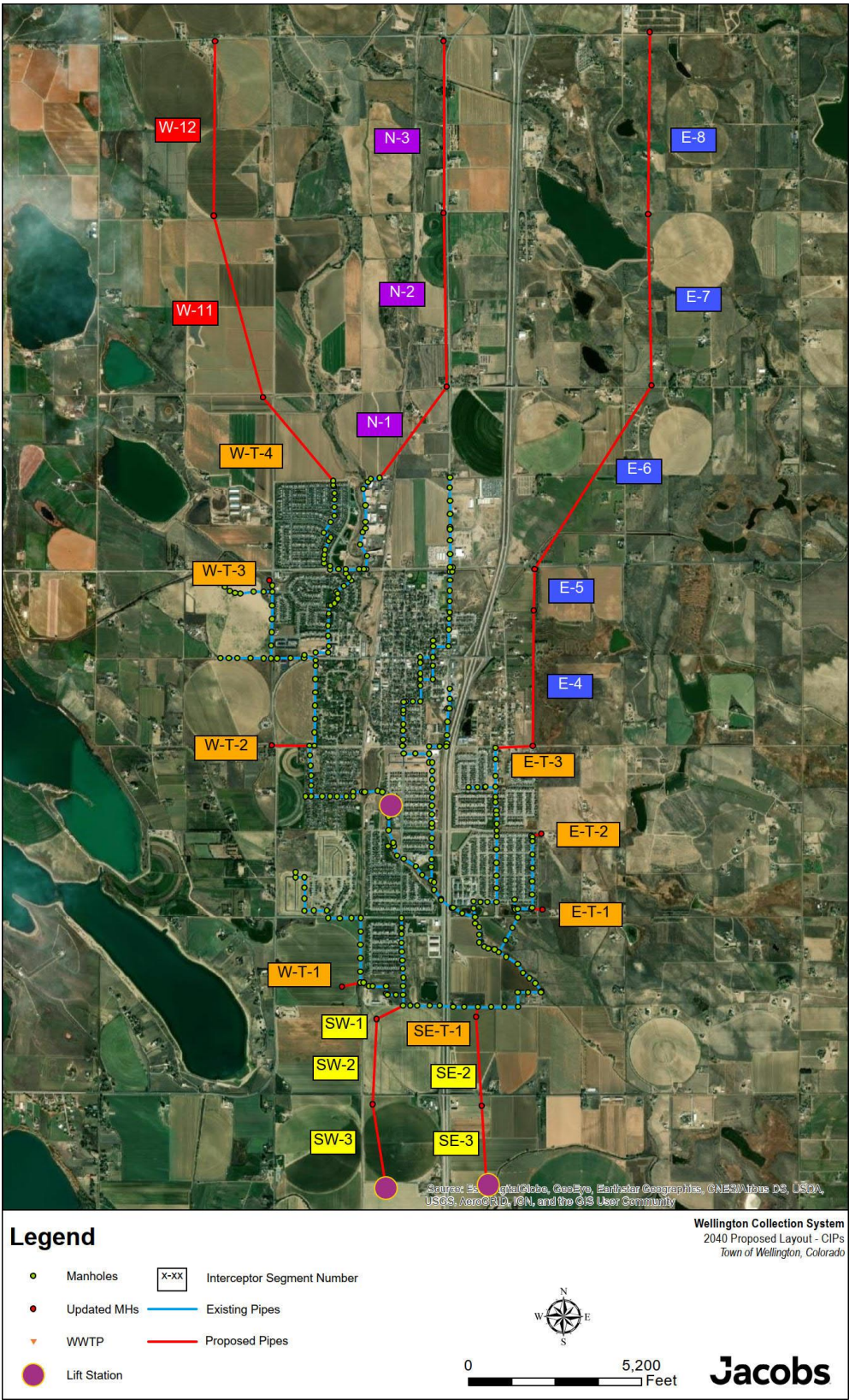


Figure 7-3. 2040 Proposed Layout with Pipes Colored by Maximum d/D

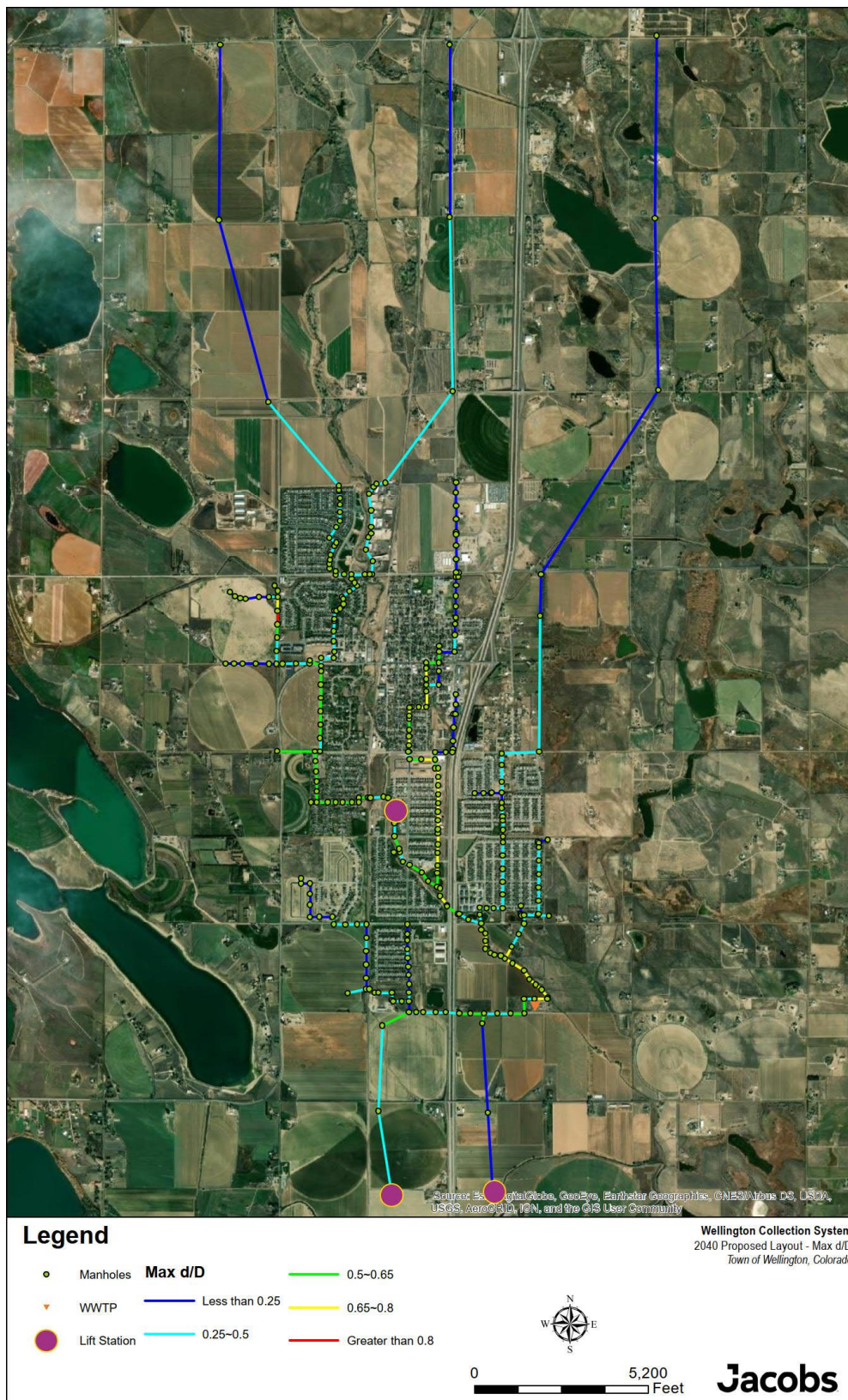
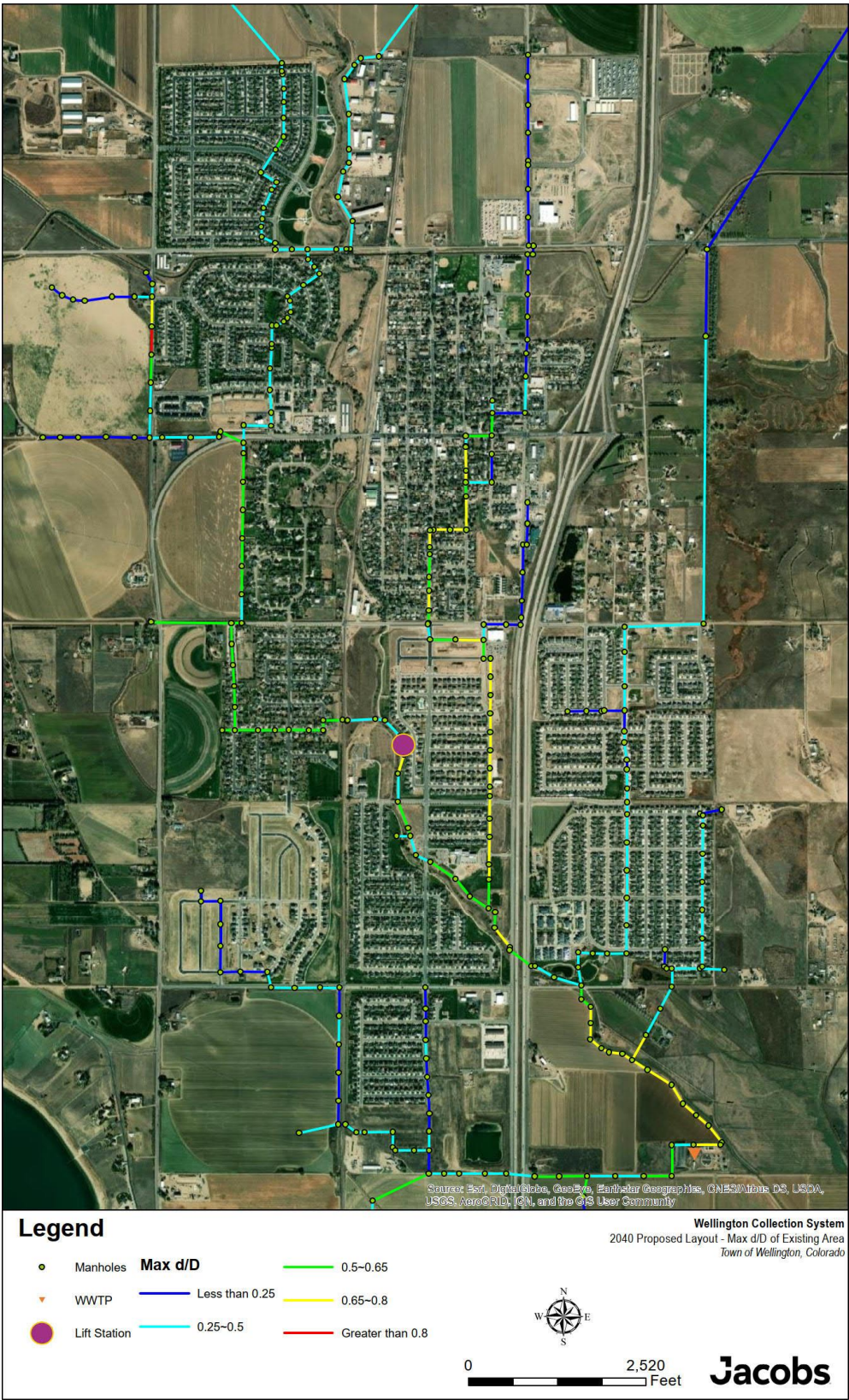


Figure 7-4. 2040 Proposed Layout with Pipes Colored by Maximum d/D, Zoom of Existing Area



8. Summary

A hydraulic model of the Town's collection system was developed, including scenarios for the existing system, a Year 2040 scenario with a population of about 25,000, and a Buildout scenario with a population of 35,500. The existing system model was validated with flow meter data. Interceptors needed to convey projected flows for the Buildout scenario were sized, and the ability of the existing system to accommodate new flows up to about Year 2040 was assessed. Recommended capital improvement projects for Buildout are identified in Tables 6-1 and 6-2 and Figure 6-4. These projects include the following:

- East, North, West, and South Interceptors to convey flows from new development and provide additional capacity to send flows to the WWTP.
- Lift stations on the South Interceptors due to lower ground elevations in the area.
- Potential upsizing of existing pipes along 5th Street due to model results showing excessive depth of flow after some growth occurs.
- Improvements at the View Pointe Lift Station in the next few years due to safety, maintenance, and capacity concerns.
- Rehabilitation and/or replacement of aging infrastructure if flow monitoring indicates excessive inflow and infiltration and/or structural concerns arise.
- Flow metering to be conducted periodically throughout the collection system to assess the impacts of growth and identify the best time to build new interceptors. It will be important to monitor flows occasionally in the pipes south of downtown along 5th Street where model results indicate pipes may flow deeper than ideal, and where changes to the Nanofiltration/RO Plant's discharges will be observed (as shown on Figure 6-3).

Model results suggest that the existing collection system can accommodate growth until around 2040 with a population of 25,000, if the locations and densities of growth are similar to those modeled. Only the portions of the proposed interceptors needed for development will be required by that time, along with some temporary connections to convey new flows to the existing system, as summarized in Table 7-1 and Figure 7-2.

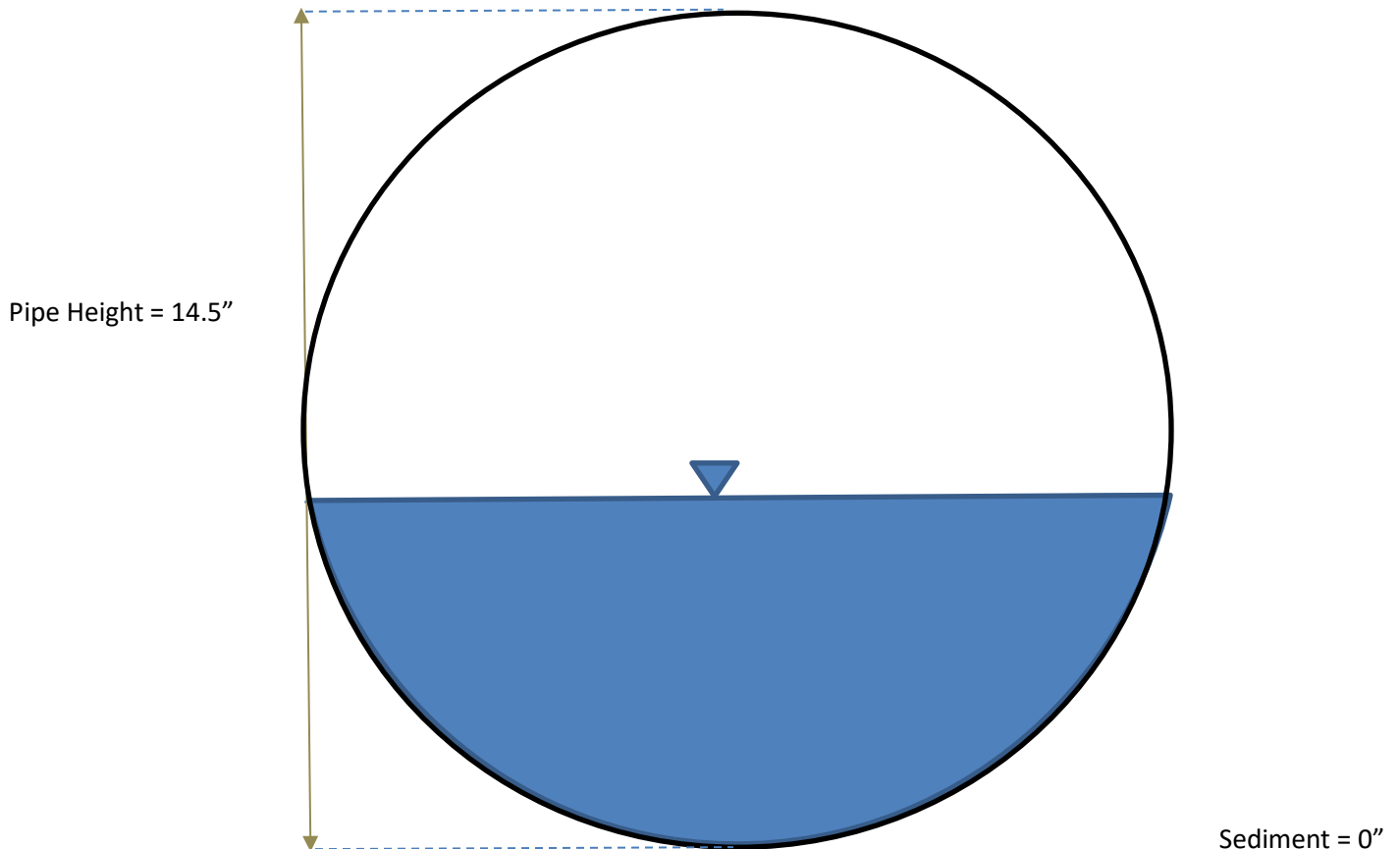
It is strongly recommended to conduct a preliminary design (30% level) for the future interceptors to better understand alignments, pipe depths, utility conflicts, road crossings, and similar concerns, so that as development occurs the new interceptor sections can be constructed in a manner that ensures the future interceptors will function correctly.

