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South Interceptor Study

PREPARED FOR

City of North Olmsted

June 2019

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CITY OF NORTH OLMSTED
BUILDING DEPARTMENT

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Executive Summary

SUMMARY

The City of North Olmsted (City) identified the South Interceptor as an area requiring study due to wet weather flows. The City has determined through system inspection and historical observations that the South Interceptor may have additional capacity due to its alignment configuration, but also has seen increased flooding conditions with historical changes in rainfall events. It is assumed that additional capacity is available if the impacts of 90° bends, and inflow and infiltration (I/I) sources can be managed or eliminated. The City retained CT Consultants (CT) to study the South Interceptor and identify alternatives to improve the system capacity. The goal is to eliminate sanitary sewer overflow (SSO) at the manhole at Bodecker Hill (the south end of Mastick cul-de-sac) for the 10 year, 6-hour storm event.

A hydraulic model was developed during the study by CT in PCSMM utilizing historical asbuilt information for the South Interceptor. The model only included the interceptor sewer and connections to Olmsted Township and the Bradley Pump Station. Individual street connector sewers were not included in this study. The model was calibrated to flow meter data from 10 flow meters installed and maintained by the City from March to December of 2018. I/I was included in the model based upon incorporating RTK values for rain dependent inflow and infiltration RDII. The RTK values for the wet season were selected as the RDII in the existing conditions model for the 10 year, 6-hour design storm event.

The existing conditions model indicated that the SSO at Bodecker Hill would occur as well as an SSO at Barton road. Approximately 80% of the interceptor from Bodecker Hill to Barton road had less than 8 feet of freeboard during peak flows.

ALTERNATIVES

The alternative analysis included elimination of the SSO during the design storm event but also providing a minimum 8 feet freeboard in the interceptor to prevent basement backups. The following 5 alternatives were considered:

Alternative 1 – Provide additional capacity parallel to the Mastick Road 24-inch sewer from Shepard Lane up to Grace Road.

Alternative 2 – Provide equalization storage in the proximity of the soccer field at the North Olmsted High School.

Alternative 3 – Provide equalization storage at 2 locations in the proximity of the soccer field at the North Olmsted High School and the woods near Spruce Elementary School.

Alternative 4 – Provide equalization storage at 2 locations in the proximity of the soccer field at the North Olmsted High School and the woods near Spruce Elementary School and modifying hydraulic limitations at system bends.

Alternative 5 – Reduce flow by I/I reduction throughout the interceptor service area.

COST

A construction cost for each alternative was developed based on 2019 dollars. Each alternative construction cost estimate was provided with a AACE class 5 contingency value of 40% for study level planning and a 25% multiplier for project costs.

Alternative Cost Estimate

Item	Multiplier	Alternative				
		1	2	3	4	5
Construction Cost		$0.35 M	$6.42	$7.42	$7.64	$27.62
AACE Class 5 Contingency	40%	$0.50 M	$8.98	$10.38	$10.70	$38.67
Project Costs	25%	$0.62 M	$11.23	$12.98	$13.37	$48.34

CONCLUSION

The study indicates that Alternatives 4 and 5 are two feasible alternatives for managing the capacity of the Southern Interceptor to the 10 year 6-hour design event. The two alternatives include multiple equalization basins or I/I removal with a 50% flow reduction. The equalization alternative is less expensive at $13.4 M, but has new continuing operational and maintenance costs and continuing wet weather volume treatment costs at the waste water treatment plant. Additionally, the system could be impacted by system age, which could create more I/I over time.

I/I removal has a much higher cost at $48.3 M and will involve working on private property. The advantage of this option is that there can actually be a return on investment due to decreased treatment costs as well as creating capacity for future development opportunities whether they be from within the City limits or from surrounding communities.

The City should review the alternatives and consider the policies required to achieve these alternatives to decide a chosen alternative or a possible combination of the two alternatives.

Depending on funding availability and schedule for the chosen alternative, the City may want to consider implementation of Alternative 1. Alternative 1 can have a localized impact of reducing the SSO and may be able to be implemented more quickly due to cost than the more regional impacts of Alternatives 4 and 5 which also reduce the SSO impact.

ADDITIONAL NOTE

The City developed a 2019 flow metering program during the development of this report. The 2019 metering program is not a part of this study, however, CT did provide a cursory review of the data to verify model calibration where the 2018 metering program had metering challenges. The 2019 program utilized different locations for metering based upon 2018 findings. A flow difference issue was identified in the 2019 data that suggests further investigation. It appears that somewhere between Burns Road and the Springvale metering site (near Centerbury Road) the flow decreases during dry weather flows.

The City should continue to utilize meters to further investigate the flow decrease to determine if this is a sewer issue or a metering issue. Additional flow metering can also determine if the system blockage upstream of Bodecker Hill is a systemic issue or a single event. Televising this investigation area could also provide further insight into these issues.

1.0 Introduction

1.1 BACKGROUND

The City of North Olmsted (City) identified the South Interceptor as an area requiring study due to wet weather flows. The City has determined through system inspection and historical observations that the South Interceptor may have additional capacity due to its alignment configuration, but also has seen increased flooding conditions with historical changes in rainfall events.

The City has identified several concerns within the South Interceptor system to be considered during the study:

- The interceptor has numerous 90° bends that may be developing significant head loss conditions.
- The interceptor crosses under a listed floodplain that could have an I/I impact.
- Houses built in the area of service are suspected for a high rate of footer/foundation drain tile connection to the sanitary system.
- A high percentage of subgrade driveways exist in the service area with drains that may be connected to the sanitary system.
- The manhole at Bodecker Hill (the south end of Mastick cul-de-sac) is a low point manhole that has experienced an SSO release in 2011 and 2014. Three additional low point manholes in the area have bolt down manhole covers to control the release location.

The purpose of studying the South Interceptor is to improve customer service by reducing sanitary sewer overflow (SSO) events near the south end of Mastick (cul-de-sac) and reducing system flooding. The City intends this study to investigate improving the level of service of the interceptor by analyzing sewer configuration improvements that create backwater conditions, adding equalization basins, and initiating an inflow/infiltration (I/I) removal program.

2.0 Data Collection

2.1 HISTORICAL DATA

The City maintains an AutoCAD map of the sanitary sewer system. CT converted this into a GIS map with conversion of the manhole labels into a GIS database and provided a GIS based map to the City for future use as a sewer system reference map. This system map is provided in **Figure 2-1**. The South Interceptor portion of this map is provided in **Figure 2-2**.

The City also provided historical sewer drawings of the South Interceptor and pertinent sewers for developing a hydraulic model of the interceptor system.

CT has recently worked with the City on upgrading the Bradley Pump Station. CT incorporated the pump curves, operations and wet well information from that project in the hydraulic model development.

2.2 FLOW MONITORING

The City owns a combination of rain gauges and Sigma 910 and FloDar flow meters for use in various flow metering programs they have performed over the years. The City staff is responsible for installation and maintenance of the flow metering equipment. The equipment included 2 rain gauges and 10 flow meters. CT assisted the City in identifying meter locations and performed data review during the collection period. Data was collected from February 21 to December 3, 2018. The location of the metering sites is provided in **Figure 2-2**.

2.2.1 Data Quality

During the data collection period, various meters had issues that were reviewed to identify periods of poor data quality. Three of the sites with data quality issues were McKenzie, Elizabeth and Golf Club. McKenzie had two issues during April and May in which the velocity maximized during surcharge depth events. The flows during this period were considered in error. Site maintenance may have corrected the velocity issues after June 2018, however, no events appeared to develop sewer surcharge depths after June.

Elizabeth flow data was determined to be poor quality when it was determined that the site installation was not measuring viable velocity data due to manhole turbulence. Depth data was considered reasonable for use as calibration information since flow turbulence does not affect depth measurement.

Golf Club flow meter data was determined to be of poor quality due to its flow pattern being dissimilar from upstream and downstream meter data during periods of backwater

surcharge conditions. The meter was directly upstream of the Springvale site, but Golf Club meter data registered flows 20% higher than the downstream meters. This is not unusual since meters can be +/-20% of each other, however, the inconsistency in flow patterns with the other meters and FloDar meters being considered a +/-15% accuracy range caused the meter and its data to be considered unusable.

The rain gauge located at Bradley Pump Station malfunctioned from October to December, 2018. The rain gauge information was supplemented with rainfall data from the wastewater treatment plant location for modeling purposes. The rain gauge located at the WWTP malfunctioned during July. The missing information was supplemented with rainfall data from the Bradley Pump Station location.

2.2.2 Data

The rainfall data for the study period are provided in **Appendix A**. A total of approximately 76 rainfall events occurred during the data collection period. Six of these events were considered significant events in which the system surcharged. **Table 2-1** lists the significant events.