Appendix A. Hach Flow Meter Data Report

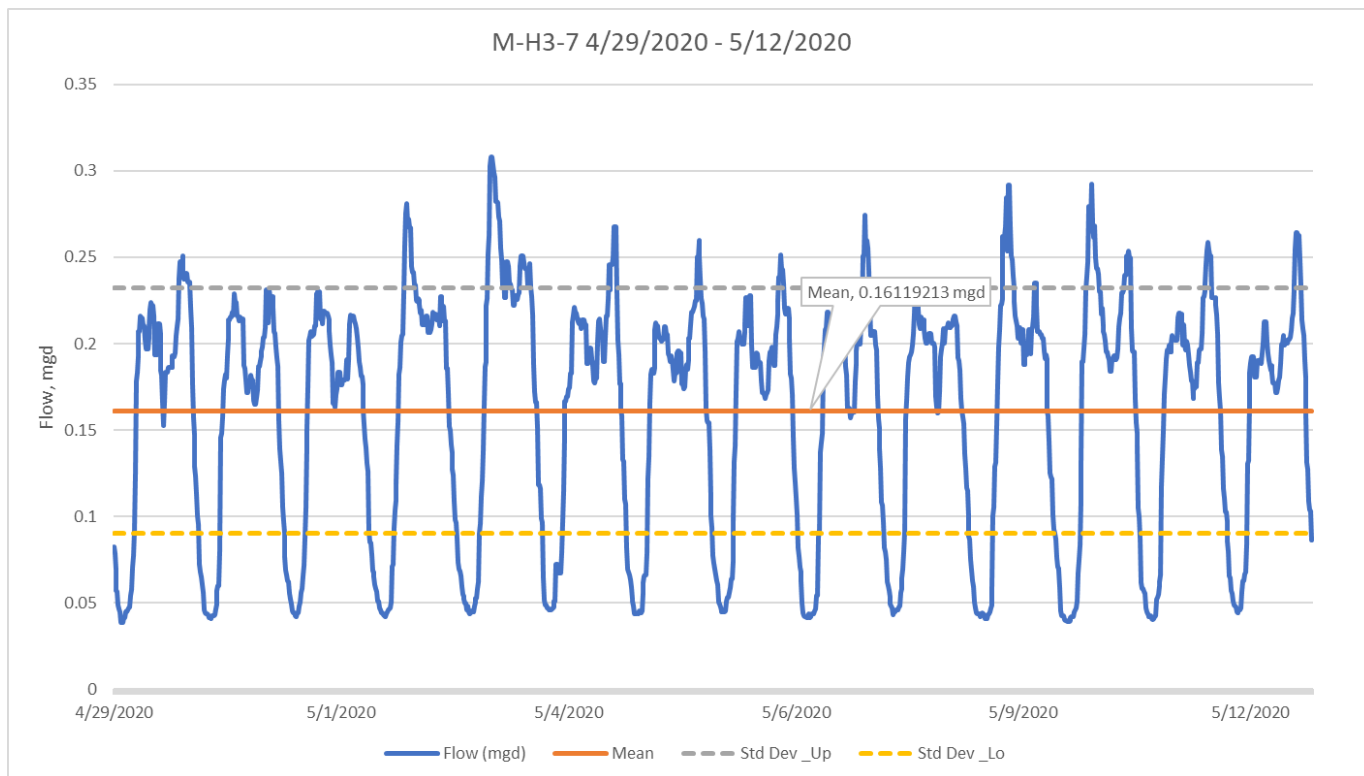
DDS Site - Data QAQC Report

Project Name: Wellington	Client: Town of Wellington	Date: 5/14/2020
Maintenance Hole Name: M-H3-7		
Street Address/Location: Cleveland Ave		
Data Time Span: 4/29/2020 0:00 – 5/12/2020 23:45		
Pipe Shape: <i>Circular</i>	Pipe ID/Material: 14.5" <i>PVC</i>	Silt: 0"
Estimated Manning's n: 0.01		
Estimated Slope: 0.0092 ft/ft		
QAQC Analysis: Emily Steele, P.E.		
Data shows high hydraulic correlation and minimal noise. Sensor is functioning well.		

Pipe Schematic

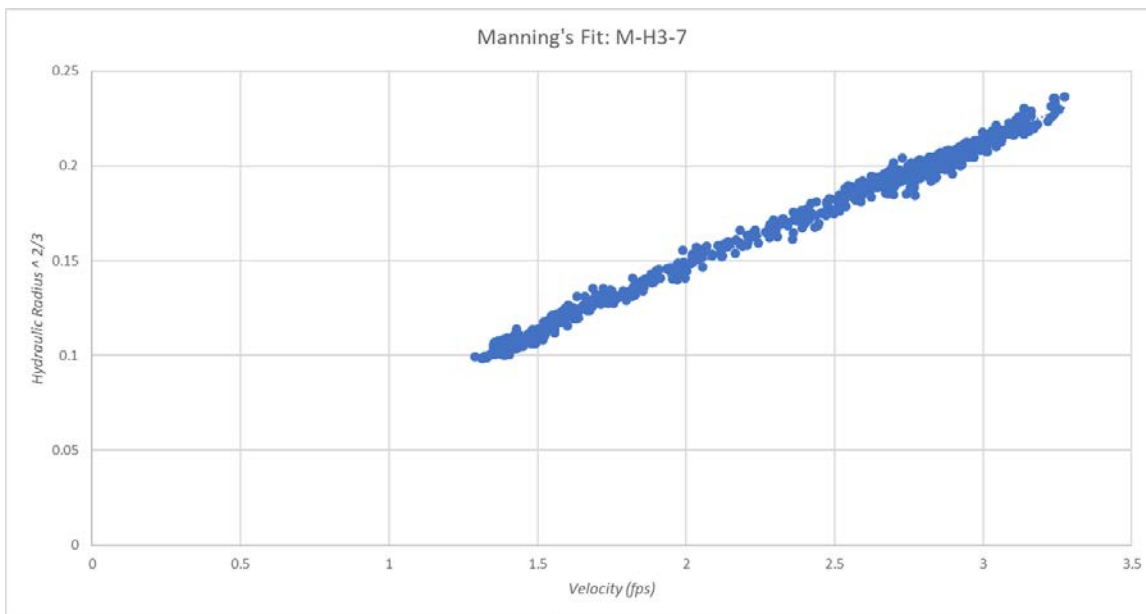


Signal to Noise Ratio

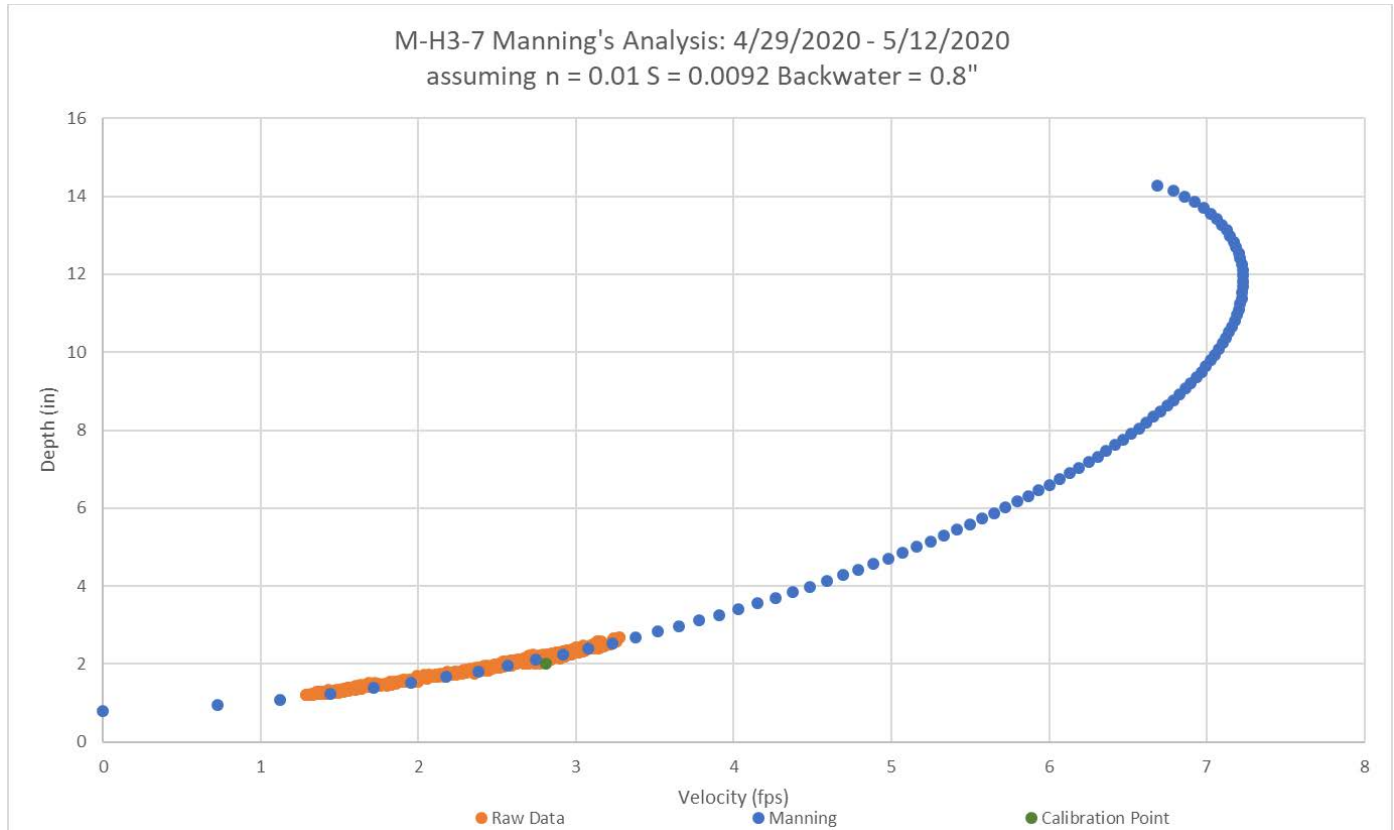


SNR = 269 Indicates excellent signal and minimal noise.

Manning's Analysis



NS = 0.98 and $R^2 = 0.99$. Excellent manning's fit.



Within minimum recommended slope range for pipe size. Indicates ~0.8" of backwater in the line.

Average Daily Flows

Date	Level, in	Velocity, fps	Flow, MGD
4/29/2020	1.97	2.47	0.16
4/30/2020	1.94	2.43	0.15
5/1/2020	1.96	2.44	0.15
5/2/2020	1.97	2.46	0.16
5/3/2020	2.04	2.53	0.17
5/4/2020	1.98	2.49	0.16
5/5/2020	1.96	2.47	0.15
5/6/2020	1.96	2.46	0.15
5/7/2020	1.95	2.45	0.15
5/8/2020	1.96	2.46	0.15
5/9/2020	1.95	2.44	0.15
5/10/2020	1.97	2.46	0.16
5/11/2020	1.95	2.48	0.16
5/12/2020	1.96	2.48	0.15

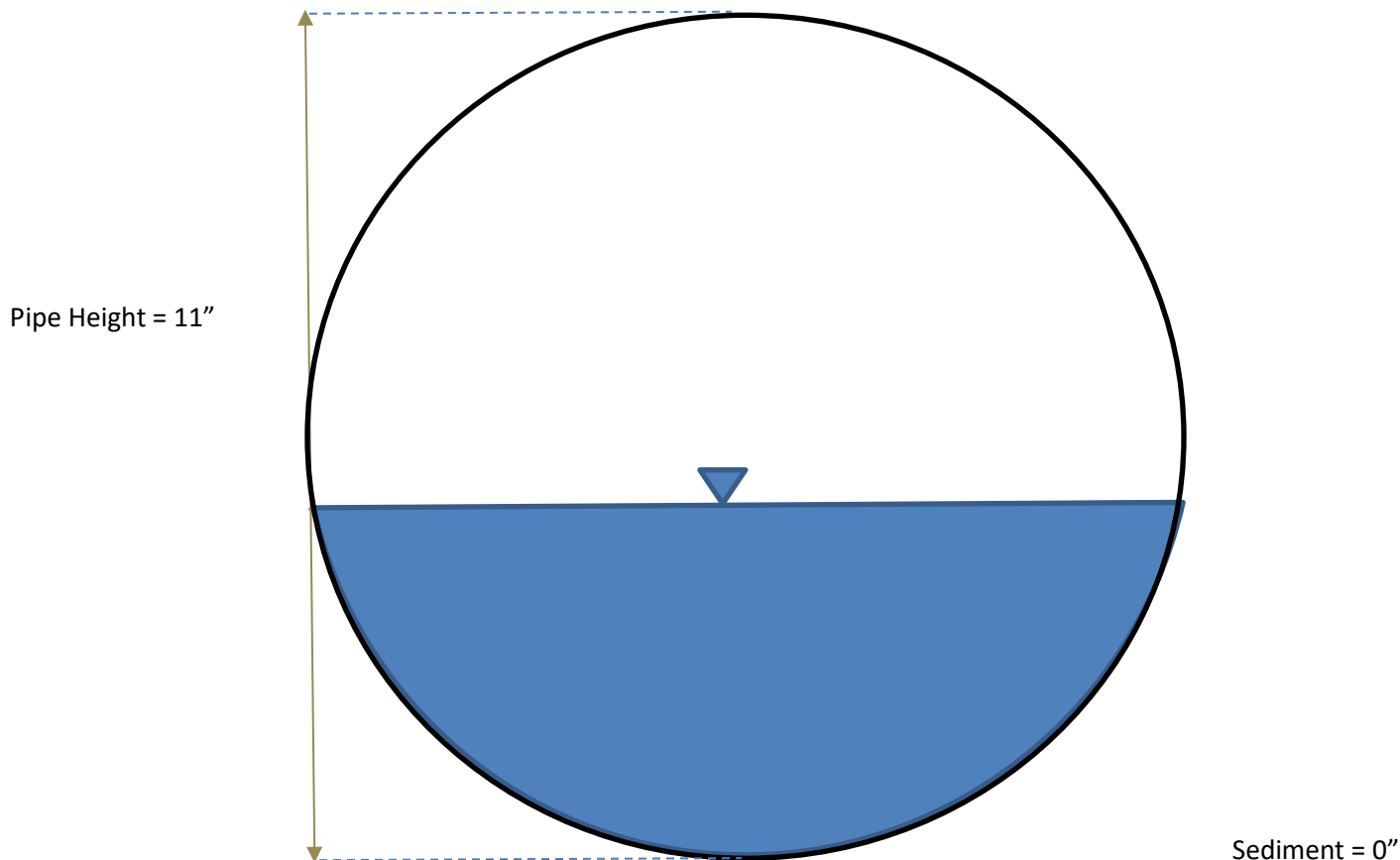
DDS Site - Data QAQC Report

Project Name: Wellington	Client: Town of Wellington	Date: 5/28/2020
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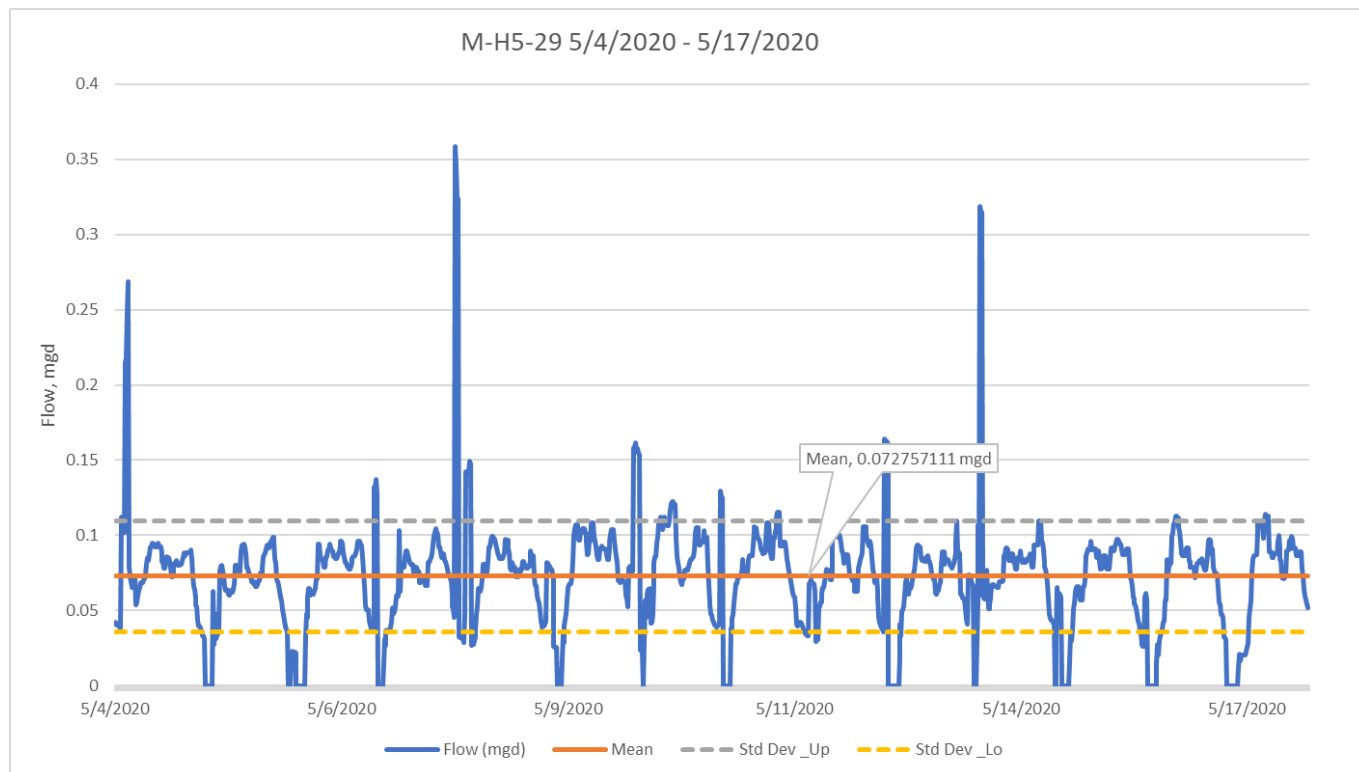
Maintenance Hole Name: M-H5-29		
Street Address/Location: 8008 4 th Street		
Data Time Span: 5/4/2020 0:00 – 5/17/2020 23:45		
Pipe Shape: <i>Circular</i>	Pipe ID/Material: 11" <i>PVC</i>	Silt: 0"

Estimated Manning's n: 0.014	Estimated Slope: 0.0025 ft/ft
QAQC Analysis: Emily Steele, P.E.	
Significant noise is observed in the velocity readings. Quality parameters should be adjusted.	

Pipe Schematic

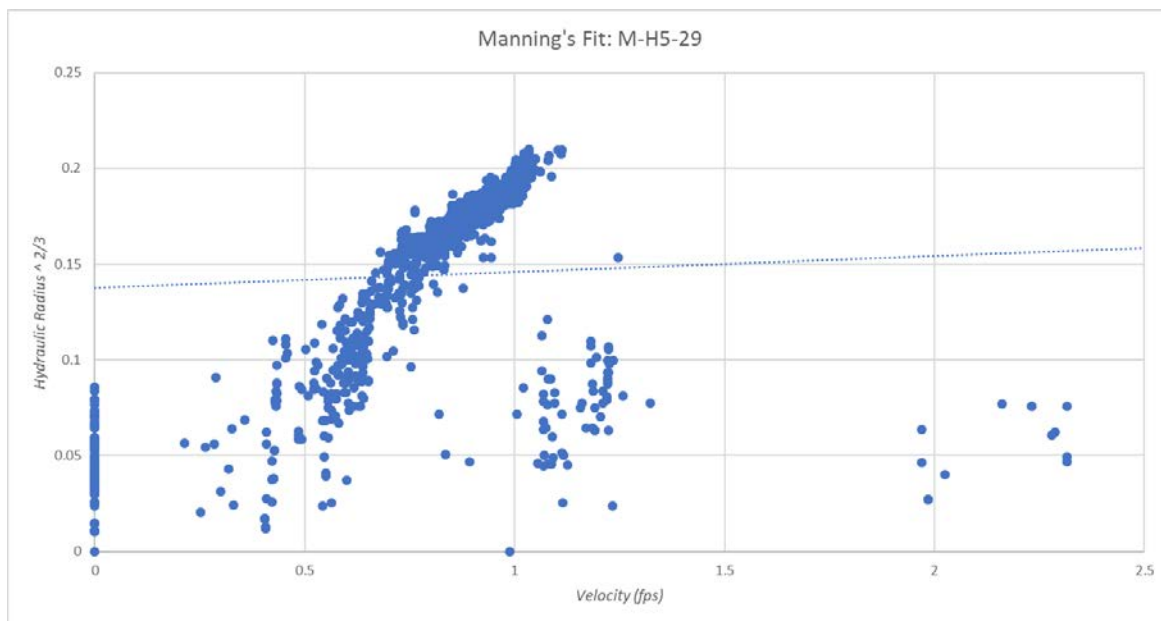


Signal to Noise Ratio

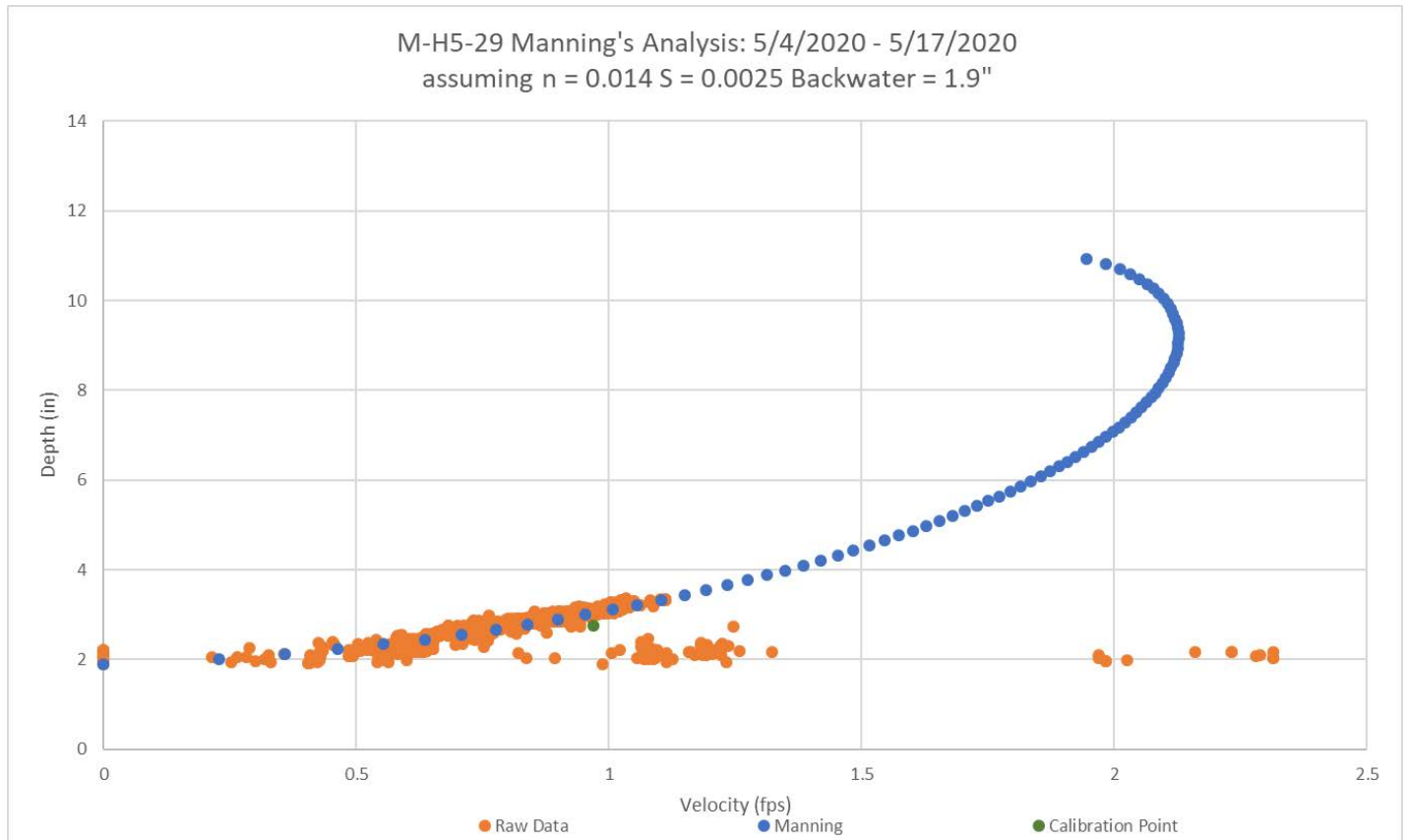


SNR = 3.7 Indicates poor signal and significant noise.

Manning's Analysis



NS < 0 and $R^2 = 0.01$. Good manning's fit, statistics skewed by velocity noise.



Equal to minimum recommended slope for pipe size. Indicates ~1.9" of backwater in the line.

Average Daily Flows

Date	Level, in	Velocity, fps	Flow, MGD
5/4/2020	2.72	1.09	0.08
5/5/2020	2.66	0.74	0.06
5/6/2020	2.69	0.70	0.06
5/7/2020	2.68	0.87	0.07
5/8/2020	2.73	1.09	0.08
5/9/2020	2.81	0.83	0.07
5/10/2020	2.78	1.05	0.08
5/11/2020	2.77	0.85	0.07
5/12/2020	2.72	0.84	0.07
5/13/2020	2.68	0.82	0.06
5/14/2020	2.69	1.04	0.08
5/15/2020	2.71	0.75	0.06
5/16/2020	2.71	0.73	0.06
5/17/2020	2.72	0.70	0.06

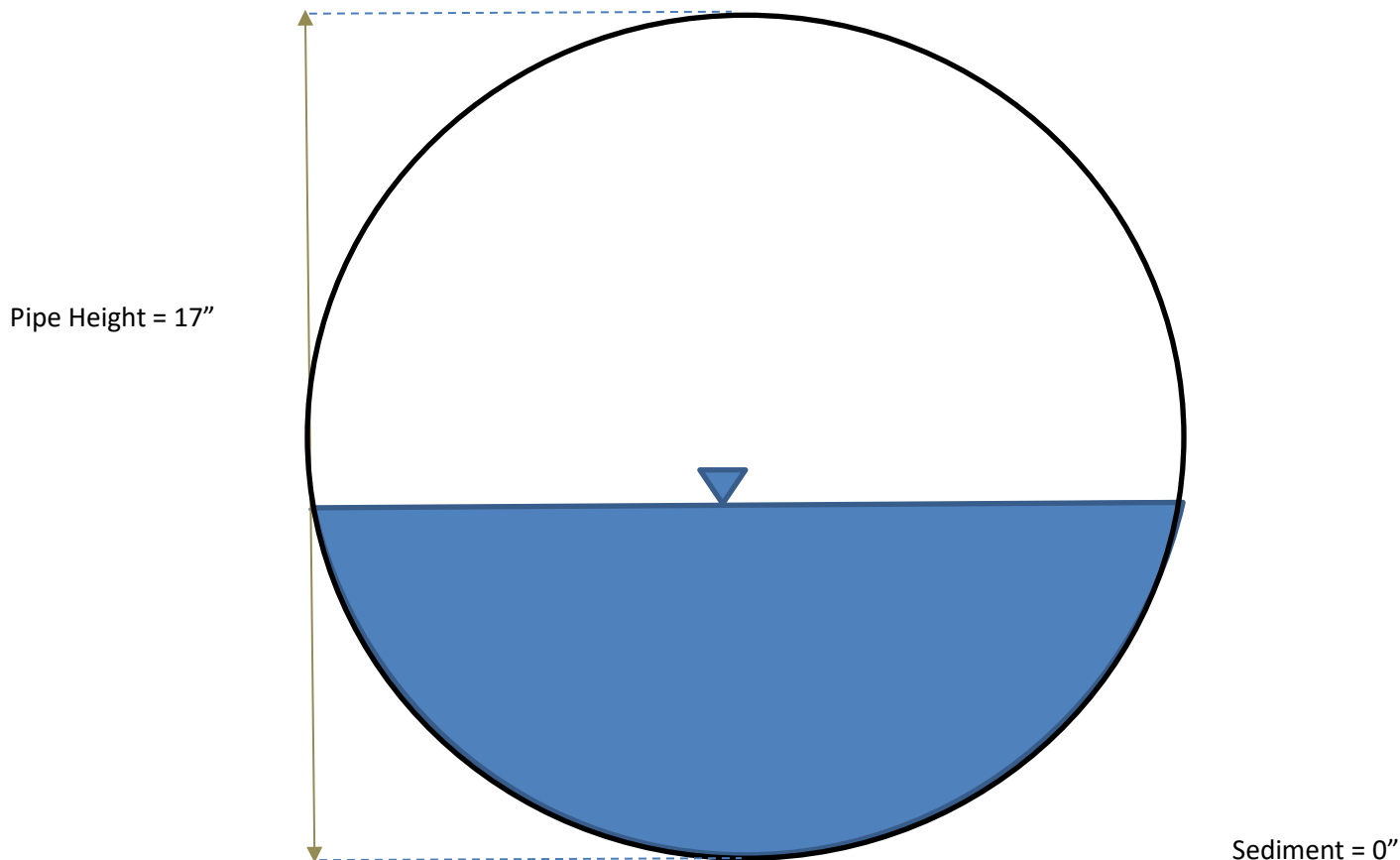
DDS Site - Data QAQC Report

Project Name: Wellington	Client: Town of Wellington	Date: 5/15/2020
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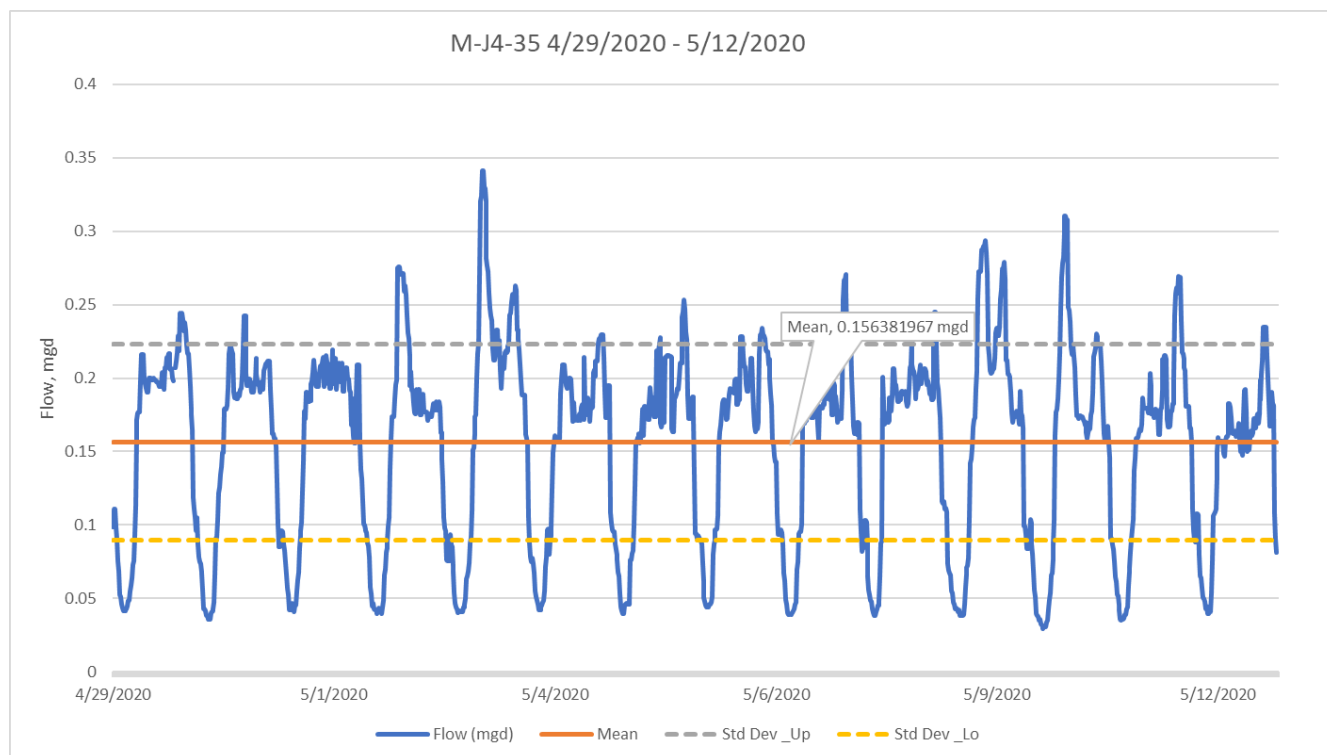
Maintenance Hole Name: M-J4-35		
Street Address/Location: North of Ronald Reagan Ave / Along Bike Path		
Data Time Span: 4/29/2020 0:00 – 5/12/2020 23:45		
Pipe Shape: <i>Circular</i>	Pipe ID/Material: 17" <i>PVC</i>	Silt: 0"

Estimated Manning's n: 0.01	Estimated Slope: 0.0047 ft/ft
QAQC Analysis: Emily Steele, P.E.	
Data shows high hydraulic correlation and minimal noise. Sensor is functioning well.	

Pipe Schematic

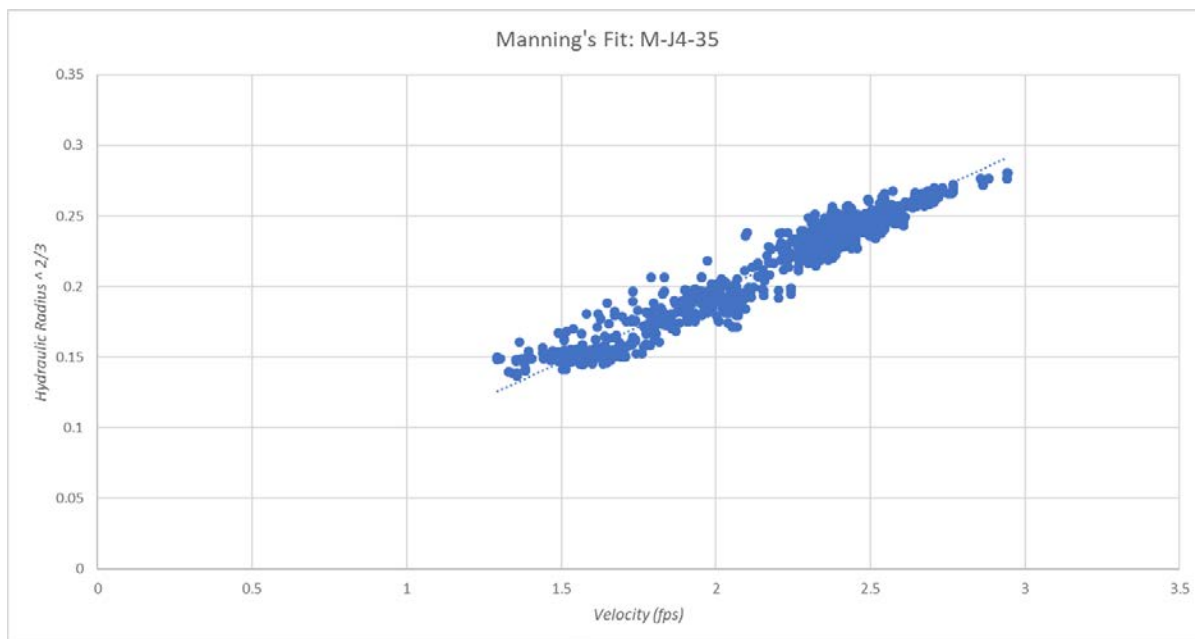


Signal to Noise Ratio

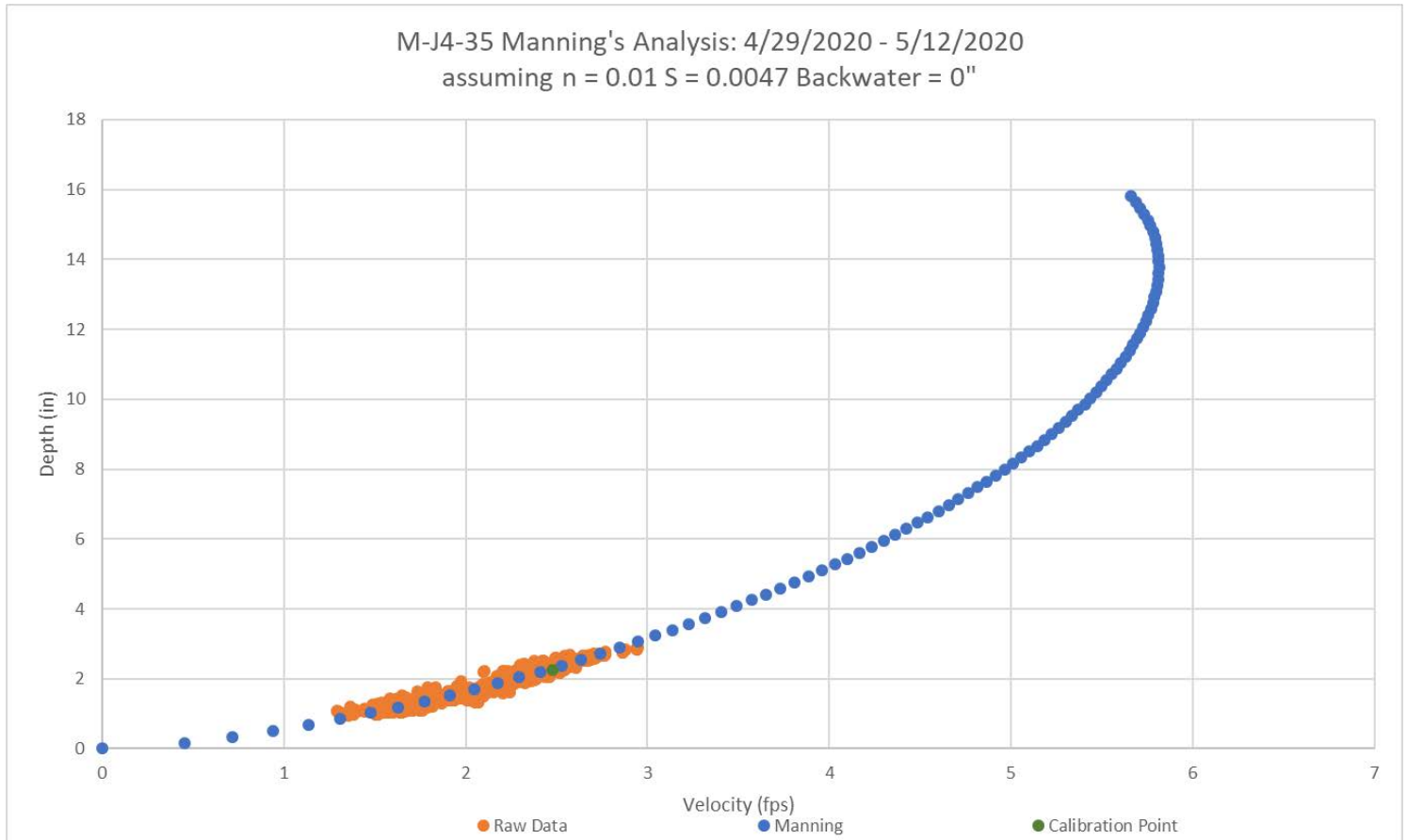


SNR = 79.2 Indicates excellent signal and minimal noise.