Table 2-1
Flow Metering Period Significant Events

Event	Date	Duration (h)	Maximum Rainfall (in/hr)	Mean Rainfall (in/hr)	Total Rainfall (in)
9	4/15/2018 2:00	27	0.4	0.07111	1.92
21	5/22/2018 8:20	4.75	3.96	0.2274	1.08
25	6/18/2018 20:45	9.42	3.6	0.1965	1.85
37	8/6/2018 22:20	7.25	5.4	0.331	2.4
54	9/26/2018 1:35	5.58	2.04	0.2651	1.48
63	10/31/2018 23:50	23.17	0.84	0.1079	2.5

Each of the significant events was reviewed against the NOAA Rainfall Atlas 14 database to identify each events rainfall return period. The event return periods are provided in **Table 2-2**.

Table 2-2
Significant Rainfall Events Return Period

Event	Date	Return Period
9	4/15/2018 2:00	0.9 year - 27 hour
21	5/22/2018 8:20	0.4 year - 4.8 hour
25	6/18/2018 20:45	0.3 year - 8.8 hour
37	8/6/2018 22:20	5.5 year - 7.3 hour
54	9/26/2018 1:35	1.0 year - 5.6 hour
63	10/31/2018 23:50	2.1 year - 23.2 hour

The flow meter data for the study period are provided in **Appendix B**. The rainfall events generated approximately 70 wet weather flow events for analysis review and considered for model calibration. Initial calibrations were based upon the spring wet weather events from March to June. After the initial model calibration discussed in the next section, it was determined that the flow and corresponding system losses during this period may not be typical system characteristics. The metered peak flows and depths indicated significant system head losses which caused significant backwater surcharging depths that were not consistent with the constructed sewer capacities. The additional data collected from June to December provided different system characteristics and a different calibration.

The April and May significant rain events were smaller than most of the other significant events and developed slightly lesser peak flows, but developed depths twice as deep at the Springvale, Southern, Stearn, Barton, and McKenzie meters. After modeling several possible scenarios such as creek inflow, outside customer significant inflow, and high manhole losses at bends, it is assumed that there was a temporary blockage in the system between the Springvale and Mastick24 flow meters.

The City regularly performs field observations of a low point manhole on the east side Cul-de-Sac at Columbia and S.R. 252 known as Bodecker Hill. Information provided by the City indicated that the low point manhole did not surcharge above the manhole rim during the flow meter period. They did indicate that the elevations within the manhole were close to the rim, but no overflows occurred. Based upon model response in this area, it is further assumed that the temporary blockage occurred between this location and the Springvale meter. It is assumed from the metering data that the blockage cleared out naturally during the month of May.

Flow meter data does indicate that there are still restrictions to flow during sewer surcharge conditions throughout the interceptor. Under sewer surcharge conditions the depth increases and velocity decreases even in the June to December data, just not as significantly as the April to May events. This occurs regularly at all of the meters. The initial consideration for this system response is the interaction of flow within manhole bends along the interceptor's alignment. It is assumed that swirling waters in a manhole bend develops restrictions to the flow.

2.2.3 Additional Data

The City determined that additional flow metering was to be performed in 2019. Locations were identified to further enhance the data collection of 2018 or provide new information

at new locations that would further characterize the systems wet weather response. This data is not utilized within this report for hydraulic model development and system characterization. However, a cursory review of the data was performed to validate some model calibration information and is further discussed in **Section 6**. The location of the proposed 2019 flow metering is provided in **Figure 2-2**.

2.3 INFLOW/INFILTRATION RESPONSE

The flow meter data has been reviewed to determine the wet weather response of the service area between each flow meter location. Based upon the flow meter data indications of backwater response, developing inflow/infiltration (I/I) response based on flow meter data is not feasible. The development of peak to average day flow multipliers by service area would be skewed due to the backwater impacts developing prior to peak flow development. I/I response by meter service area will be developed within the hydraulic analysis, **Section 3.0**, of this report.

3.0 Hydraulic Analysis

The hydraulic model was developed utilizing PCSWMM software developed by Computational Hydraulics, Int. It utilizes the United States Environmental Protection Agency's (USEPA) Storm Water Management Model (SWMM) as the platform to perform the model hydraulic analysis. It also provides user friendly graphical tools for analyzing the results. These include mapping color coding and labeling, time based hydraulic grade line review, and data graphing. Other tools include data analysis to define exceedance occurrences and volumetric analysis.

SWMM is a dynamic simulation model used for single-event and long-term precipitation, runoff, and collection system modeling. It is one of the most common publicly available software programs used for analysis, design, and planning of storm water collection systems, combined sewer systems, and sanitary sewer collection systems. SWMM is used to simulate catchment area hydrology and conveyance system hydraulics from primarily urban areas, although suburban and rural areas have been successfully modeled in SWMM as well.

The general operation of the model is relatively simple and intuitive. During a simulation, precipitation is simulated over hydrologic subcatchments and excess precipitation (that which exceeds the infiltration rate of a given subcatchment) becomes runoff. The runoff is computed by a non-linear reservoir routing equation and is then hydraulically routed through a collection system by applying the continuity equation coupled with the momentum equation or full St. Venant dynamic flow routing equation. During a simulation, SWMM performs the hydrologic and hydraulic calculations at regular time intervals and tracks the quantity and quality of runoff from each subcatchment, the flow rate, flow depth, and quality of water in each node and link.

This allows SWMM to model various flow regimes, such as backwater, surcharging, and reverse flow. SWMM is preloaded with many different standard closed conduit and open channel shapes and has the ability to allow the user to create unique cross-sections for natural channels or other unique conveyance pathways. Users can also specify dynamic controls in the system such as gates or valves opening or closing in real-time.

3.1 MODEL DEVELOPMENT

Development of the sewer model utilized City provided manhole location from the City AutoCAD base map which CT developed into Geographical Information System (GIS). The GIS data did not provide sewer inverts. This data was developed through review of historical as-built plans. A detailed model that included The South Interceptor from Bradley Pump Station to the WWTP intake was developed for this study. The model also included two lateral connections to Olmsted Township. A layout of the sanitary sewer system model is provided in **Figure 3-1**.

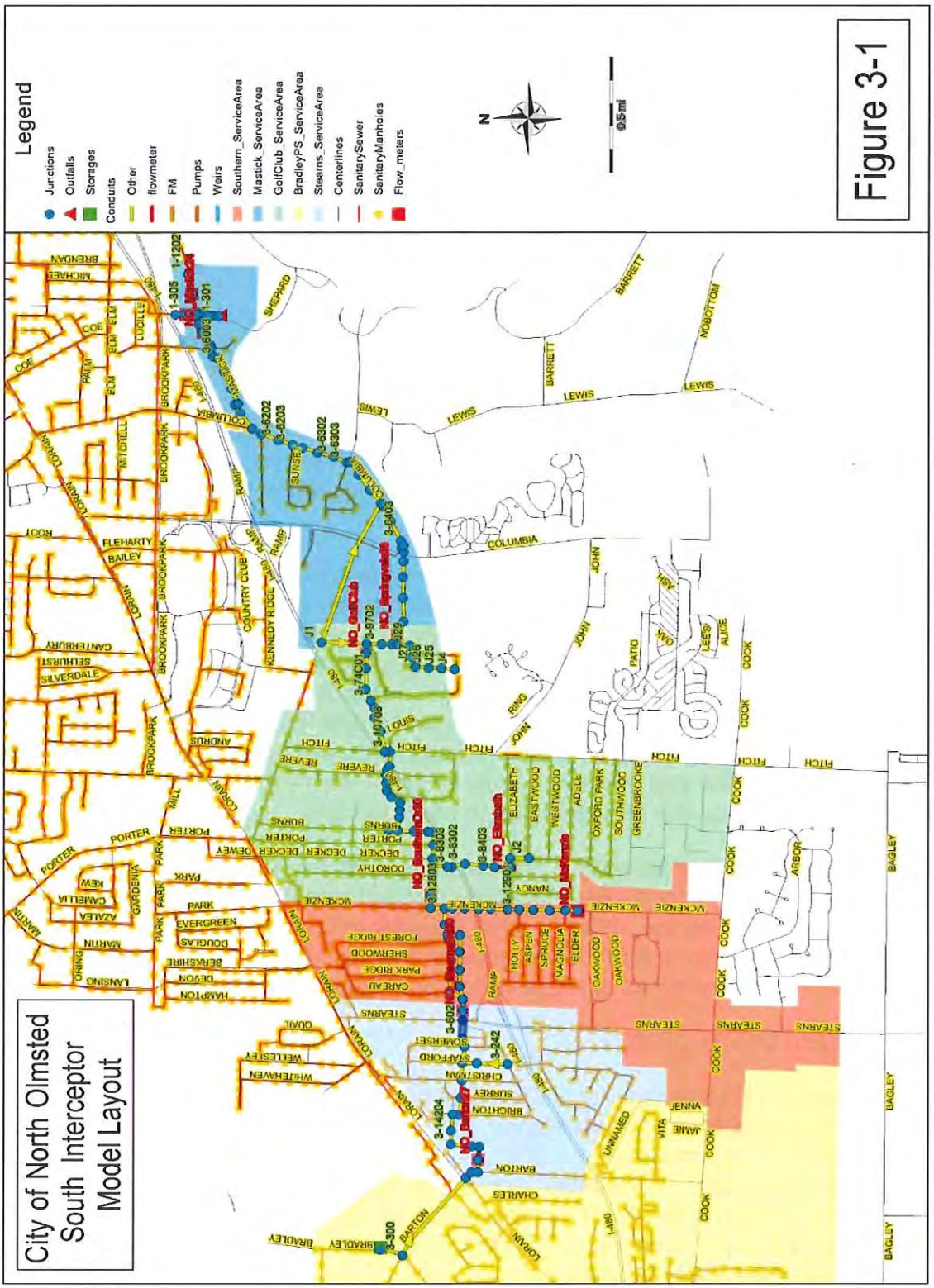


Figure 3-1
Legend
Junctions
Outfalls
Storages
Conduits
Other
flowmeter
FM
Pumps
Weirs
Southern_ServiceArea
Mastick_ServiceArea
GolfClub_ServiceArea
BradleyPS_ServiceArea
Stearns_ServiceArea
Centerlines
SanitarySewer
SanitaryManholes
Flow_meters
N
0.5mi
City of North Olmsted
South Interceptor
Model Layout

3.2 CALIBRATION

Calibration consists of identifying key flows into the system and relating them to rainfall data. The key flows included in this model include:

- Daily/hourly wastewater usage
- Baseline dry weather infiltration
- Rainfall Dependent I/I (RDII)

RDII is further broken down into 3 components:

- Short Term – typically direct inflow that peaks and declines in a matter of minutes or hours.
- Medium Term – typically ground infiltration that finds a route to a direct inflow connection that peaks and declines in a matter of hours or few days.
- Long Term – typically ground infiltration that peaks in a matter of hours to a day and declines over a few days to weeks.

In order to identify these system flows, the sewer system was divided into subcatchment areas and flow metered in conjunction with rainfall metering. The flow meter data was broken down for each district to identify the listed components and utilized as a starting point to begin the calibration. The model analysis represents the RDII with unit hydrograph RTK parameters for each component. The values represent the following:

- R – Percent of rainfall captured for I/I
- T – Time to peak for the unit hydrograph
- K – Ratio or rising to falling limb of the unit hydrograph

The calibration progresses by applying adjustment factors to baseline infiltration and average daily usage for dry weather calibration and then adjusting the appropriate RTK factors to better represent the wet weather system response to rainfall events. The model was considered calibrated when the modeled flow patterns resembled meter flow patterns and peak flows and volumes for multiple rain events were within +/- 20%.

3.2.1 Calibration Results

The calibration results are provided in **Appendix C**. The calibration reviews maximum flow and total volume based upon events from March to December, 2018, however, depth

calibration only considered events from June to December of 2018. Modeling indicated that the maximum flow may occur just prior to backwater impacts from the assumed blockage. This differs from the assumption stated in **Section 2.3** for developing I/I impacts based upon metering data. However, the statement in **Section 2.3** still holds due to the sewer system dynamics of travel time between meters and how short the time period is between peak flow and backwater impacts in the meter data.

It should be noted that the McKenzie and Elizabeth calibrations are based only on depth response to the wet weather events due to velocity issues discussed in **Section 2.0**. These sites were reviewed based upon June to December 2018 events due to possible impacts by the assumed blockage in April and May.

The flow pattern during backwater conditions were difficult to replicate and are not exact. This is due to modeling limitations on how SWMM can simulate losses at manholes and the variability in sewer system bend manhole losses associated with hydraulic flow coming into a manhole, turbulence or swirling conditions and exit flow impedance. Further investigation/metering may provide better insight to develop a hydraulic flow/head loss pattern reflective of the system conditions. Due to the model limitations, the manhole losses due to bends were simulated with pipe entrance losses for the sewer pipe downstream of each bend. This method can only provide a constant increase of head loss based on increased flow. This was determined to be the best method for representing bend manhole hydraulics since development of a hydraulic flow/head loss pattern would be a detailed hydraulic study of significance.

3.2.2 I/I Evaluation

The I/I impact of each metering district was evaluated based upon the wet season RTK values utilized for the existing conditions model. The evaluation was based upon the Total R captured per estimated service area of each district. The Barton District is not included in the evaluation due to the Bradley Pump Station impacts on the service area. The RTK values for the Barton District were developed based on replicating wet weather pumping cycles since no meters were placed above the pump station. The rankings are provided in **Table 3-1**. The highest I/I service area is the Springvale service area. Additional metering should be performed to better define the impacts of Olmsted Twp and McKenzie. The Mastik24 service area ranking may change as well due to the discussion in **Section 6**.

Table 3-1
I/I Service Area Rankings

Service Area	Ranking
Springvale	1
Elizabeth	2
Stearns	3
Olmsted Twp	4
Southern	5
McKenzie	6
Mastick24	7

3.3 DESIGN STORM EVENT

The City has determined that the design storm event that should be utilized for developing sanitary sewer system capacity is the 10-year 6-hour event. This is the same design event utilized for the equalization basin design at the Dover Pump Station. The 10-year 6-hour event has a total rainfall of 2.57 inches.

3.4 EXISTING SYSTEM ANALYSIS

The existing system model is developed utilizing the design storm event with RTK unit hydrograph values based upon wet season conditions. For this model the RTK values for the months of March, April, May, and November were averaged together and utilized to establish the existing conditions RTK unit hydrograph for existing conditions wet weather response.

3.4.1 Existing Conditions Results

The existing conditions model for the design storm event develops a peak flow of approximately 20 mgd and develops a hydraulic grade line (HGL) that activates the Bodecker Hill SSO location for 3,000 gal SSO.

The upper end of the interceptor also overflows at the 90-degree bend on Barton Road. Observations of the sewer south of this location indicated that flows due to the Bradley Pump Station discharge could impact the southern sewer discharge to the interceptor. The City modified the manhole at Barton Road to a wye configuration rather than a Tee configuration which appeared to alleviate this issue. However, during the design event,

system surcharging occurs and an overflow event at the Barton Road bend is predicted. The overflow volume during the design storm event is approximately 0.6 mg, which is larger than the event that occurs at Bodecker Hill.

It should be noted that during the design storm event, the Bradley Pump Station appears to operate at full capacity. The model hydraulics indicate that the station reached a peak capacity of approximately 5.6 mgd. It is assumed that the station does not reach a peak capacity of 6 mgd due to backwater HGL at the discharge location.

The peak HGL profile for the interceptor is provided in **Figure 3-2**. In addition, the design storm flows at the flow meter locations are provided in **Graph 3-1**. The data indicates that the wet weather impact generates flows greater than the interceptor's capacity from Shepard lane to the parallel pipe section in the Winsor Drive area.

Assuming an 8 feet freeboard is needed to prevent basement backups, almost all of the system upstream of Bodecker Hill has less than 8 feet of freeboard.