Manning's Analysis



NS = 0.90 and $R^2 = 0.94$. Excellent manning's fit.



Within minimum recommended slope range for pipe size. Indicates ~0" of backwater in the line.

Average Daily Flows

Date	Level, in	Velocity, fps	Flow, MGD
4/29/2020	2.03	2.22	0.16
4/30/2020	1.96	2.19	0.15
5/1/2020	1.99	2.23	0.15
5/2/2020	1.93	2.2	0.15
5/3/2020	2.05	2.28	0.17
5/4/2020	1.95	2.22	0.15
5/5/2020	1.98	2.21	0.15
5/6/2020	1.99	2.22	0.15
5/7/2020	1.95	2.20	0.15
5/8/2020	2.00	2.22	0.15
5/9/2020	1.98	2.23	0.16
5/10/2020	1.94	2.16	0.15
5/11/2020	1.93	2.21	0.15
5/12/2020	1.89	2.19	0.14

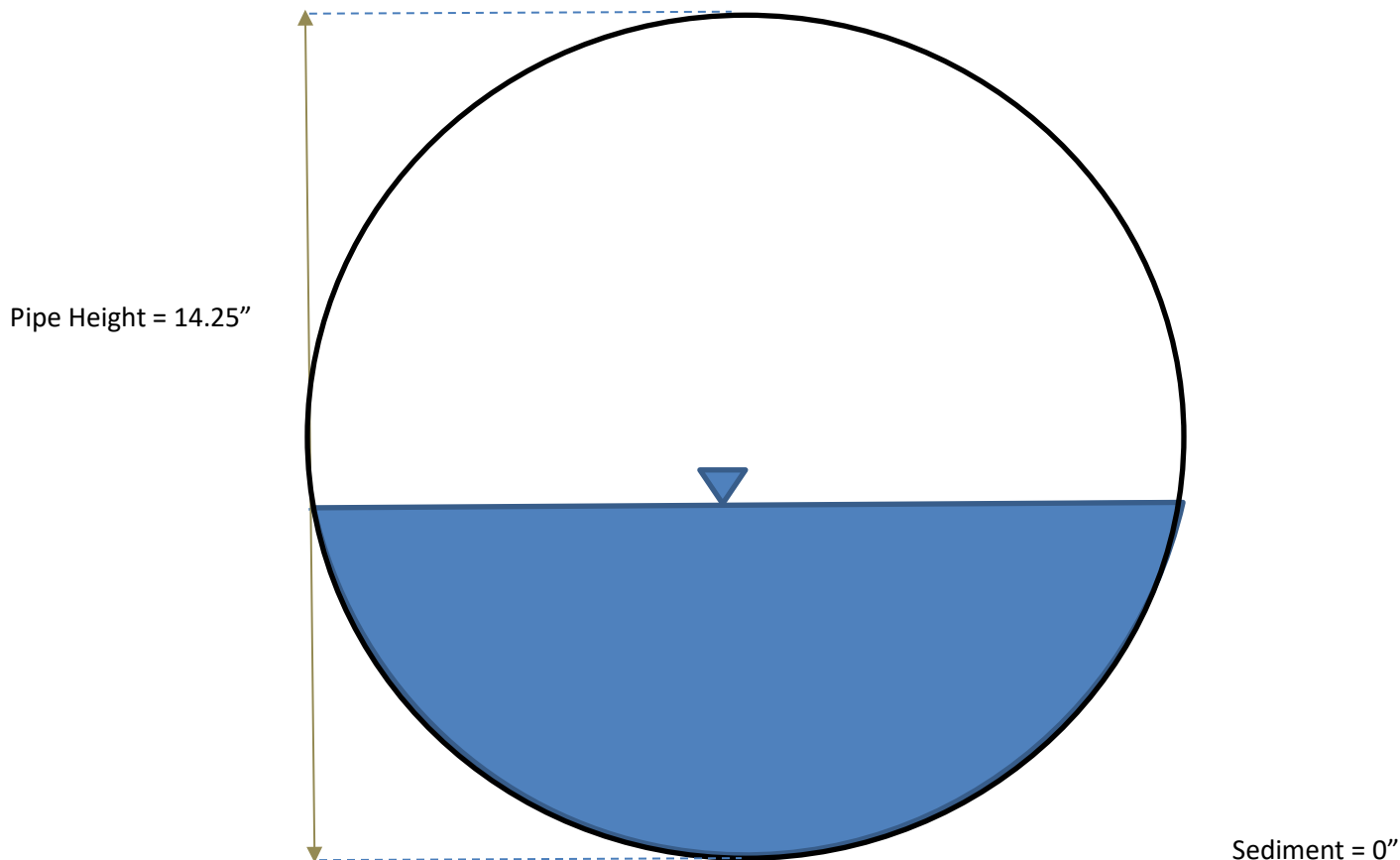
DDS Site - Data QAQC Report

Project Name: Wellington	Client: Town of Wellington	Date: 5/15/2020
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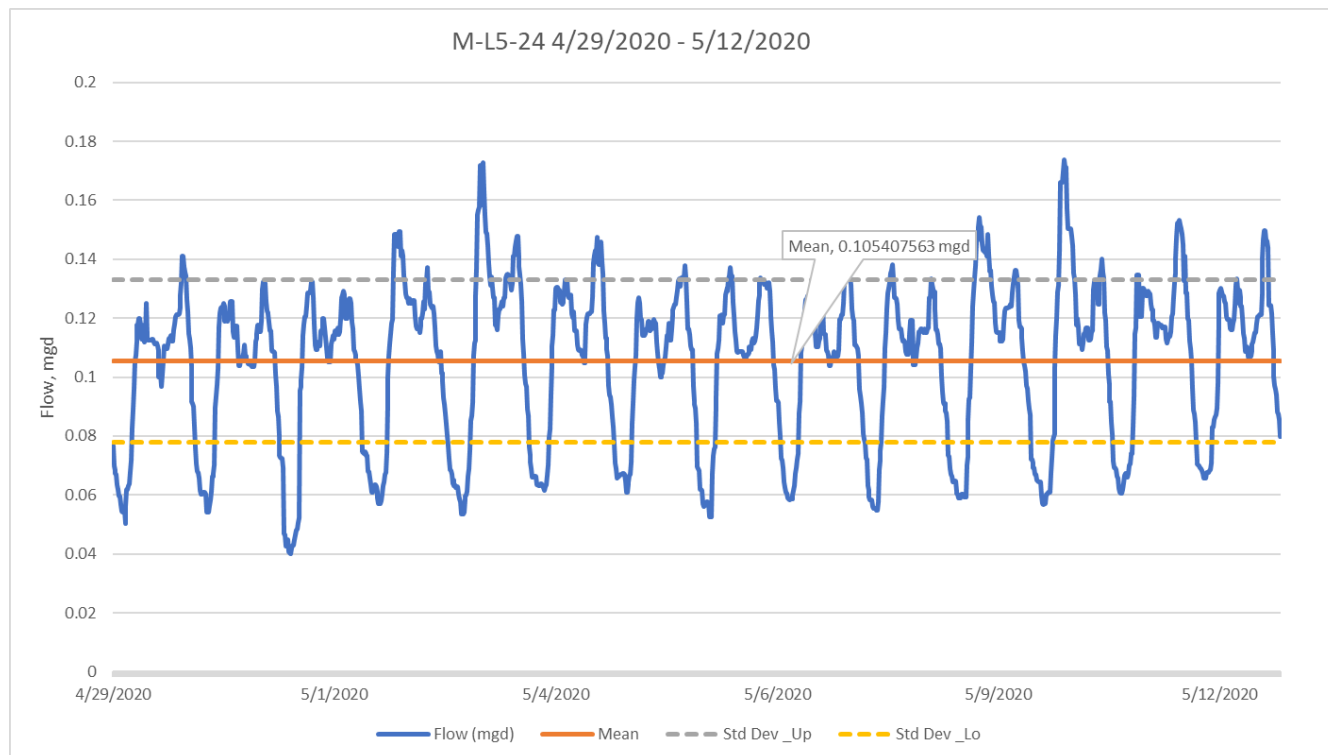
Maintenance Hole Name: M-L5-24		
Street Address/Location: Wellville Park		
Data Time Span: 4/29/2020 0:00 – 5/12/2020 23:45		
Pipe Shape: <i>Circular</i>	Pipe ID/Material: 14.25" <i>PVC</i>	Silt: 0"

Estimated Manning's n: 0.01	Estimated Slope: 0.0076 ft/ft
QAQC Analysis: Emily Steele, P.E.	
Data shows high hydraulic correlation and minimal noise. Sensor is functioning well.	

Pipe Schematic

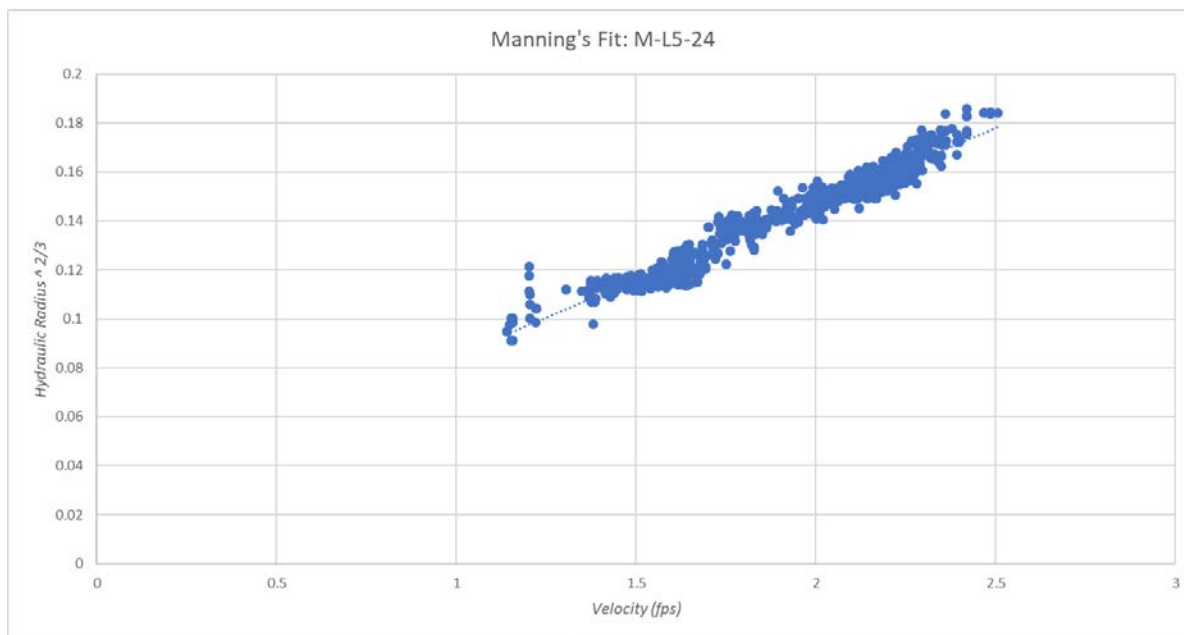


Signal to Noise Ratio

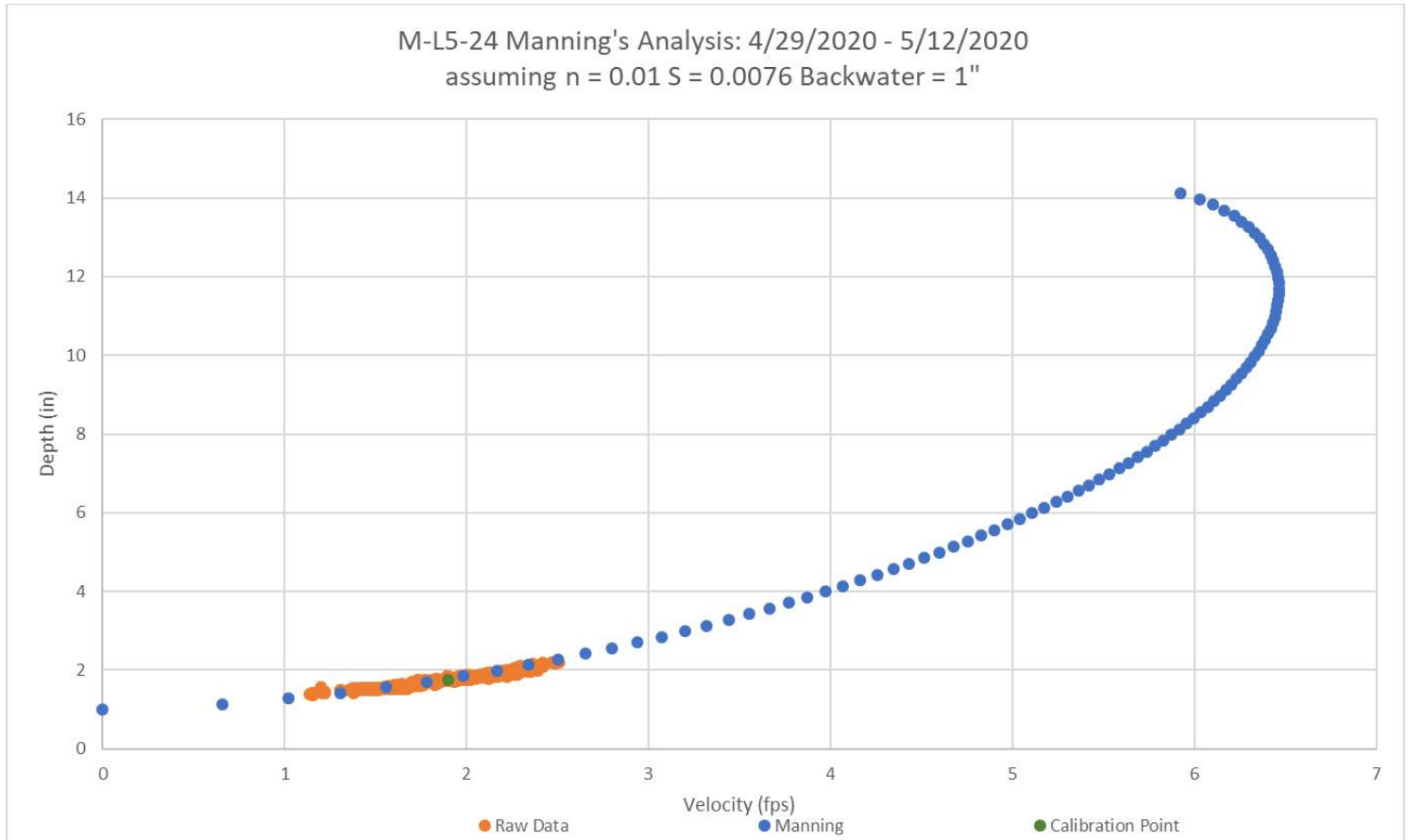


SNR = 164.6 Indicates excellent signal and minimal noise.

Manning's Analysis



NS = 0.76 and $R^2 = 0.96$. Excellent manning's fit.



Within minimum recommended slope range for pipe size. Indicates ~1" of backwater in the line.

Average Daily Flows

Date	Level, in	Velocity, fps	Flow, MGD
4/29/2020	1.76	1.94	0.10
4/30/2020	1.75	1.95	0.10
5/1/2020	1.75	1.91	0.09
5/2/2020	1.79	1.97	0.10
5/3/2020	1.82	2.00	0.11
5/4/2020	1.79	2.00	0.10
5/5/2020	1.78	1.98	0.10
5/6/2020	1.79	1.95	0.10
5/7/2020	1.79	1.96	0.10
5/8/2020	1.79	1.95	0.10
5/9/2020	1.82	1.98	0.10
5/10/2020	1.82	1.98	0.10
5/11/2020	1.81	2.01	0.10
5/12/2020	1.81	1.99	0.10

DDS Site - Data QAQC Report

Project Name: Wellington	Client: Town of Wellington	Date: 5/15/2020
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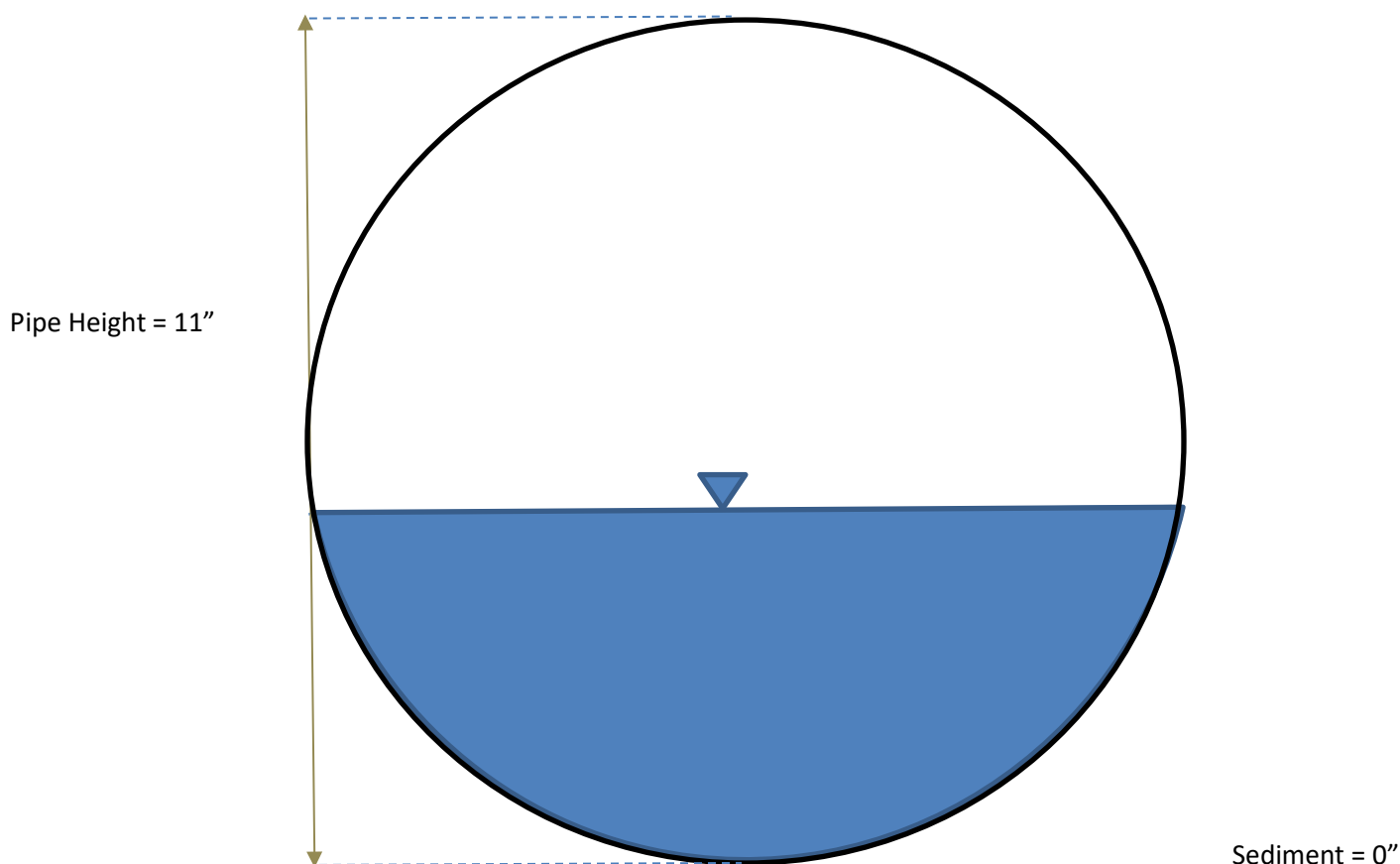
Maintenance Hole Name: M-L5-25		
Street Address/Location: 5 th Street & Mt. Owens		
Data Time Span: 4/29/2020 0:00 – 5/12/2020 23:45		
Pipe Shape: <i>Circular</i>	Pipe ID/Material: 11" <i>PVC</i>	Silt: 0"

Estimated Manning's n: 0.01	Estimated Slope: 0.004 ft/ft
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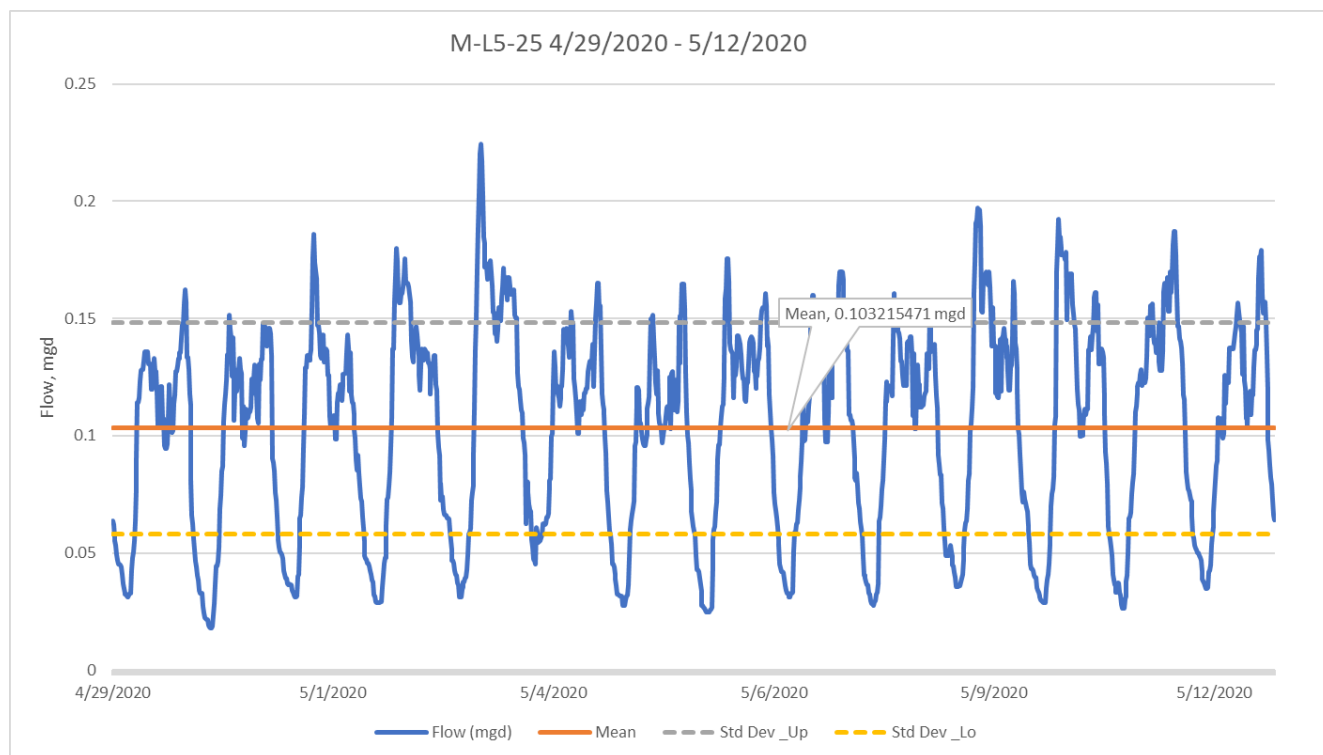
QAQC Analysis: Emily Steele, P.E.

Data shows high hydraulic correlation and minimal noise. Average Froude number of 0.92 indicating surface undulations and potential hydraulic jump conditions, however sensor is functioning well.

Pipe Schematic

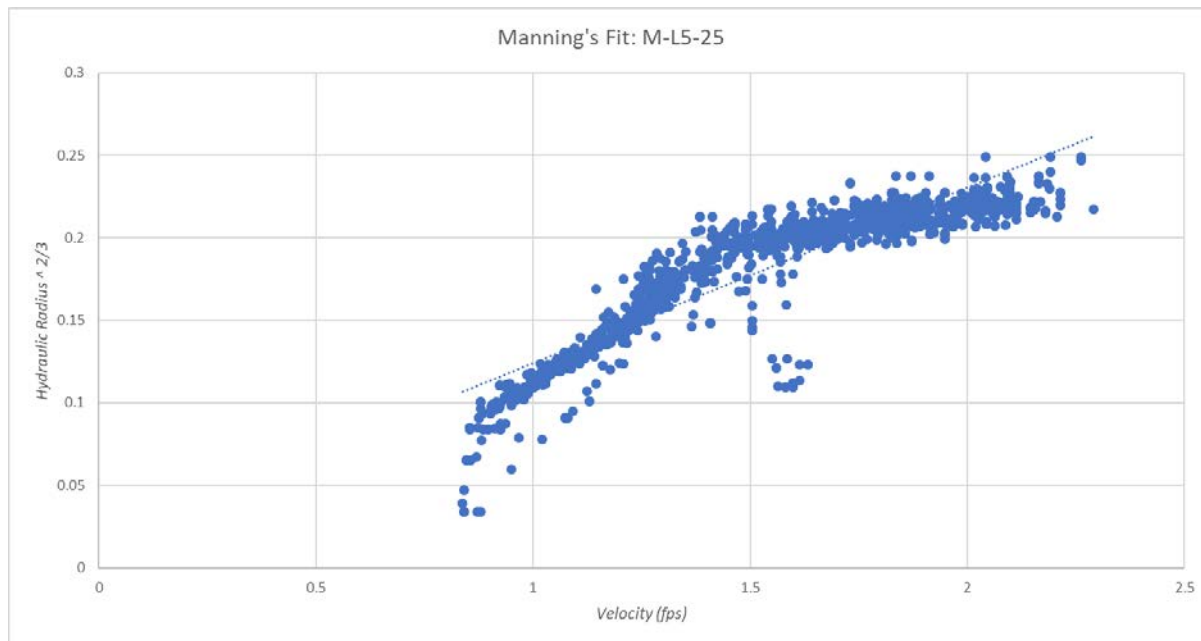


Signal to Noise Ratio

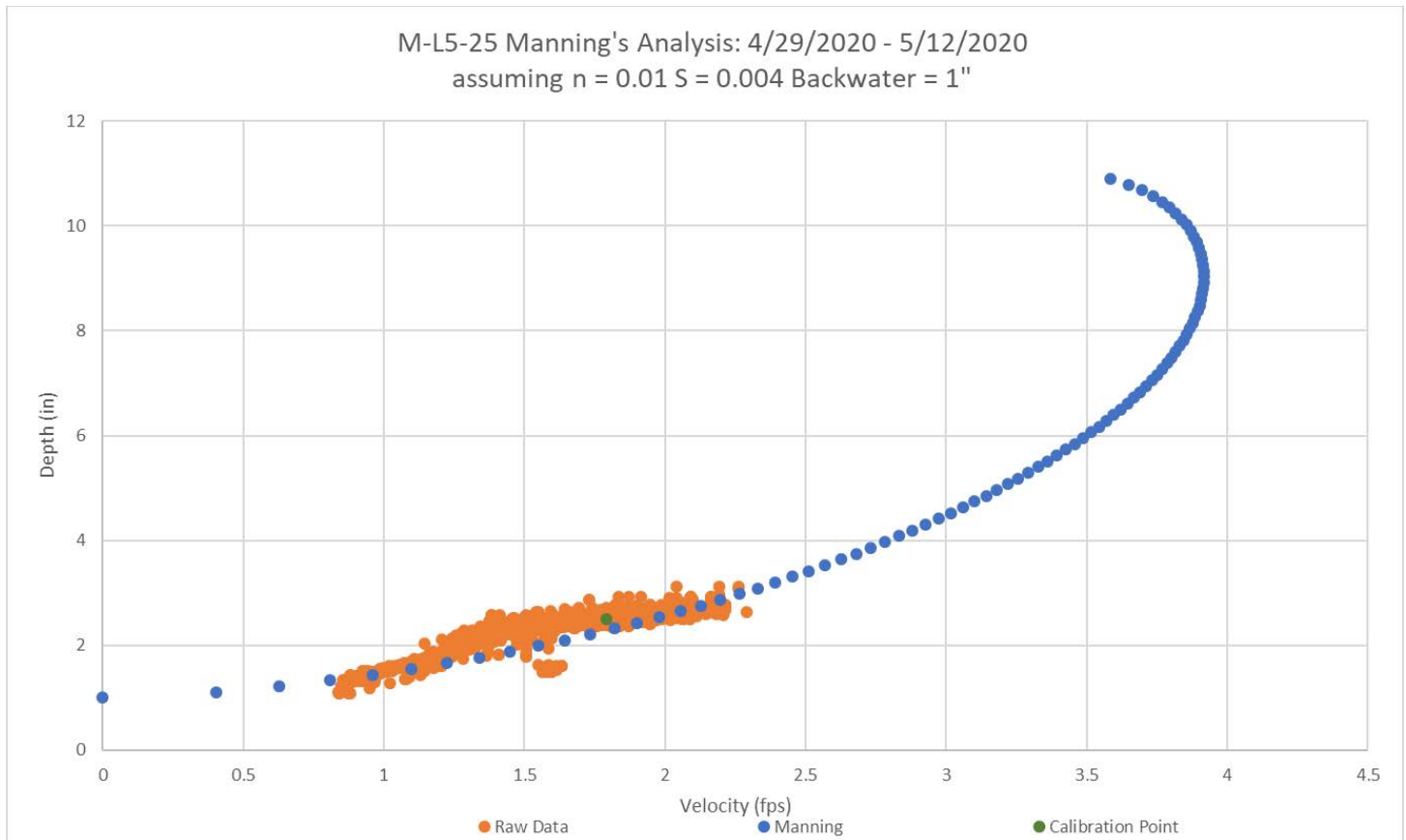


SNR = 76.2 Indicates excellent signal and minimal noise.

Manning's Analysis



NS = 0.65 and $R^2 = 0.82$. Excellent manning's fit.



Within minimum recommended slope range for pipe size. Indicates ~1" of backwater in the line.

Average Daily Flows

Date	Level, in	Velocity, fps	Flow, MGD
4/29/2020	2.18	1.52	0.09
4/30/2020	2.09	1.47	0.09
5/1/2020	2.23	1.51	0.10
5/2/2020	2.22	1.55	0.10
5/3/2020	2.31	1.65	0.11
5/4/2020	2.25	1.63	0.10
5/5/2020	2.19	1.46	0.09
5/6/2020	2.24	1.52	0.10
5/7/2020	2.27	1.52	0.10
5/8/2020	2.25	1.51	0.10
5/9/2020	2.31	1.57	0.10
5/10/2020	2.23	1.59	0.10
5/11/2020	2.24	1.59	0.10
5/12/2020	2.27	1.54	0.10

DDS Site - Data QAQC Report

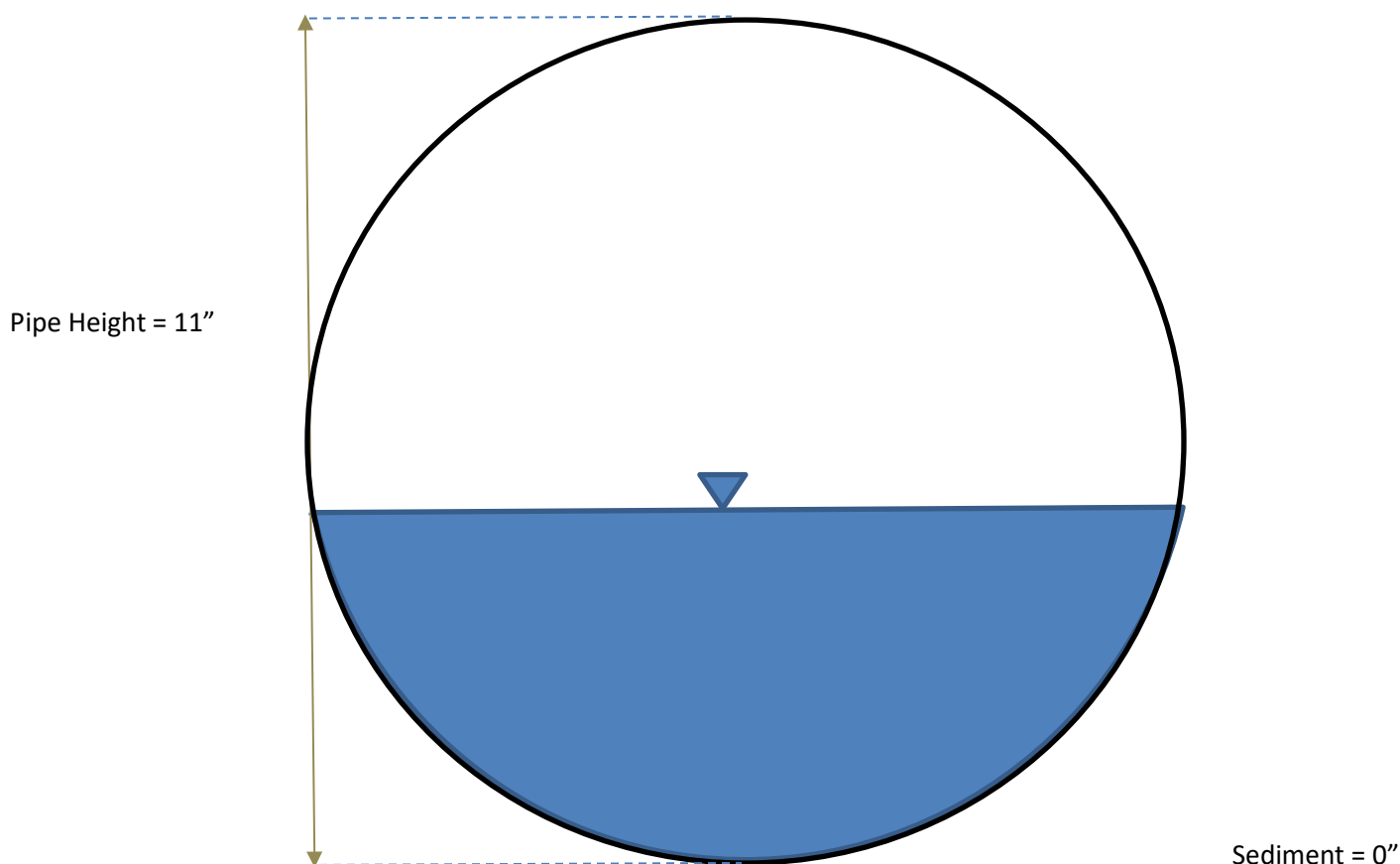
Project Name: Wellington	Client: Town of Wellington	Date: 5/15/2020
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Maintenance Hole Name: M-L6-13		
Street Address/Location: 4237 E CR 60		
Data Time Span: 4/20/2020 0:00 – 5/3/2020 13:45		
Pipe Shape: <i>Circular</i>	Pipe ID/Material: 11" <i>PVC</i>	Silt: 0"

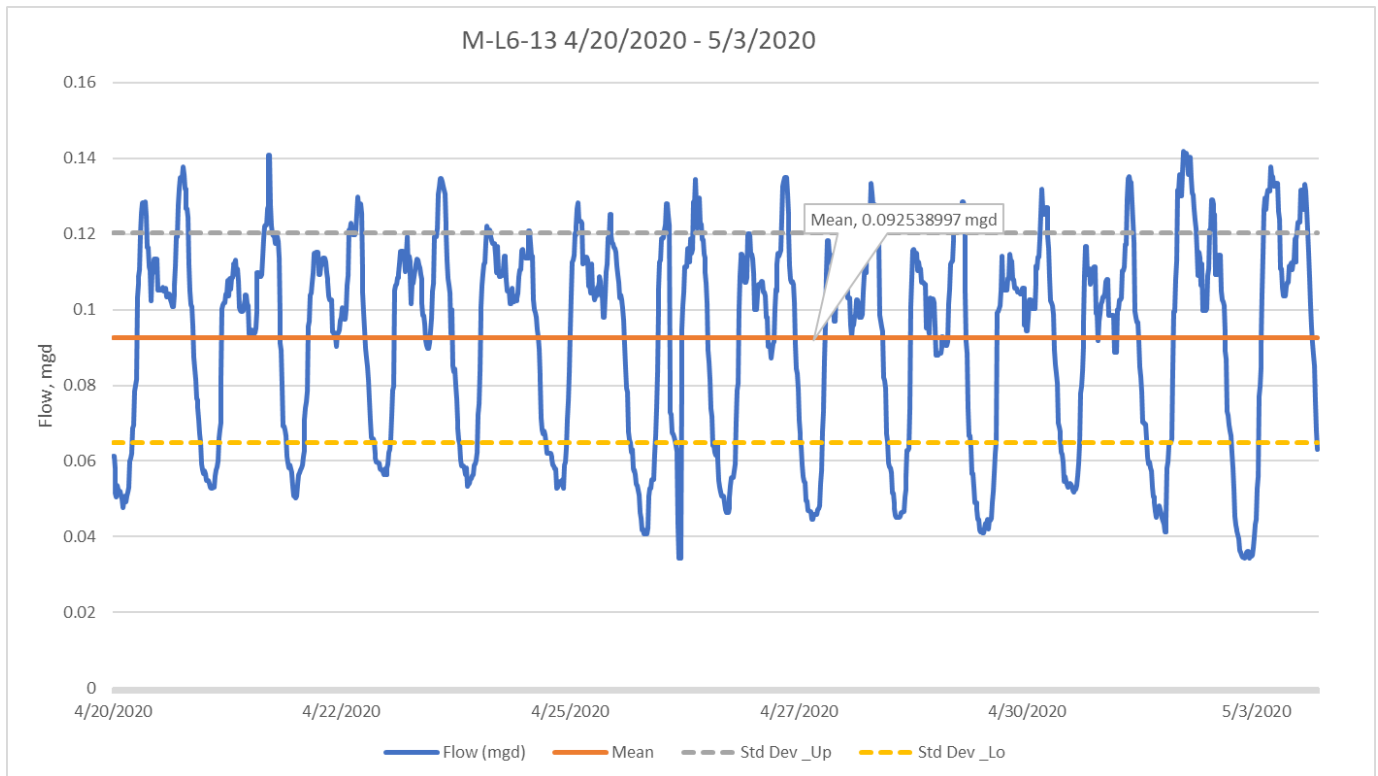
Estimated Manning's n: 0.01	Estimated Slope: 0.0027 ft/ft
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QAQC Analysis: Emily Steele, P.E.
Data shows high hydraulic correlation and minimal noise. Data shows a shift in the backwater effect indicating changes in downstream debris. Sensor is functioning well.