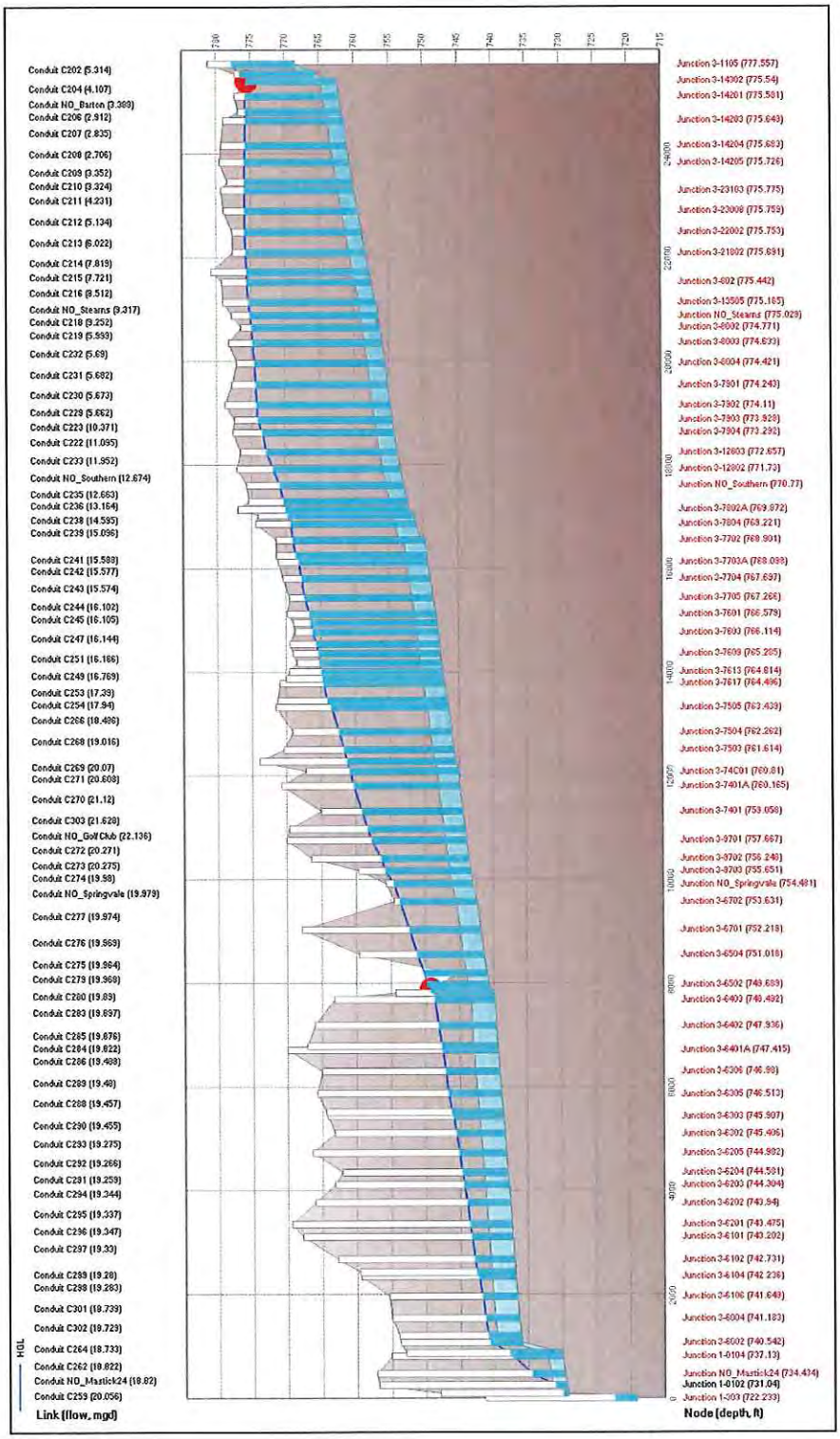
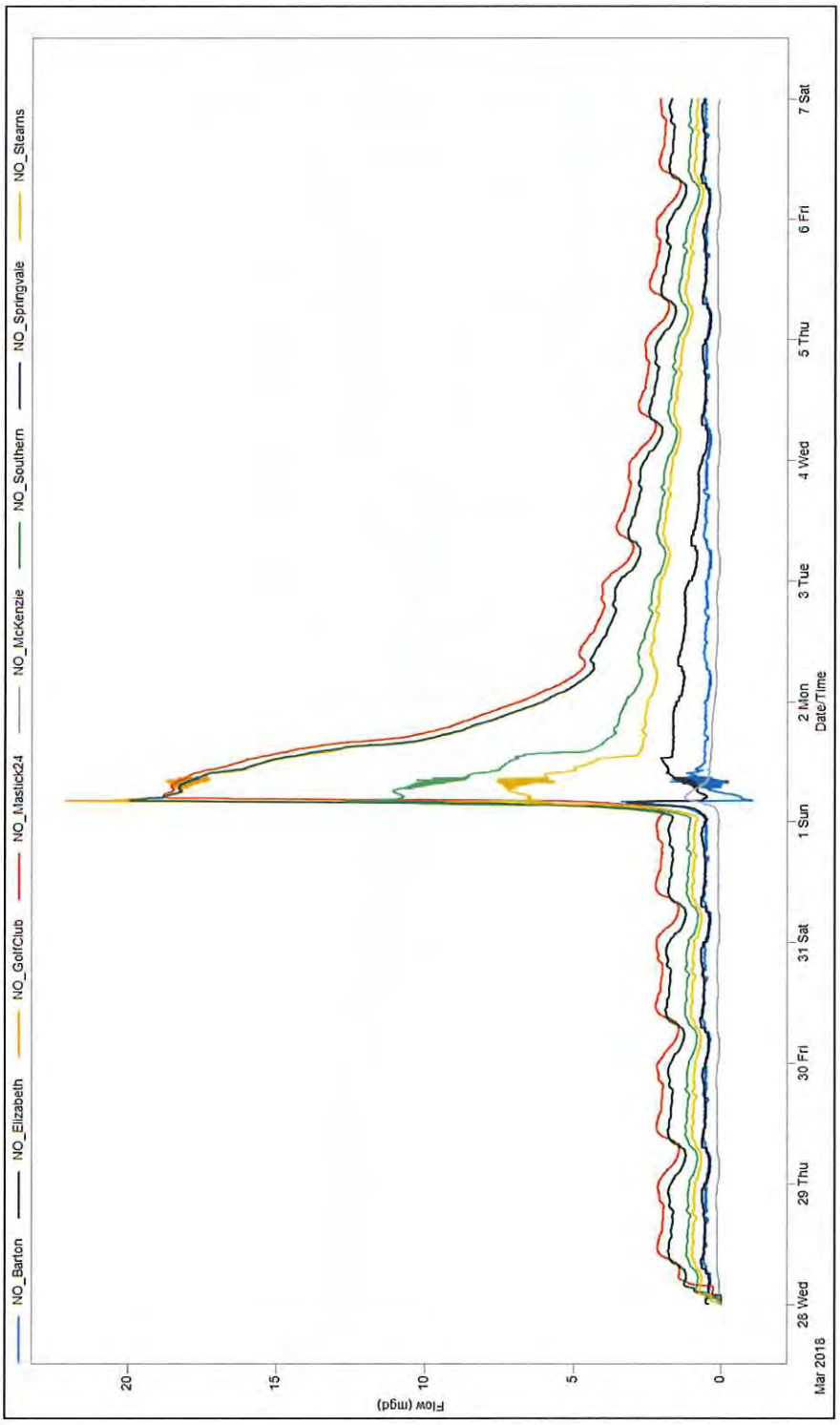


Figure 3-2
Existing Conditions
Maximum Hydraulic Grade Line

Graph 3-1
Existing Conditions
Maximum Flow



4.0 Alternative Review

Upon review of the existing conditions, several alternatives were considered in providing a sewer system that prevented basement backups or SSO's during the design storm event. The alternatives considered are as follows:

Alternative 1 – Provide additional capacity parallel to the Mastick Road 24-inch sewer from Shepard Lane up to Grace Road.

Alternative 2 – Provide equalization storage in the proximity of the soccer field at the North Olmsted High School.

Alternative 3 – Provide equalization storage at 2 locations in the proximity of the soccer field at the North Olmsted High School and the woods near Spruce Elementary School.

Alternative 4 – Provide equalization storage at 2 locations in the proximity of the soccer field at the North Olmsted High School and the woods near Spruce Elementary School and modifying hydraulic limitations at system bends.

Alternative 5 – Reduce flow by I/I reduction throughout the interceptor service area.

4.1 ALTERNATIVE 1

Alternative 1 reviews the impact of increasing the capacity of the 24-inch portion of the sewer from Shepard Lane to Grace Road along Mastick Road. This alternative lowers the HGL to the SSO location at Bodecker Hill such that it does not activate. However, almost all of the system upstream of Bodecker Hill has less than 8 feet of freeboard.

The upper end of the interceptor overflows at the 90-degree bend on Barton Road in this alternative. The overflow volume during the design storm event is approximately 0.6 mg. The peak HGL profile for the interceptor is provided in **Figure 4-1**.