Pipe Schematic

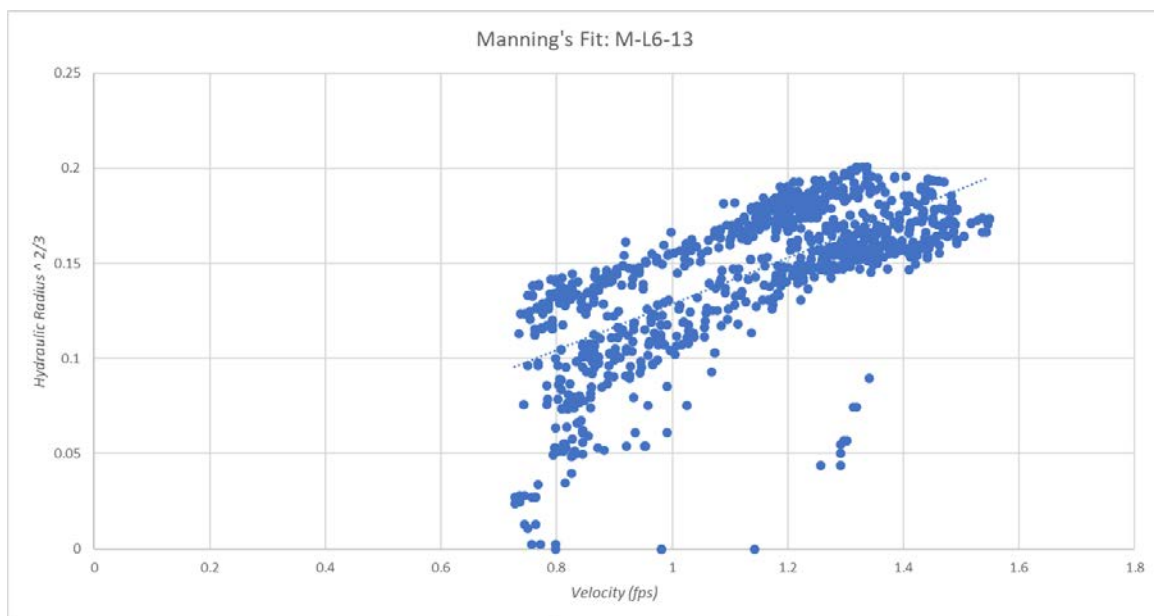


Signal to Noise Ratio

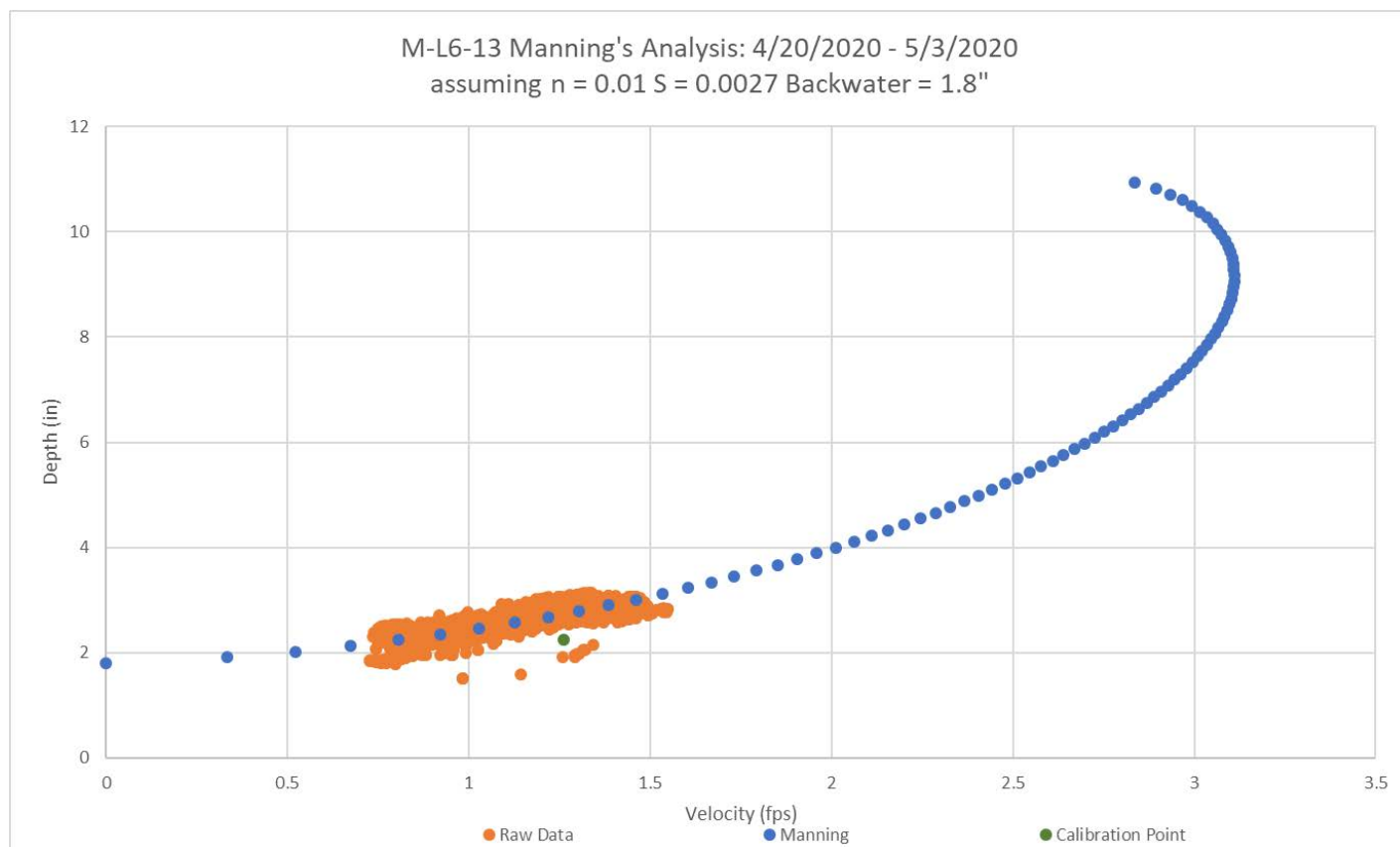


SNR = 124.6 Indicates excellent signal and minimal noise.

Manning's Analysis



NS = 0.51 and $R^2 = 0.520$ Good Manning's fit.



Within minimum recommended slope range for pipe size. Indicates ~1.8" of backwater in the line.

Average Daily Flows

Date	Level, in	Velocity, fps	Flow, MGD
4/20/2020	2.77	1.11	0.09
4/21/2020	2.74	1.11	0.09
4/22/2020	2.75	1.10	0.09
4/23/2020	2.81	1.10	0.09
4/24/2020	2.81	1.08	0.09
4/25/2020	2.62	1.18	0.09
4/26/2020	2.38	1.19	0.08
4/27/2020	2.53	1.22	0.09
4/28/2020	2.52	1.21	0.09
4/29/2020	2.50	1.18	0.08
4/30/2020	2.50	1.18	0.08
5/1/2020	2.62	1.15	0.09
5/2/2020	2.61	1.17	0.09
5/3/2020	2.49	1.18	0.09

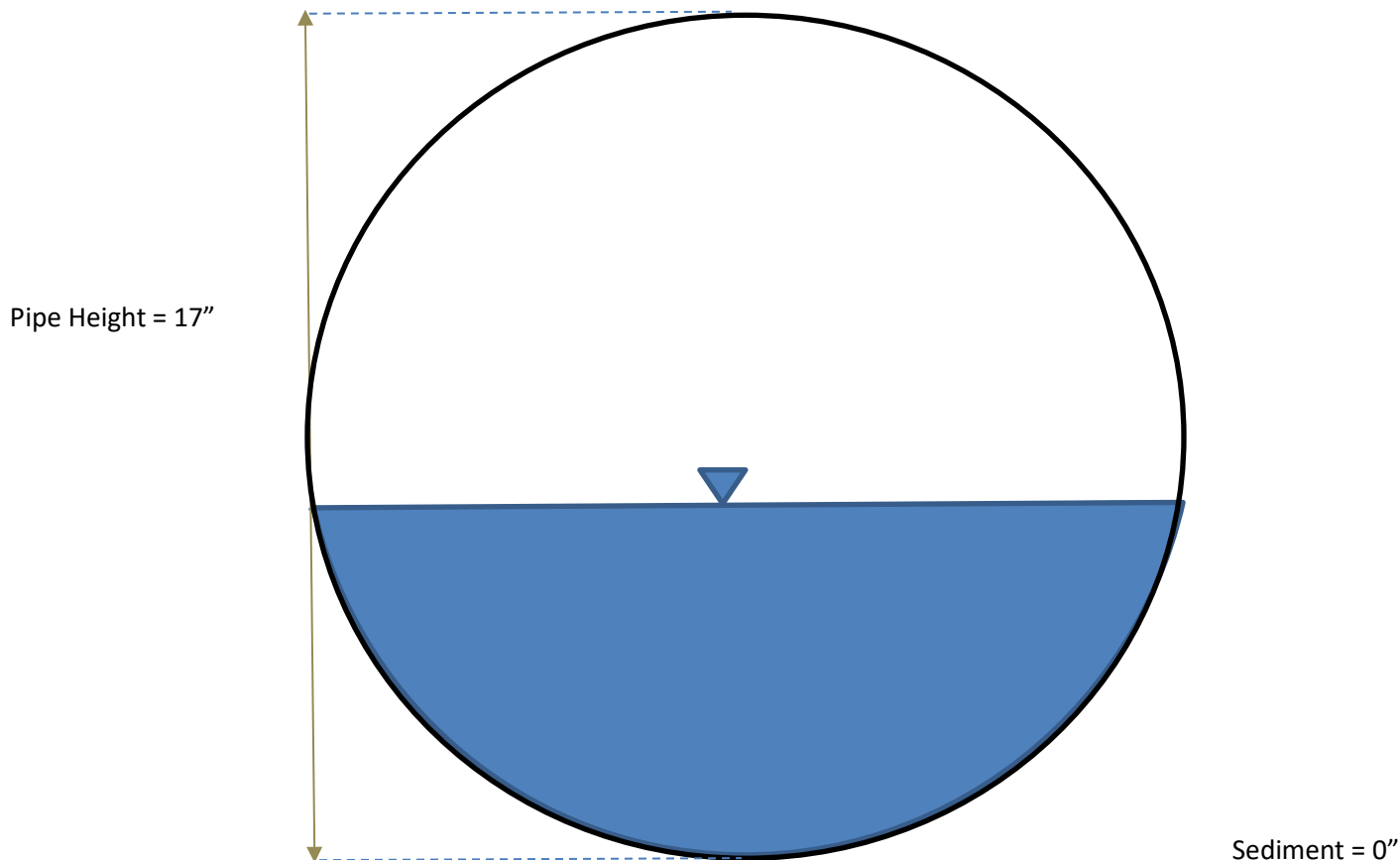
DDS Site - Data QAQC Report

Project Name: Wellington	Client: Town of Wellington	Date: 5/15/2020
--------------------------	----------------------------	-----------------

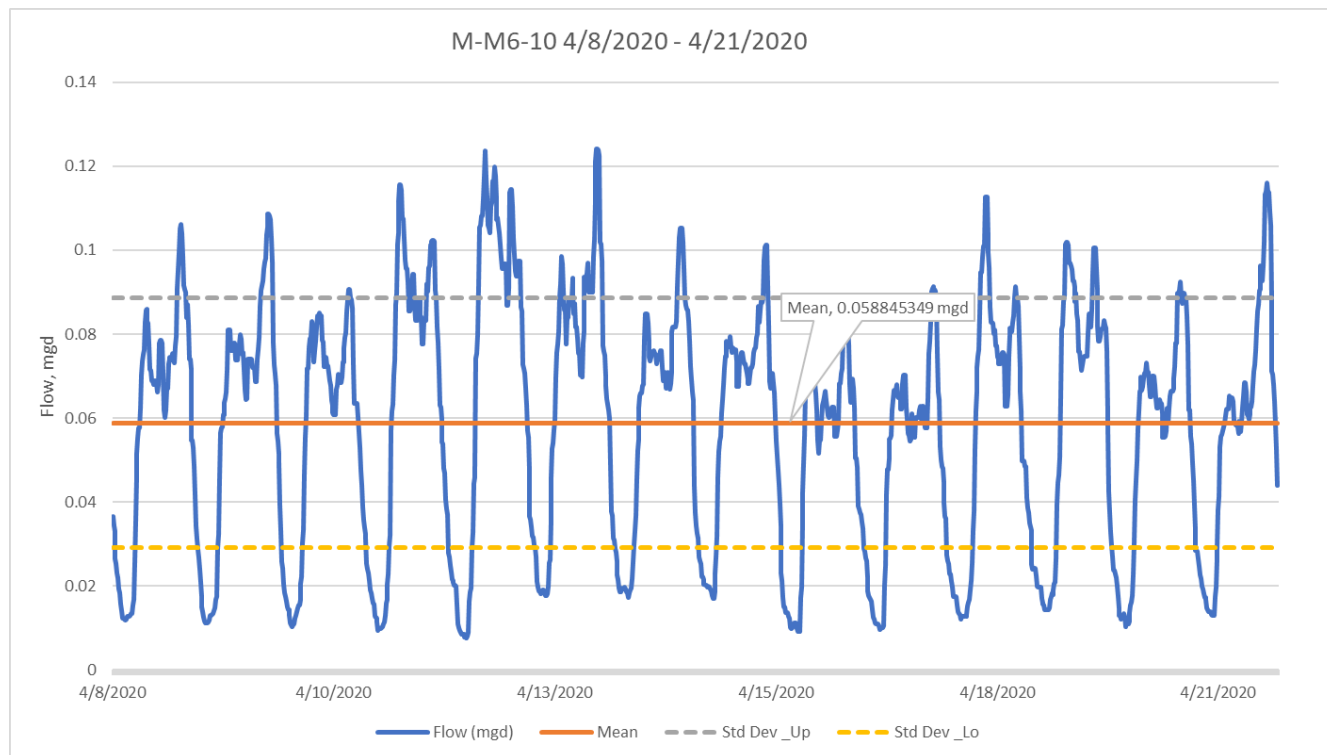
Maintenance Hole Name: M-M6-10		
Street Address/Location: Sewer Plant Road		
Data Time Span: 4/8/2020 0:00 – 4/21/2020 23:45		
Pipe Shape: <i>Circular</i>	Pipe ID/Material: 17" <i>RCP</i>	Silt: 0"

Estimated Manning's n: 0.013	Estimated Slope: 0.0018 ft/ft
QAQC Analysis: Emily Steele, P.E.	
Data shows high hydraulic correlation and minimal noise. Sensor is functioning well.	

Pipe Schematic

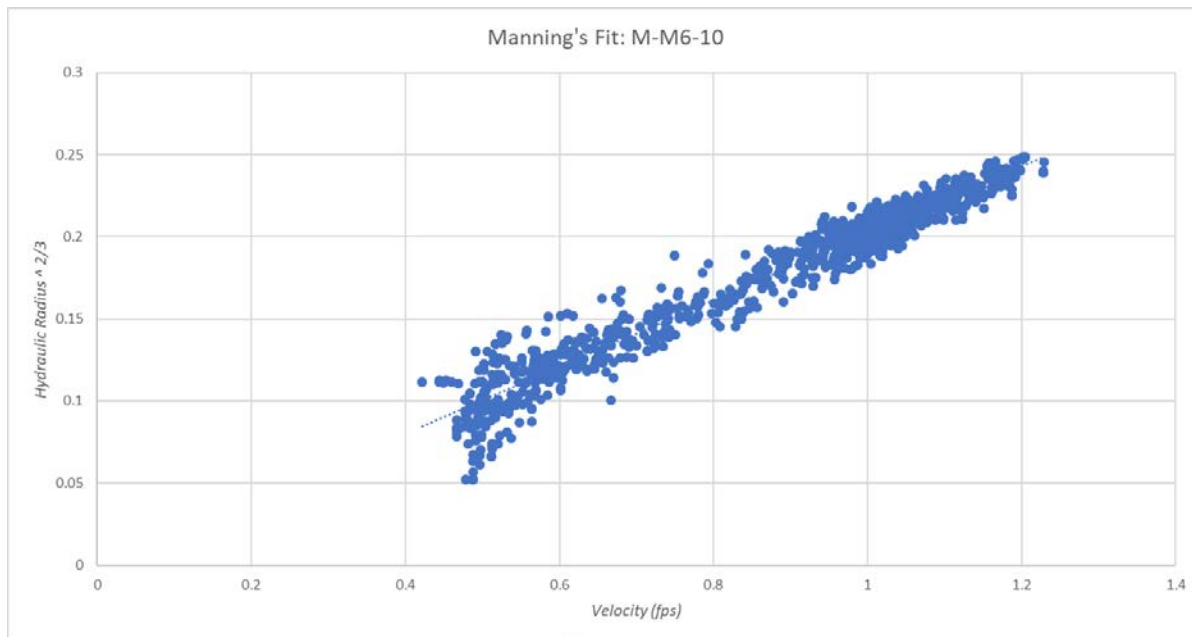


Signal to Noise Ratio

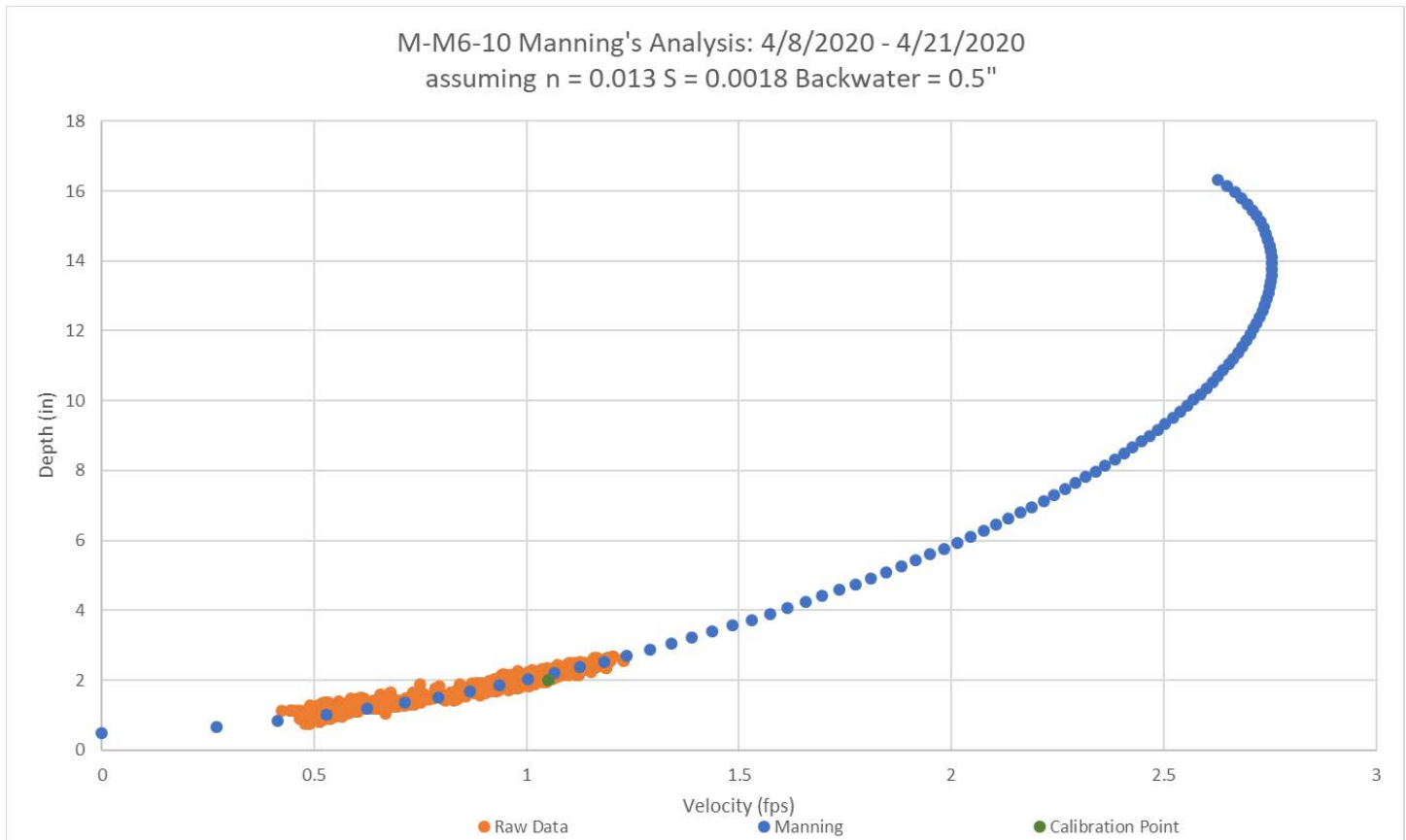


SNR = 176.4 Indicates excellent signal and minimal noise.

Manning's Analysis



NS = 0.94 and $R^2 = 0.95$. Excellent Manning's fit.



Within minimum recommended slope range for pipe size. Indicates ~0.5" of backwater in the line.

Average Daily Flows

Date	Level, in	Velocity, fps	Flow, MGD
4/8/2020	1.83	0.89	0.05
4/9/2020	1.85	0.90	0.06
4/10/2020	1.81	0.87	0.05
4/11/2020	1.86	0.91	0.06
4/12/2020	1.92	0.94	0.06
4/13/2020	1.99	0.94	0.06
4/14/2020	1.92	0.89	0.06
4/15/2020	1.92	0.88	0.06
4/16/2020	1.65	0.88	0.04
4/17/2020	1.68	0.87	0.04
4/18/2020	1.83	0.85	0.05
4/19/2020	1.85	0.88	0.05
4/20/2020	1.76	0.86	0.05
4/21/2020	1.81	0.89	0.05

Appendix B. Town of Wellington View Pointe Lift Station Capacity Analysis



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info@jvajva.com

November 11, 2019

Bob Gowing
Public Works Director
Town of Wellington
3735 Cleveland Ave
Wellington, CO 80549

www.jvajva.com

RE: Town of Wellington View Pointe Lift Station Capacity Analysis
JVA Job No. 2867c

Dear Bob:

JVA has been working with the Poudre School District (District) on the design of a new High School (School) located in Wellington, CO. The Schools sanitary service is proposed to be connected to the existing sanitary collection system owned and operated by the Town of Wellington (Town). The connection to this system is upstream of an existing lift station called the Viewpoint Lift Station (VPLS).

The purpose of this technical memorandum is to summarize the capacity of the VPLS regarding its ability to serve the new School for a maximum enrollment of 1500 students and 200 staff

EXISTING LIFT STATION ANALYSIS



Image 1: Control Building Looking North

On August 22, 2019 a site visit was conducted with JVA and Town Staff to the VPLS. The goal of the site visit was to observe the condition and rate the performance of the existing lift station. The lift station footprint is enclosed by a perimeter fence, which was in good condition. Within the perimeter fence the site includes an approach influent manhole to the lift station, duplex submersible lift station, lift station valve vault, small control building, a forcemain discharge manhole, effluent magnetic flowmeter and vault and backup-generator. The small control building houses the electrical gear and control / instrument panels for the lift station and back-up propane fired generator.

The precast concrete wetwell is eight feet in diameter and approximately 30 feet deep, exact depth could not be verified with the system in operation. An engineering report prepared by Sear Brown in May 2001 provides design information for the VPLS which is included with this memo as Attachment 1. The design information shows that the wet well is 8-feet in diameter with an 18-inch influent sewer invert of 50.13 feet and wetwell basin floor of 41.38 feet. GIS data provided by the Town does not show the top of wet well and invert elevation for the VPLS. There is GIS data for the upstream and downstream manholes from the VPLS which is provided in Attachment 2 of this memo. It is likely the survey datums used during the 2001 study and the Town GIS are different but the relative elevation differences should be the same. Using this

rationale, it is estimated that the 18-inch sewer invert and wetwell floor elevation is 20 feet and 30 feet below the top of VPLS wetwell.



Image 2: Lift Station Valve Vault

The joints between the different wetwell risers sections appeared to be in good condition, and no infiltration and inflow (I&I) was observed. Two Bilco-style stainless steel hatches are used to access the interior of the wetwell. Within the wetwell, a duplex pumping system lifts raw sewage to the manhole located by the entrance gate where it breaks head and is conveyed via gravity to the Town's WWTF. The duplex submersible pumps can be removed using Davit crane and hoist system from the back of a service truck. The VPLS is equipped with a submersible pressure transducer with back-up low cut off float switch, Pump ON float switch and high-water alarm float switch in case the primary level control system (transducer) fails. In automatic mode the operators can set the ON and OFF levels via transducer interface for both the lead and lag pumps and can alternate lead pump on a periodic basis. The platform grating and support brackets inside the wet well show signs of corrosion. It is noted that the electrical junction box that contains the connection between the pump power chords and secondary electric feed is in very poor condition and represents a dangerous or outage situation should the junction box completely corrode, and wiring become exposed and short out. This junction box should immediately be replaced and located outside the hazardous air space of the wet well. It was also observed that the top rail support bracket for each pump was mounted backwards (see image below) making it very difficult for pump removal and replacement. In addition, the material used for the top rail support bracket is carbon steel and fully corroded. The top support bracket for the rail system will have to be replaced with 316 stainless steel and mounted correctly for proper pump removal.

The heart of the VPLS are the two ABS Model AFP 1046 submersible pumps. Based on the pump curve and existing documentation, each ABS pump is rated for 620 gallons per minute (gpm) at 33.5 feet of total dynamic head (TDH). Attachment 3 to this memo is the pump performance curve



Image 3: Wetwell Exterior & Fuel Tank

for the submersible pump. To meet CDPHE requirements for duplex pumps, each pump must be rated for peak hourly flow conditions with one stand-by. To confirm flowrates, a real time flow was calculated by two different methods: 1) recording the quantity of gallons pumped through the magnetic flowmeter over a pump cycle time; and 2) the level drop over a period of time. Method 1 yielded a 770 gpm flowrate, and Method 2) yielded a 729 gpm flowrate, which resulted in an average flowrate of 750 gpm. For Method 1, the flow meter totalized flow was recorded prior to the pump coming on at operator set PUMP ON wet well level of 7.5 feet. The pump turned on at 7.0 feet and continued until the pump turned off at 4.0 feet at which time the pump flow meter totalizer was recorded. For Method 2, the pump cycle wet well levels of 7.5 feet and 4.0 feet were used to calculate the draw down volume pumped over the time period the pump ran when activated at 7.5 feet and turned off at 4.0 feet. Using the



Image 4: Wetwell Interior



wet well diameter of 8.0 feet the total volume was calculated and divided by the time it took for the pump to complete a full cycle. This method was more conservative because it did not account for the incoming flow during the draw down test, thus the reason for the two different values between Method 1 and Method 2. Based on field measurements, the actual operating capacity of the lift station is approximately 130 gpm higher than the design point. Using the pump performance curve, the verified pump capacity of 750 gpm was plotted on the curve and lined up with the rated TDH of 39 feet for the pump. It is theorized that the reason for the higher pump capacity is that the original lift station forcemain was upsized from a 4-inch to an 8-inch diameter resulting in less friction head and higher pump capacity. The rated pump station capacity of 750 gpm will be used for assessing additional capacity for the VPLS.

Town staff noted that the pumps short cycle, meaning that the number of pump starts and stops across the day is excessive. The Town has adjusted the ON-OFF levels a greater distance apart which has prolonged the ON time and has reduced number of pump starts and stops but staff is still in the opinion that this is too excessive. It is proposed that each motor be integrated with a new variable frequency drive (VFD) which will allow staff to run the pumps for longer periods of time while maintaining an operator set level in the wet well with the pump increasing and decreasing in speed to maintain that set wet well level. Another advantage of the VFD is that there is a built-in soft start and soft stop for the pump motor prolonging the lifetime.

FLOW ANALYSIS

To determine whether the existing VPLS can accept increased flows from the School, anticipated per capita flows from the School were calculated and added to historic flow measurements from the VPLS. Anticipated per capita School flows were derived from a comparison between historic per capita flowrates from similar schools. Referenced attached electronic Excel spreadsheet that summarizes the VPLS flow log reports.

The District provided comparable monthly water meter usage for existing high schools and middle schools in Fort Collins along with school enrollment. The water meter usage data and enrollment from Wellington Middle School, Preston Middle School, and Fossil Ridge High School was recorded for calendar years 2016 through 2018 and evaluated for comparison with the new School. It should be noted that its industry standard to deduct ten percent due to consumption when using water meter readings for wastewater flow rates, but is not included in this analysis to be conservative. As seen below in Table 1, the highest average per capita flowrate is 6.6 gpd/student. This per capita flowrate is in agreeance with other high school and middle school systems evaluated by JVA throughout Colorado. It is noted that the most applicable data for the School would be the Fossil Ridge High School based on student and staff water usage and activities (eating cafeteria vs eating out, showering, etc.); however, for the purpose of being more conservative the highest recorded student usage of 6.6 gpd was used.

Table 1 – PSD Schools Per Capita Flowrates

Year	School	Enrollment	Water Use over the School Year (185 Days) (average daily flow usage in gallons)	Per Capita Flow (gpd/student)
	Wellington Middle School	566	3,124	5.5

2018 – 2019	Preston Middle School	1,147	4,691	4.1
	Fossil Ridge High School	2,041	9,786	4.8
2017 – 2018	Wellington Middle School	550	3,049	5.5
	Preston Middle School	1,146	7,608	6.6
	Fossil Ridge High School	1,992	10,116	5.1
2016 – 2017	Wellington Middle School	504	2,600	5.1
	Preston Middle School	1,154	6,628	5.7
	Fossil Ridge High School	1,950	9,811	5.0

The calculated increased daily flows to the VPLS from the new School are shown in Table 2. The wastewater flows generated by the new School were estimated using two approaches; one for generated flow across a 24-hour period; and second for generated flow across a school day period of 8 hours. The 8-hour school day approach captures the peak flows to the VPLS during the time the students are in school and represents the most conservative approach. As seen below in Table 2, anticipated maximum daily flowrate to the VPLS range will be 9,900 gpd. It is pointed out that the selected per capita (student) daily flow rate is conservative considering it is taking into account water usage during times the students are not at school.

Table 2 – Increased Daily Flow to the View Pointe Lift Station

Flow Criteria	Per Capita Flow	Number of New Students	Increased Flow to View Pointe Lift Station (gpd)
Poudre School Recorded Maximum Daily Flowrate	6.6	1500	9,900

The Town provided VPLS flow metering records from May 2017 through July 2019 which were metered and recorded by the existing magnetic flow meter and transmitter. Throughout this time period, the highest peak day and average daily flows were 376,708 and 187,573 gpd, respectively, as shown in Table 3. Documentation of the flow data shown in Table 3 is provided in the attached electronic Excel spreadsheet (Attachment 5). The highest monthly average flow and corresponding peak daily flow recorded at the View Pointe Lift Station between May 2017 to July 2019 were used to be most conservative in evaluating the impact of the new School.

Table 3 – Monthly and Peak Daily Flows to the View Pointe Lift Station

Date	Historic Average Flow (GPD)	Peak Flow Rate (GPD)
May-17	157,644	309,567
Jun-17	155,434	530,532 (see note 1)
Jul-17	157,252	294,248
Aug-17	156,370	224,049



Sep-17	154,935	189,668
Oct-17	164,264	230,689
Nov-17	161,483	227,867
Dec-17	187,573	376,708
Jan-18	160,617	224,314
Feb-18	165,327	269,025
Mar-18	162,453	303,439
Apr-18	162,170	201,676
May-18	157,188	191,988
Jun-18	159,263	232,321
Jul-18	148,417	239,131
Aug-18	155,637	186,417
Sep-18	146,566	188,886
Oct-18	159,098	184,793
Nov-18	163,559	201,149
Dec-18	170,175	210,271
Jan-19	169,094	192,419
Feb-19	182,620	222,979
Mar-19	181,506	314,784
Apr-19	155,986	320,094
May-19	181,142	299,966
Jun-19	174,862	297,629
Jul-19	149,775	180,662

Note (1): This flow rate is considered an outlier based on the statistical comparison and not recommended to use for existing flow

VIEW POINTE LIFT STATION IMPACT ANALYSIS

The overall wastewater flow impact to the VPLS from the new School was quantified by using the per capita flow criteria established with both the percent increase to peak historic flow, and the increased daily pump run times. As seen in Table 4 below, the percent increase in flow to the peak historic daily flow is very small equating to a daily average increase of 2.6 percent. Similarly, the increased daily pump run times is small equating to a daily average increase in run time of 13.2 minutes.



Table 4 – View Pointe Lift Station Impact from New High School

Flow Criteria	Percent Increase to Maximum Historic Flow Recorded in Dec 2017	Increased Daily Run Time per Day to Lift Station at 750 gpm Pump Capacity (min)
New High School Estimated Average Flow Impact (+9,900 gpd)	2.6%	13.2

Shown in Table 5 is an estimate for total flow projection to the VPLS based on several different flow conditions. These conditions include average daily flows and peak hourly flows. In accordance with CDPHE regulations, lift stations must be capable of handling peak hourly flows with the largest pump out of service. As described earlier, the VPLS is a duplex station meaning that each pump must be sized for the design peak hourly flows. Peak hourly flows are typically calculated using the daily average flows multiplied by a peak hourly flow factor. The industry standard that is widely accepted by review agencies for peaking factors is from the document called “Recommended Standards for Wastewater Facilities – 2014 Edition”. The peaking factor is derived from the population base of the area served. For the VPLS, it is estimated that approximately 2000 capita are served based on the estimated average daily flow in Table 5 of 197,500 gpd using 100 gallons per capita per day. Attachment 4 to this memo is the peaking factor curve based on 2000 capita and resulting peaking factor of 3.7. To validate the wastewater peaking factor for the New School, Fossil Ridge High School (FRHS) hourly water flow meter records were provided by the District. The 24-hour flow meter records for FRHS is attached to this memo (Attachment 5). Using maximum day flow consumption for the month (March 12, 2019) of 9520 gallons divided by 24 hours equates to an average hourly flow of 398 gallons. The average hourly flow of 398 gallons was divided by the recorded peak hour flow of 1520 gallons equating to a peaking factor of 3.8. Multiplying the peak factor of 3.8 to the New School flow of 9,900 GPD yields a peak hourly flow of 37,620 GPD. The increase in VPLS rated capacity the peak hourly flow from the New School will be 4 percent. Keep in mind that the flows generated from the new School represent the most conservative based on the flow records used.

Table 5 – Total Flow to View Pointe Lift Station – Historic and New School

Flow Criteria	Average Daily Flows (GPD)	Peak Hourly Flows (GPD)	Peak Hourly Flows (GPM)
Historic Flow	187,600	694,120	482
New School Flow	9,900	37,620	26
Total Flow	197,500	731,740	508
Rated Capacity of VPLS	-	1,080,000	750
Increase Percentage of VPLS Capacity by the New School	-	4%	4%

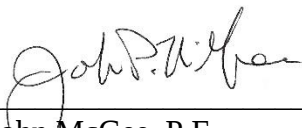
The Town requested estimated organic loading impact from the New School. CDPHE Regulation 43 provides institutional (schools) organic loadings for on-site wastewater treatment systems. The most representative organic loading per student for the New School is 0.08 lbs BOD5 per day. For 1500 students and 200 staff members, the total organic loading impact from the new school is 136 lbs BOD5 per day.



CONCLUSIONS

The added flow from the new School will increase the rated capacity of the VPLS from 64 to 68 percent or a 4 percent increase. The percent capacity was calculated using the historic and School peak hour flows and rated capacity of VPLS shown in Table 5. The VPLS has adequate capacity to serve the ultimate wastewater flows from the new School.

Sincerely,
JVA, INCORPORATED

By: 

John McGee, P.E.
Principal

ATTACHMENT 1

**ENGINEERING REPORT
FOR
WELLINGTON VIEW POINTE LIFT STATION
EXPANSION**

**Prepared For:
Town of Wellington, Colorado**

**Prepared By:
The Sear Brown Group
209 S. Meldrum
Fort Collins, CO 80521
970-482-5922**

Job No. 212-023A

May, 2001

TABLE OF CONTENTS

A. Existing Lift Station Design Information.....	1
B. Proposed Expansion	1
C. Hydraulic Analysis	1
D. Cost Estimate for the proposed Expansion	2

Attachments:

1. Hydraulic Calculations and Pump Curves.
2. Cost Estimate.

ENGINEERING REPORT FOR WELLINGTON VIEWPOINTE LIFT STATION

A. EXISTING LIFT STATION DESIGN INFORMATION

1. Pumps: Duplex submersible pumps
ABS Model AFP 1040 4P.
140 gpm @ 26 feet TDH.
2. Static Head: 25.19 feet.
3. Wetwell: 8 feet diameter precast concrete basin.
18" Influent pipe invert = 50.13 feet.
Wetwell basin floor elevation = 41.38 feet.
4. Float switches: Low water float: 42.13
High water float: 46.50
5. Forcemain: 4" DIP Class 50.
6. Link-seal: 12" opening with two link-seal units for wall penetrations.