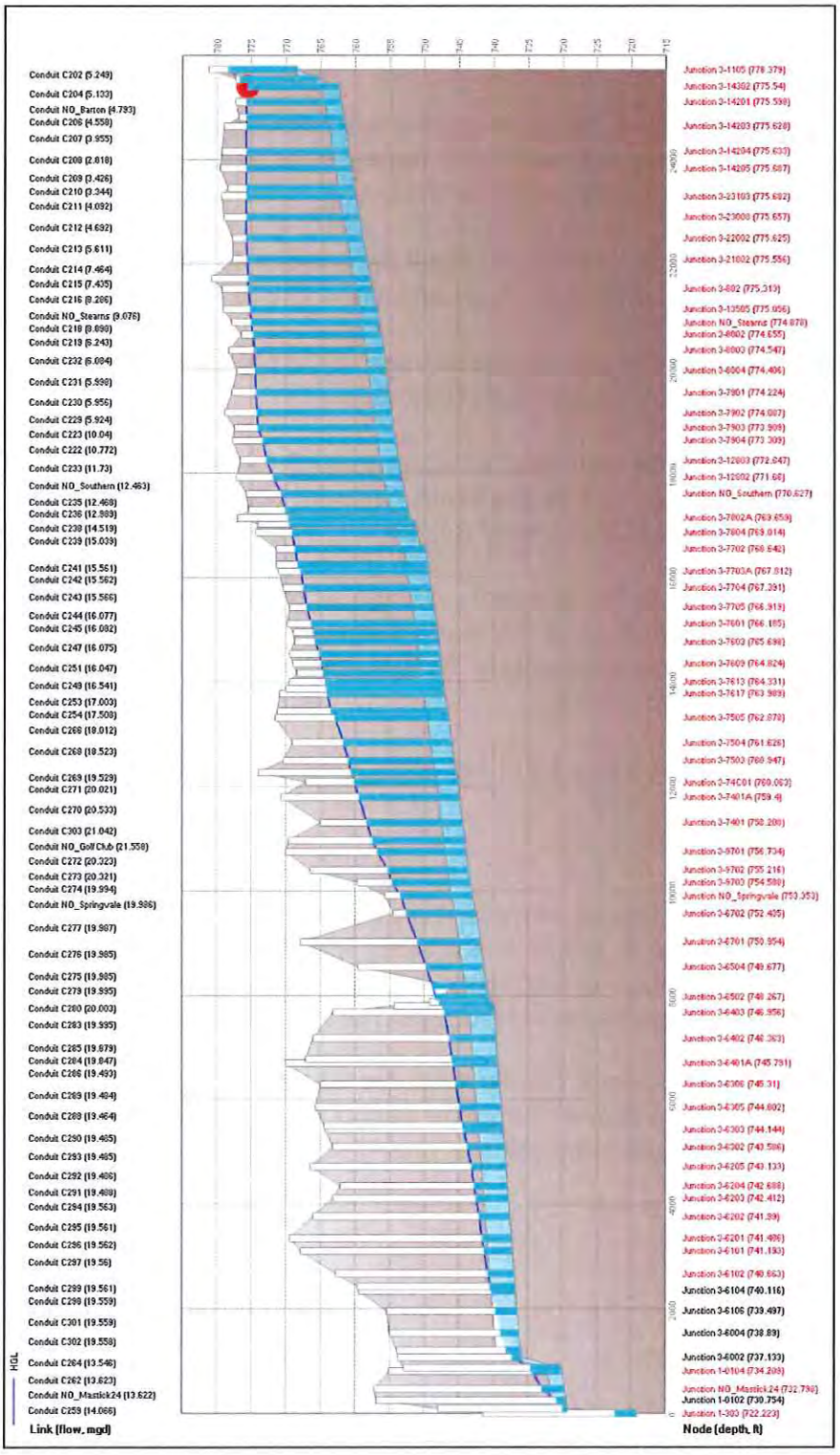


Figure 4-1
Alternative 1
Maximum Hydraulic Grade Line

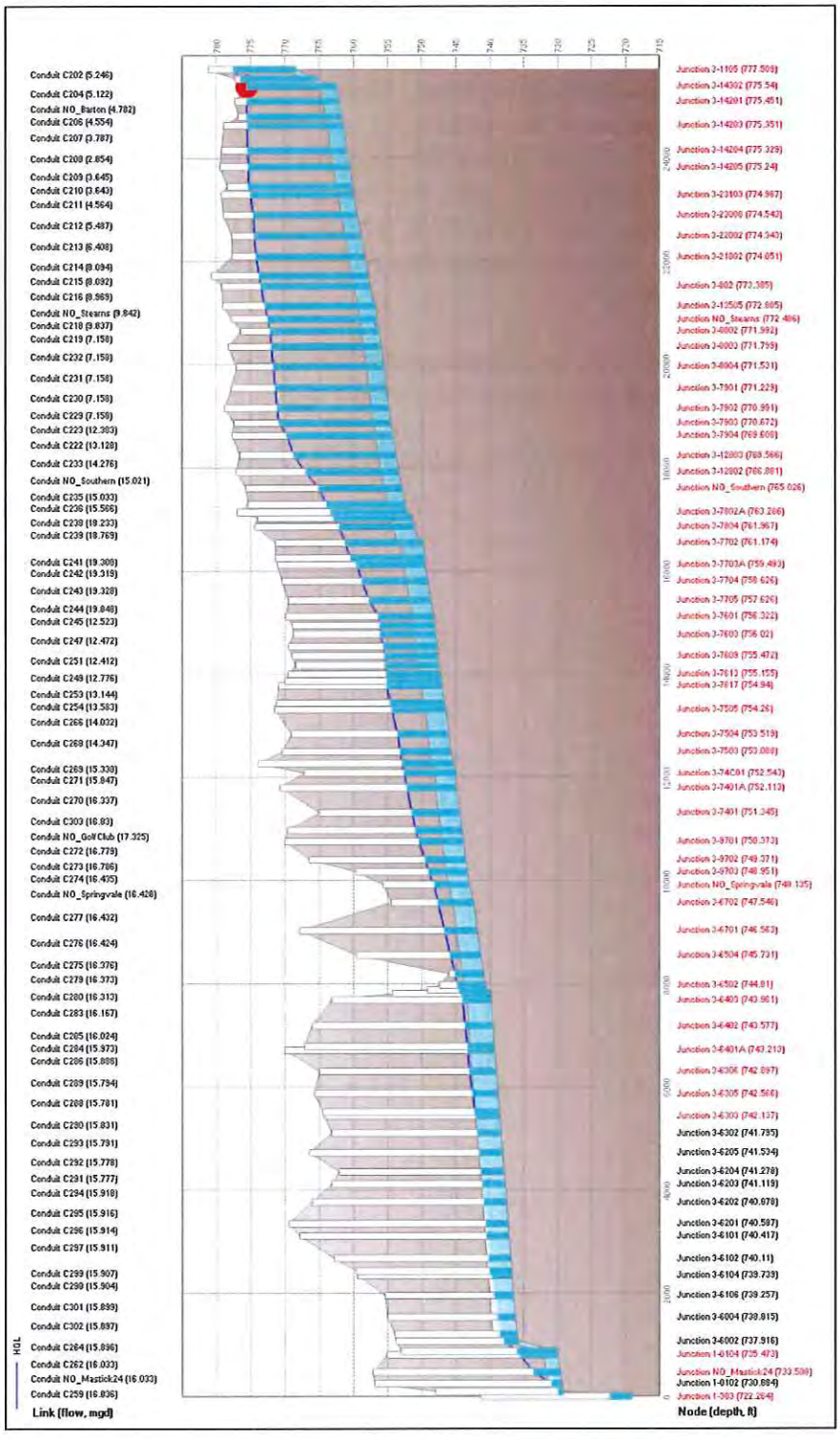
4.2 ALTERNATIVE 2

Alternative 2 reviews the impact of providing an equalization basin in the area of the soccer field at the North Olmsted High School. This location was considered due to space and upstream flow impacts to the Elizabeth/Olmsted Township connection. The overflow to the basin at this location is 7 feet above the existing interceptor and the resulting basin size is 1.5 mg. This alternative eliminated the need for the parallel sewer discussed in Alternative 1. This alternative lowers the HGL to the SSO location at Bodecker Hill such that it does not activate. It also lowers the HGL upstream to McKenzie Road to a freeboard of greater than 8 feet.

The upper end of the interceptor overflows at the 90-degree bend on Barton Road in this alternative. The overflow volume during the design storm event is approximately 0.2 mg.

The peak HGL profile for the interceptor is provided in **Figure 4-2**.

Figure 4-2
Alternative 2
Maximum Hydraulic Grade Line



4.3 ALTERNATIVE 3

Alternative 3 reviews the impact of providing an additional equalization basin in the wooded area near the Spruce Elementary School. The Alternative 2 equalization basin in the area of the soccer field at the North Olmsted High School is included in this alternative. The overflow to the basin at this location is 7 feet above the existing interceptor similar to the soccer field site and the resulting basin size is 0.75 mg. The soccer field basin size decreases to 1.0 mg. This alternative eliminated the need for the parallel sewer discussed in Alternative 1. This alternative lowers the HGL to the SSO location at Bodecker Hill such that it does not activate. It also lowers the HGL upstream to Stafford Drive to a freeboard of greater than 8 feet.

The upper end of the interceptor still overflows at the 90-degree bend on Barton Road in this alternative. The overflow volume during the design storm event is approximately 0.03 mg.

The peak HGL profile for the interceptor is provided in **Figure 4-3**.

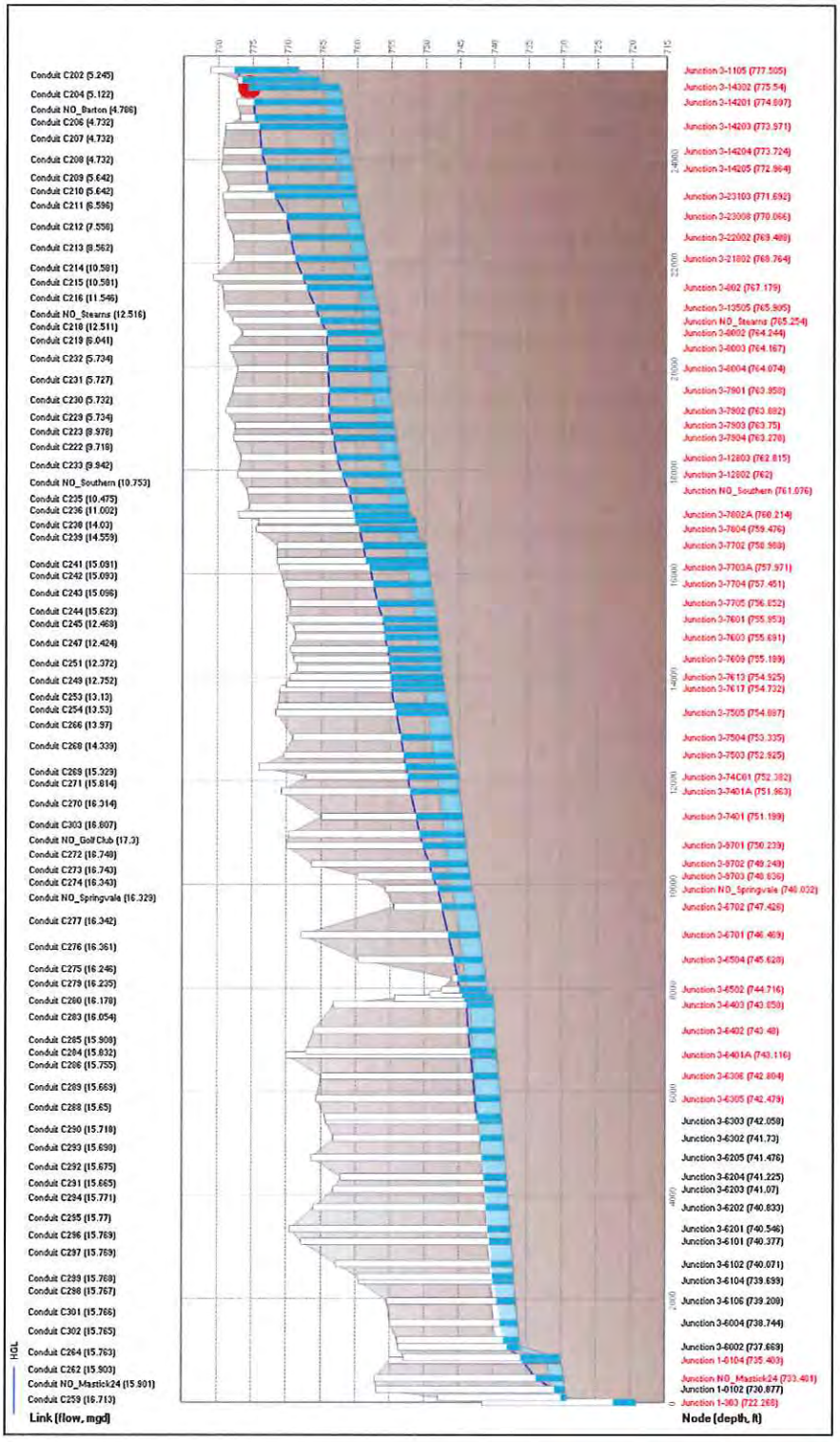


Figure 4-3
Alternative 3
Maximum Hydraulic Grade Line

4.4 ALTERNATIVE 4

Alternative 4 reviews the impact of providing sewer bend efficient manholes upstream of Stafford Drive. Calibration indicates that there are significant head losses in the sewer system between the Stearns and Barton flow meter sites. It is assumed that the combination of the pump station flows, sewer size and additional I/I flows develop a larger head loss at bend manholes due to the energy and turbulence in the system flows. With 2 pumps in operation at the Bradley Pump Station, the resulting flows utilize approximately 70% of the 30-inch sewer capacity. It is assumed based on calibration, that this flow creates significant losses at the manhole bends. The additional I/I impacts of the Stearns flow meter service area create additional losses due to exceeding the sewer capacity. The combination of these system losses generate an overflow at the Barton Road location. All 90-degree bend manholes and Tee manholes are assumed to be modified with lesser degree bends and wye manholes to eliminate significant hydraulic capacity losses due to turbulence. Sewer pipe entrance losses were reduced to model these impacts.

The equalization basins in Alternative 3 are included in this alternative. The Spruce Elementary and soccer field basin sizes are 0.8 mg and 1.0 mg, respectively. This alternative eliminated the need for the parallel sewer discussed in Alternative 1. This alternative lowers the HGL at the SSO location at Bodecker Hill such that it does not activate. While this alternative eliminates the overflow at Barton Road the freeboard conditions remain similar to Alternative 3. The HGL freeboard of greater than 8 feet is limited to Stafford Drive and downstream.

The upper end of the interceptor no longer overflows at the 90-degree bend on Barton Road in this alternative. However, the freeboard is less than 8 feet due to capacity of the sewers being similar to the upstream pump station capacity from Stafford Drive to Barton Road.

The peak HGL profile for the interceptor is provided in **Figure 4-4**. Since this alternative appears to provide the best HGL/freeboard of the first 4 alternatives, the design storm flows at the flow meter locations are provided in **Graph 4-1**.

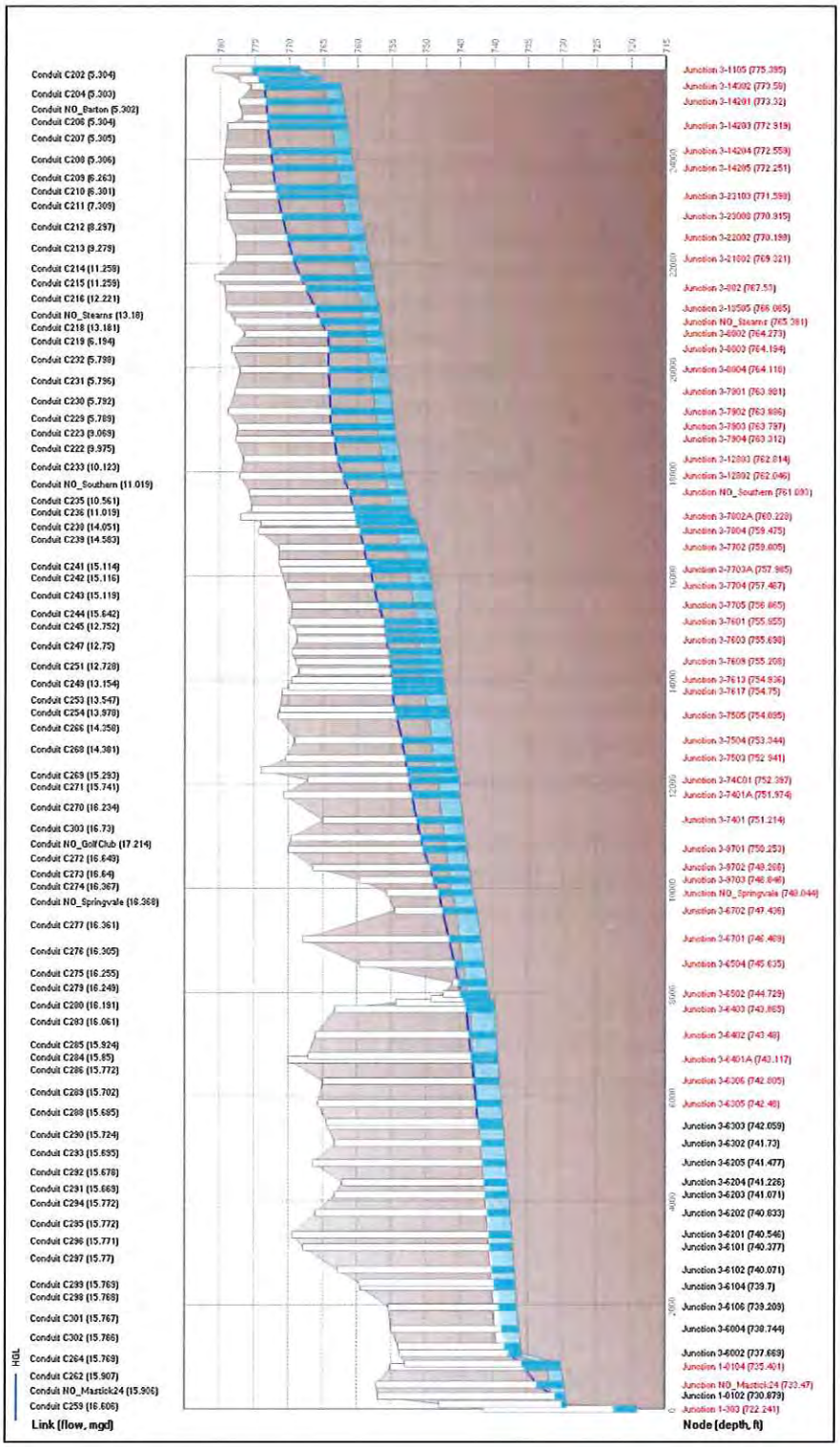
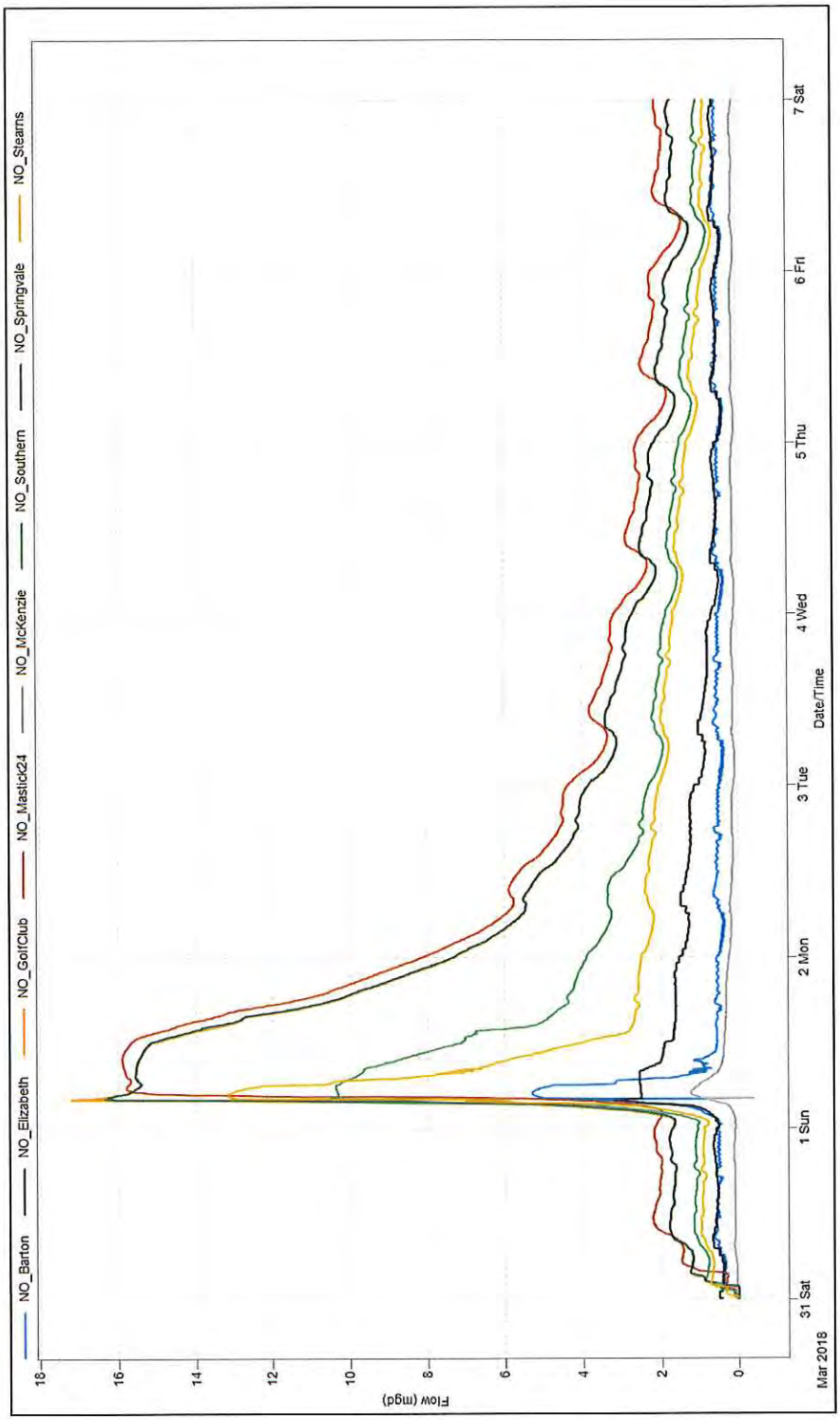