B. PROPOSED EXPANSION

1. Pumps: Replace the existing pumps with ABS Model AFP 1046 duplex pumps, with #2 impeller.
2. Forcemain: Add a 8" DIP Class 50 forcemain paralleling to the existing 4" forcemain. The existing 4" forcemain will be abandoned in place.
3. Discharge Piping: Replace the existing the 4" pump discharge piping with 8" piping. Use a 4"x8" reducer at the pump outlets for 8" pipes connections.
4. Valves and fittings: Replace all existing valves and fittings with 8" new valves and fittings except the 4" plug valve on the existing 4" forcemain in the valve vault. Use a 4"x8" reducer to connect the 8" new pipe to the existing 4" forcemain.
5. Float switches: Adjust floats switch elevation settings for the new pumps.
6. Pipe Supports: Modify existing pipe supports for the 8" new piping and fittings.

C. HYDRAULIC ANALYSIS

1. 4" Forcemain: Add a new 4" forcemain paralleling to the existing 4" forcemain. This option can deliver 540 gpm flow at 38.5 TDH for the selected AFP 1046 pump with #2 impeller. However, the velocity in the 4"

- discharge pipe will be as prohibitively high as 12 ft/s.
2. 6" Forcemain: Add a new 6" forcemain paralleling to the existing 4" forcemain. The 4" forcemain will be abandoned in place. Discharge piping, valves and fittings will be replaced with 6" new piping, valves and fittings. This option will delivered 590 gpm at 35 feet TDH, 70% pump efficiency and a moderately high velocity 7 feet/s.
 3. 8" forcemain: Add a new 8" forcemain paralleling to the existing 4" forcemain. The 4" forcemain will be abandoned in place. Discharge piping, valves and fittings will be replaced with 8" new piping, valves and fittings. This option will delivered 620 gpm at 33.5 feet TDH, 70% pump efficiency and a low velocity 3.5 feet/s.

D. COST ESTIMATE FOR THE PROPOSED EXPANSION

Total expansion cost was estimated at approximately **\$116,600**, which includes 15% engineering fee and 10% for contingency. Cost with an optional variable frequency drive (VFD) was estimated at approximately **\$126,600**. Sear-Brown does not have control for cost of labor, materials, equipment or services provided by others, or over the contractor's methods of determining price, or over competitive bidding or market conditions. It is understood that the above opinion of probable construction cost estimates are provided for the project based on Sear-Brown's best professional judgement, experiences and information available to us at present. The opinion of probable cost is an order-of-magnitude estimate. The expected accuracy ranges from plus 15% to minus 10% of the actual project cost. The opinion of probable cost should be used budgeting purposes.

Attachment #1:

Hydraulic Calculations and Pump Curves

Attachment #2:

Cost Estimate.

ATTACHMENT 2



FIGURE # - TITLE
JOB NAME
DATE



JVA, Inc.
1319 Spruce Street
Boulder, CO 80302
303.444.1951
www.jvajva.com
Boulder • Fort Collins • Winter Park
Glenwood Springs • Denver

ATTACHMENT 3



Pump performance curves AFP 1546 60 HZ

Curve number

Reference curve
AFP1546

Frequency
60 Hz

Date
2007-01-22

NPSH

Density
62.428 lb/ft³

Viscosity
0.000016813 ft²/s

Testroom
Hydraulic Institute

Discharge
6"

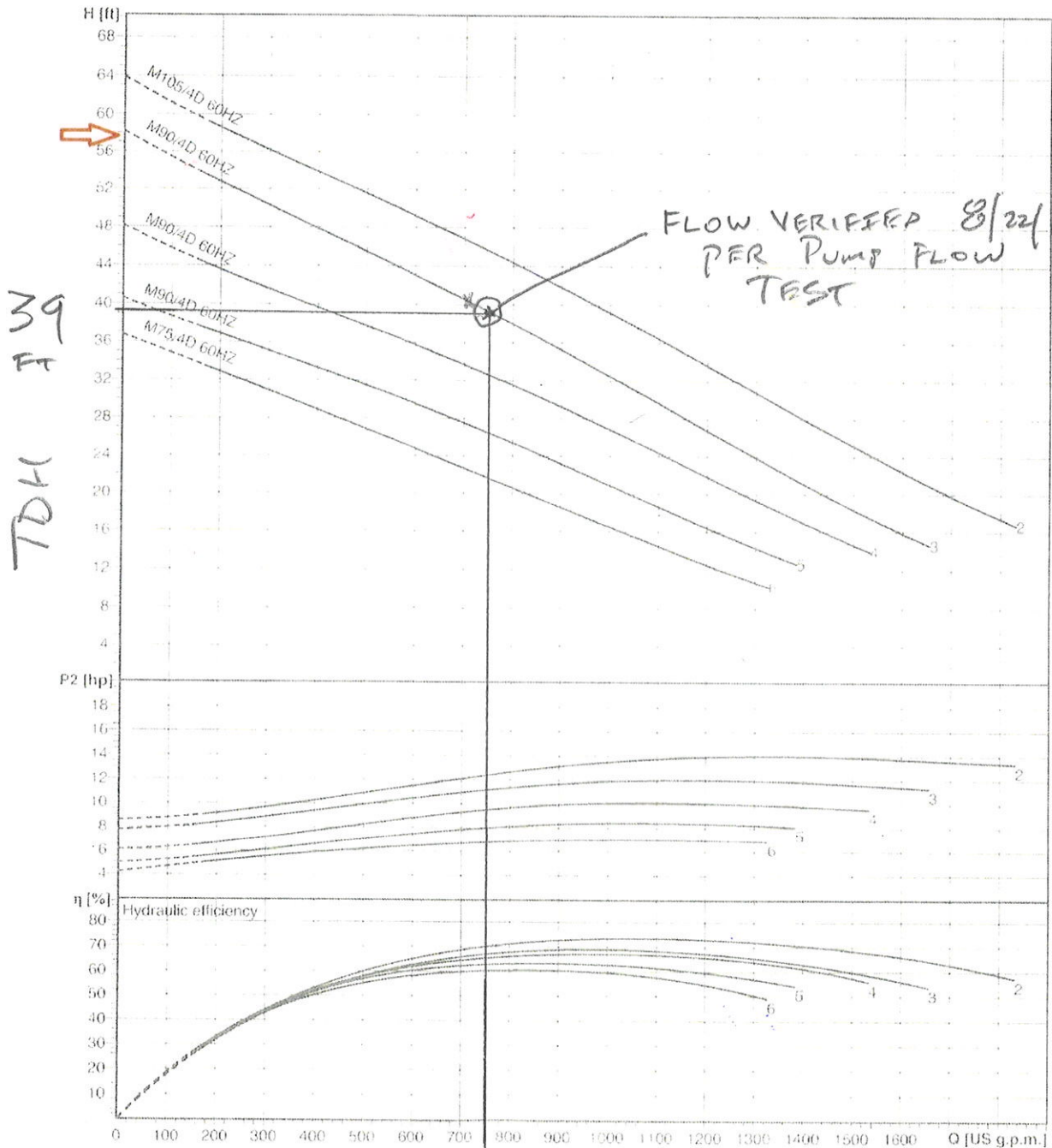
Rated speed
1735-1700 rpm

Hydraulic efficiency

Flow

Head

Rated power



Impeller size

N° of vanes
2

Impeller
2-Channel ContraBlock impeller

Seal size
3"

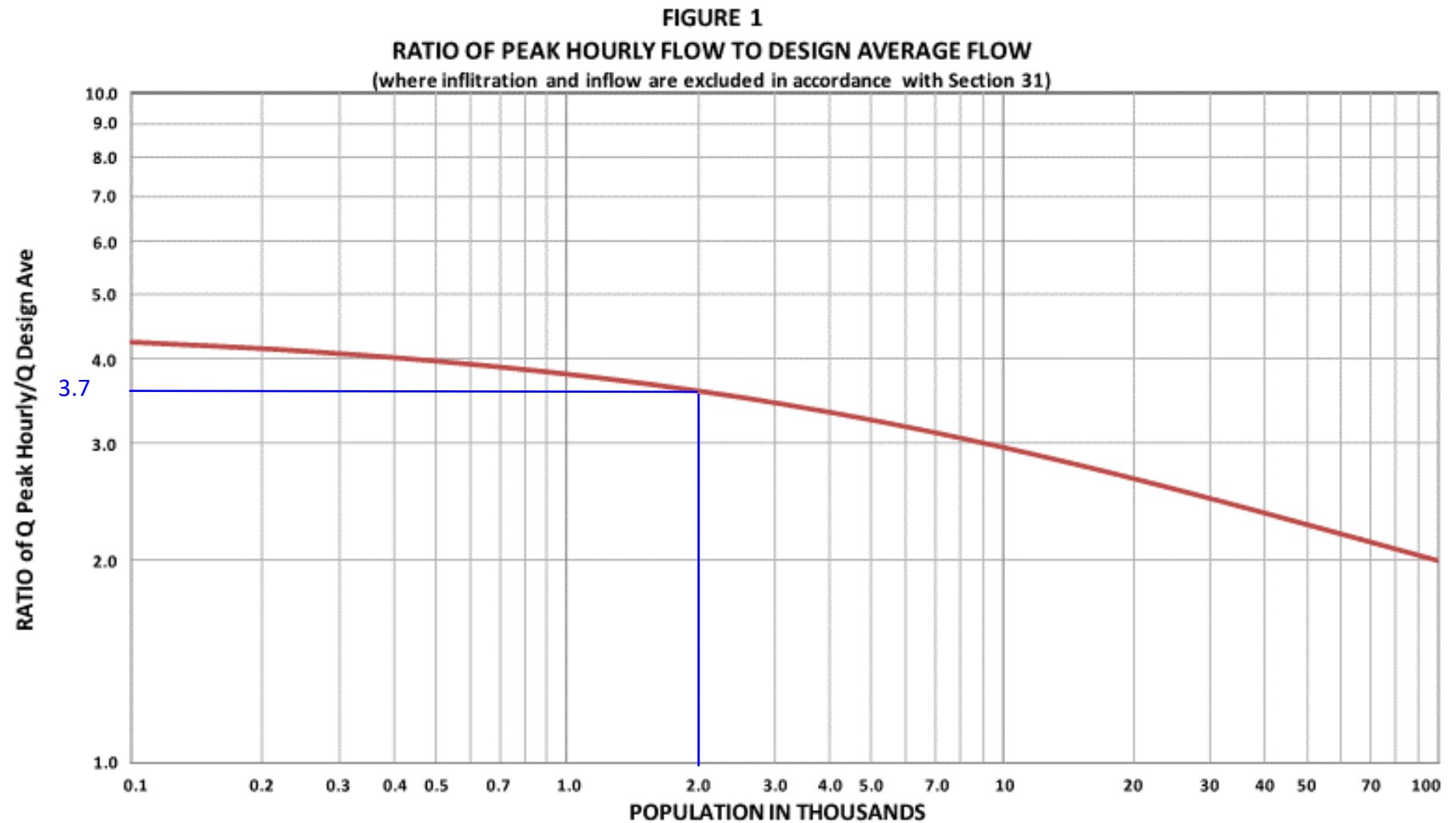
Revision
2005-06-14

ABS reserves the right to change any data and dimensions without prior notice and can not be held responsible for the use of information contained in this software

ABSL PRO 1.7.1 2005-03-17

ATTACHMENT 4

ATTACHMENT 4: PEAKING FACTOR CURVE



Q peak hourly: Maximum Rate of Wastewater Flow (Peak Hourly Flow)

Q design ave: Design Average Daily Wastewater Flow

Source:
$$\text{Q Peak Hourly/Q Design Ave} = \frac{18 + \sqrt{P}}{4 + \sqrt{P}} \quad (P = \text{population in thousands})$$

Fair, G. M. and Geyer, J. C., "Water Supply and Waste-water Disposal"
 1 st Ed., John Wiley & Sons, Inc., New York (1954), p. 136

ATTACHMENT 5

ATTACHMENT 5: FRHS 24-HOUR FLOW METER RECORDS

Hourly Water Usage				
Time	3/4/2019	3/5/2019	3/6/2019	3/12/2019
0:00	0	10	30	10
1:00	0	0	10	0
2:00	0	10	10	0
3:00	0	0	0	0
4:00	0	0	10	0
5:00	0	10	10	10
6:00	60	60	60	10
7:00	20	110	60	110
8:00	530	520	350	610
9:00	590	580	610	670
10:00	910	930	840	1040
11:00	950	1280	1270	1520
12:00	1000	860	940	740
13:00	1070	1100	1130	1180
14:00	1020	950	1010	950
15:00	910	960	1070	1010
16:00	470	400	470	460
17:00	200	170	380	130
18:00	260	160	180	230
19:00	230	150	110	310
20:00	130	110	50	70
21:00	130	120	220	250
22:00	120	90	130	90
23:00	100	80	70	120
Daily Total:	8700	8660	9020	9520

Appendix C. Dwelling Unit Grid for Existing Conditions Model

RES. DENSITY DIST.

1-4-21

[*] = 160 AC
= DU's

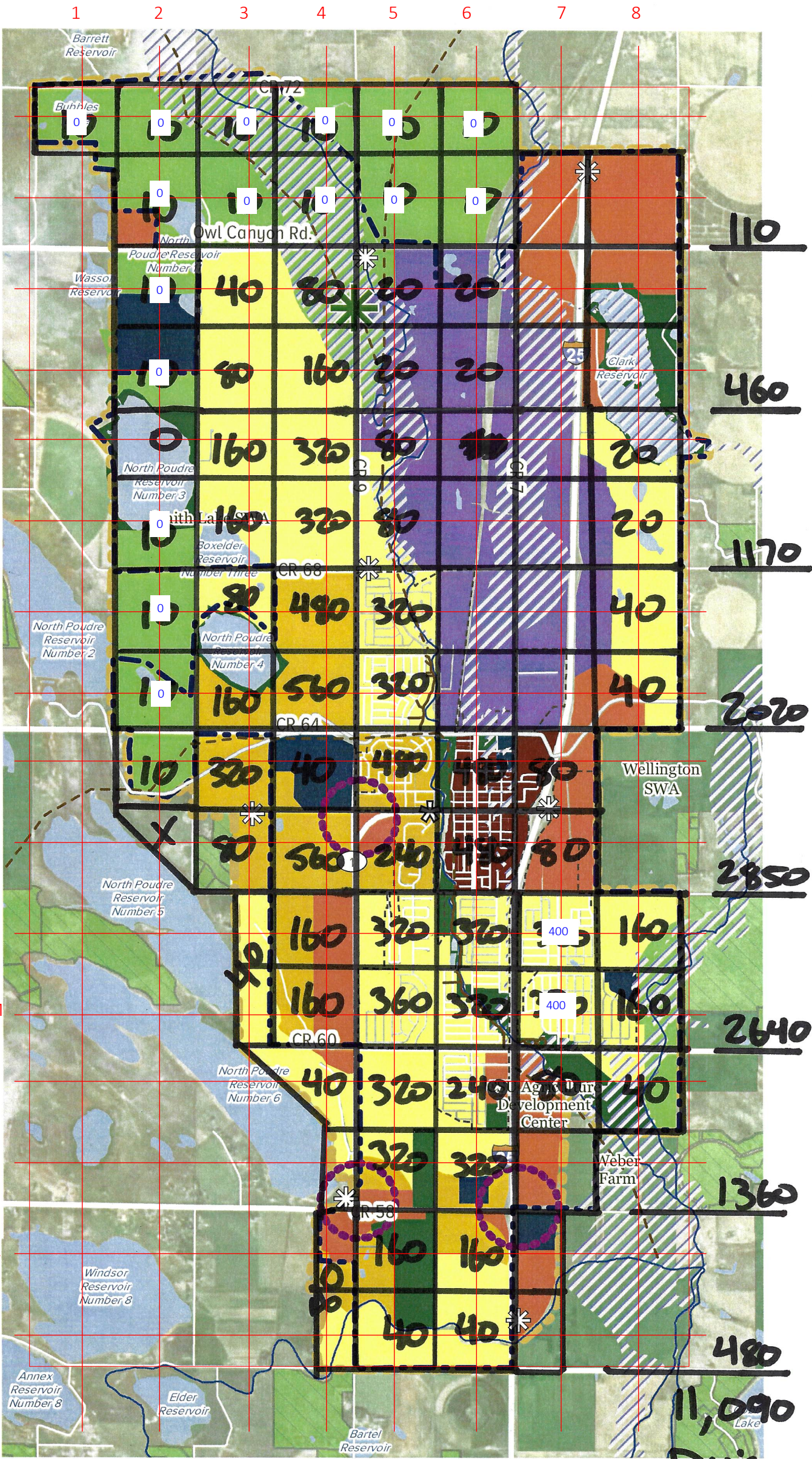
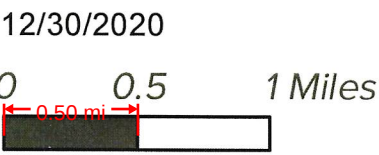
GRID NUMBERING: LETTERS FOR ROWS,
NUMBERS FOR COLUMNS

MAP #.

WELLINGTON FUTURE LAND USE

- Gateway Entrances
- Regional Park
- Existing Trails
- Proposed Trails
- Irrigation Ditches
- Streams and Rivers
- Activity Center
- 100 Yr
- City
- Growth Management Area
- Expanded Growth Management Area
- Future Land**
 - Parks and Open Space
 - Agriculture
 - Low Residential
 - Medium Residential
 - Commercial
 - Downtown
 - Industrial
 - Civic
- Conserved**
 - Private Conservation
 - State Owned Conservation

Provided for discussion purposes only. Final map/legend will have just one GMA.



(Source: Logan Simpson, 2019)

DU's

@ 3.2 PPH =
35,488 Pop.

- General Location of Creek
- Existing Town Extents
- West Region
- North Region
- East Region
- South Region

Grid	Modeled Dwelling Units	Anticipated Buildout Dwelling Units	Remaining Dwelling Units
H5	320	320	0
H6	0	0	0
J5	480	480	0
J6	255	480	225
J7	40	80	40
K5	0	240	240
K6	210	480	270
K7	80	80	0
L5	170	320	150
L6	130	320	190
L7	400	400	0
M5	60	360	300
M6	320	320	0
M7	400	400	0
N5	0	320	320
N6	130	240	110
N7	0	80	80