Figure 4-4
Alternative 4
Maximum Hydraulic Grade Line

Graph 4-1
Alternative 4
Maximum Flow



4.5 ALTERNATIVE 5

Alternative 5 reviews the impact of developing an I/I reduction program. This is modeled by reducing the R factor of the unit hydrographs. Calibration indicates that the Stearns, Southern, Elizabeth, and Springvale flow meter service areas have the largest I/I impact on the system.

It is assumed that the Elizabeth service area will be difficult to reduce I/I impacts from due to it being a Olmsted Township service area. It should be noted that while they do have a significant I/I impact; it is limited by the City 15-inch sewer capacity downstream of the flow meter site. The Olmsted Township peak impact to the interceptor system is approximately 3.25 mgd.

The existing system has a peak flow of approximately 21 mgd at the time the SSO activates. Based on modeling efforts, it has been determined that the system needs to be limited to approximately 15 mgd at this location to develop similar impacts to Alternative 4. Due to capacity issues upstream of the Elizabeth connection point to the interceptor, the primary I/I flow reduction should occur along the interceptor system. A reduction from the Olmsted Township system will only have a limited impact to upstream interceptor capacity issues.

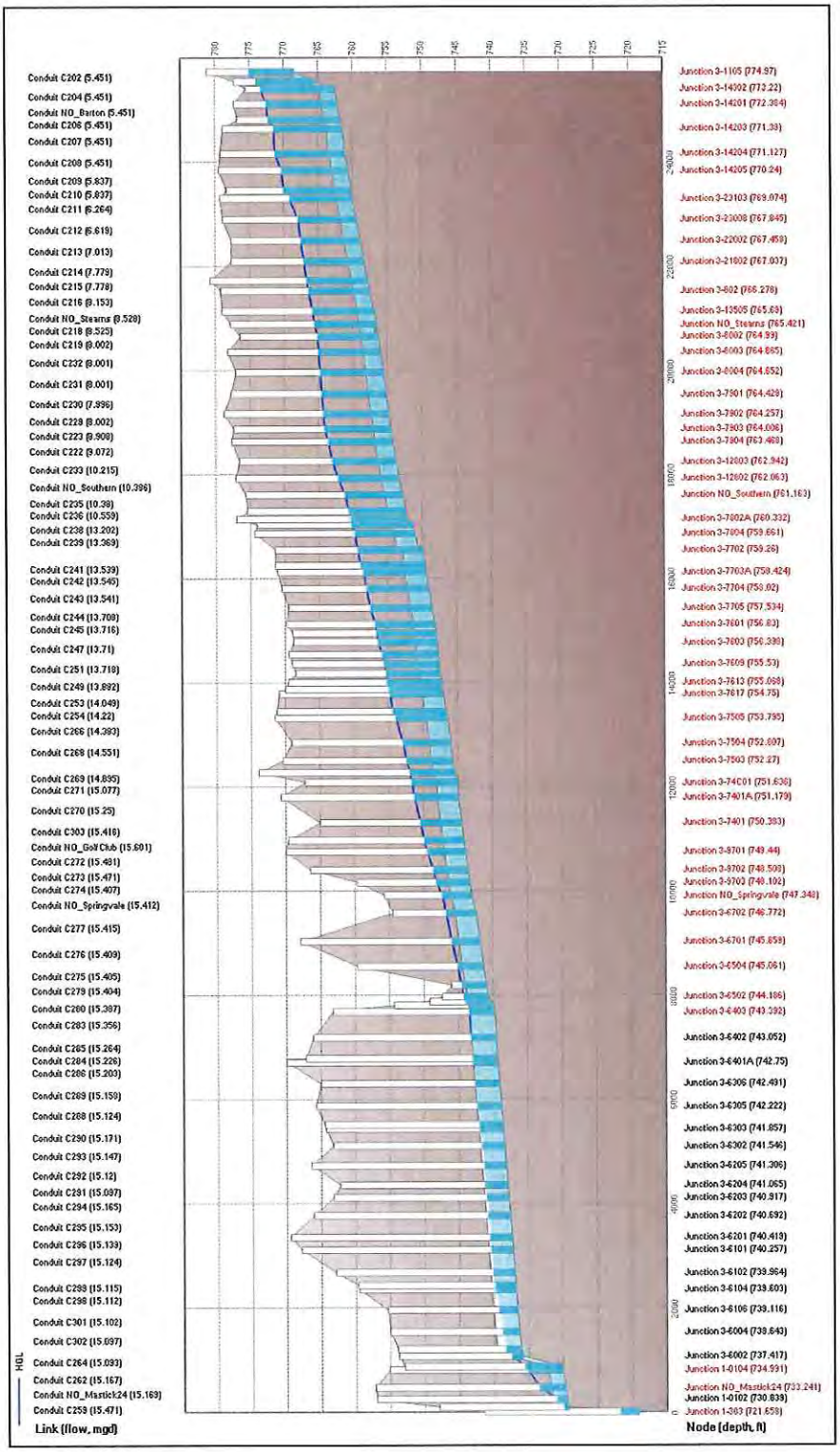
Assuming I/I reductions only within the Stearns, Southern, and Springvale flow meter service area, a 60% reduction in I/I flows would need to be achieved to develop HGL impacts similar to Alternative 4. Typically, only 10 to 15% reduction can be achieved within public right-of-way work. Some of this has already been completed in the Southern service area. It is estimated that approximately 50% of the reduction will need to be achieved through work on private property primarily in these three service areas.

The equalization basins in Alternative 3 are included in this alternative. The Spruce Elementary and soccer field basin sizes are 0.8 mg and 1.0 mg, respectively. This alternative eliminated the need for the parallel sewer discussed in Alternative 1. This alternative lowers the HGL to the SSO location at Bodecker Hill such that it does not activate. While this alternative eliminates the overflow at Barton Road the freeboard conditions remain similar to Alternative 4. The HGL freeboard of greater than 8 feet is limited to Stafford Drive and downstream.

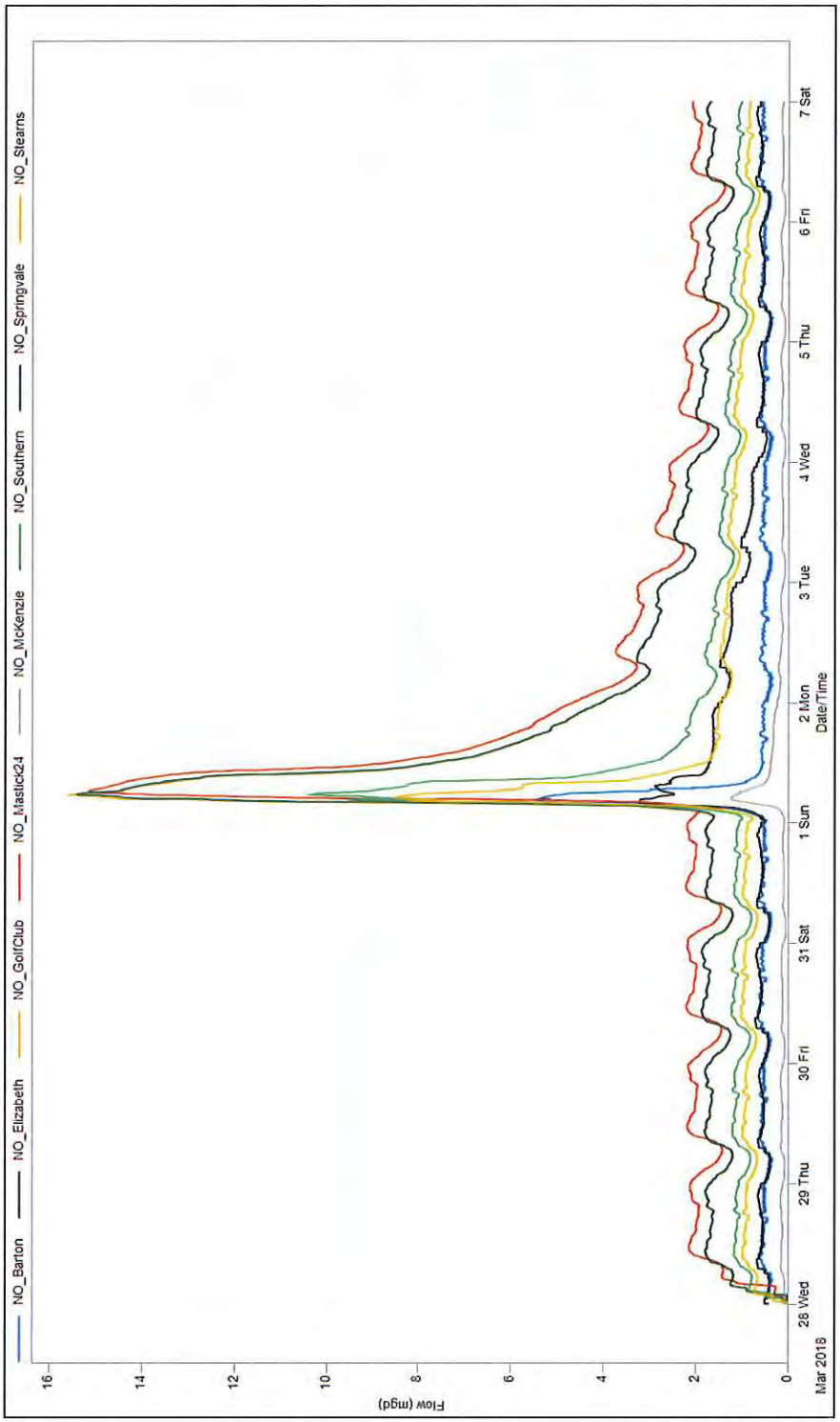
The upper end of the interceptor no longer overflows at the 90-degree bend on Barton Road in this alternative. However, the freeboard is less than 8 feet due to capacity of the sewers being similar to the upstream pump station capacity from Stafford Drive to Barton Road.

The peak HGL profile for the interceptor is provided in **Figure 4-5**. Since this alternative also appears to provide the best HGL/freeboard of the first 4 alternatives, the design storm flows at the flow meter locations are provided in **Graph 4-2**.

Figure 4-5
Alternative 5
Maximum Hydraulic Grade Line



Graph 4-2
Alternative 5
Maximum Flow



5.0 Alternative Costs

A construction cost for each alternative was developed based on 2019 dollars. Each alternative construction cost estimate was provided with a AACE class 5 contingency value of 40% for study level planning and a 25% multiplier for project costs. The alternative costs are summarized in **Table 5-1**.

Table 5-1
Alternative Cost Estimate

Item	Multiplier	Alternative				
		1	2	3	4	5
Construction Cost		$0.35 M	$6.42	$7.42	$7.64	$27.62
AACE Class 5 Contingency	40%	$0.50 M	$8.98	$10.38	$10.70	$38.67
Project Costs	25%	$0.62 M	$11.23	$12.98	$13.37	$48.34

The peak HGL profile for each alternative is provided in **Figure 5-1** for a comparison of the level of service obtained with each cost.

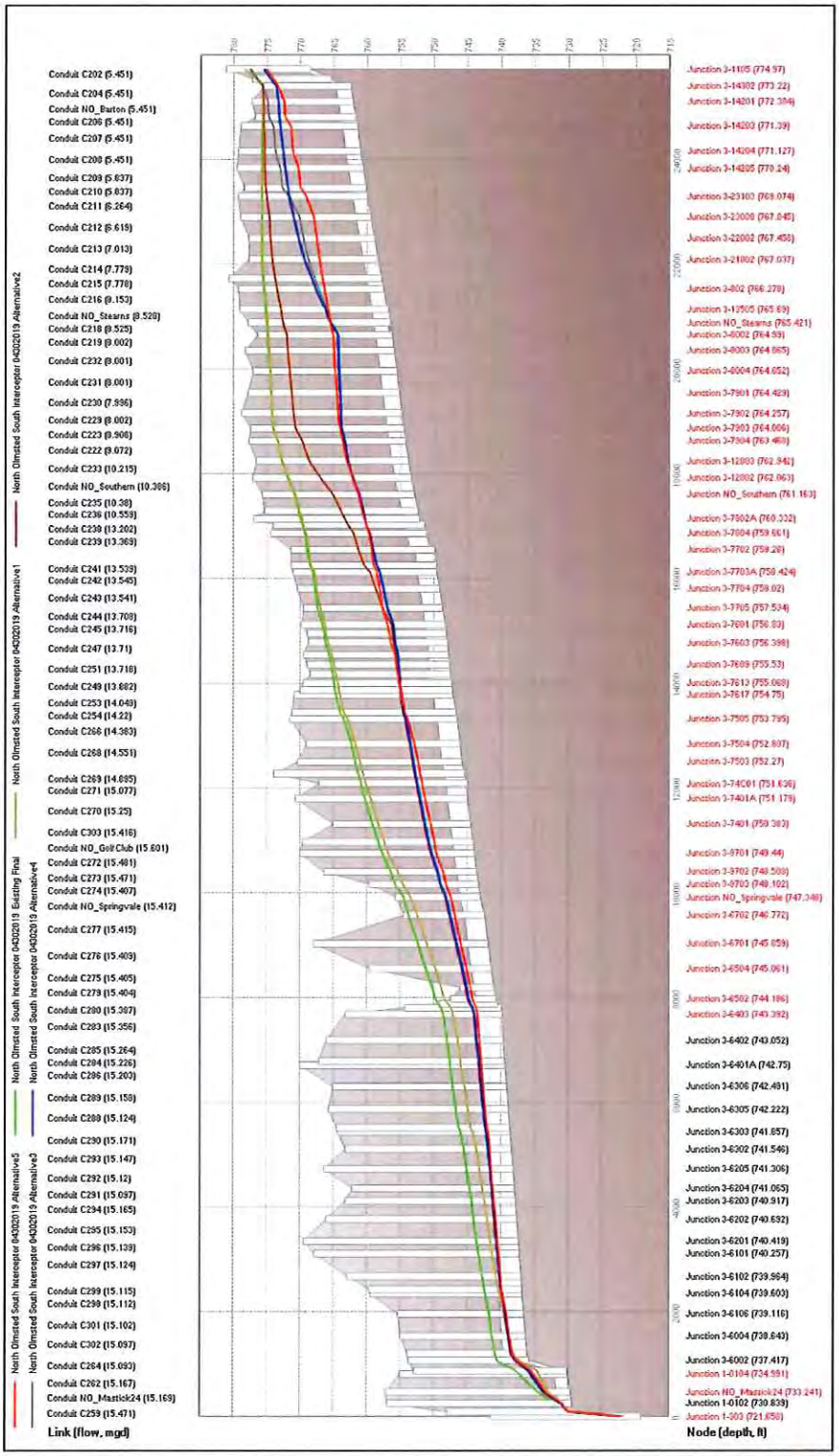


Figure 5-1
Alternative Comparison
Maximum Hydraulic Grade Line

6.0 Discussion

The calibration of the Southern Interceptor was a challenging task due to understanding the interactions of system bend losses, possible system blockage, and meter installation/error issues. A 2019 metering program was discussed to enhance future modeling efforts and target specific areas with additional flow response information. The 2019 metering program and any corresponding data analysis is not a part of this study, but as the data collection occurred during the development of the study report, CT did perform a cursory review of the initial data collected. Comments pertinent to this study are discussed in this section, however, all other comments are provided in the technical memo provided in **Appendix D**.

The 2019 program included moving the Elizabeth and McKenzie meters downstream to better installation sites. Meters were re-installed at Southern and Springvale sites for comparison to 2018 data. An additional meter at the Bodecker Hill SSO location to verify how close the system might come to an overflow. Another meter at the North Olmsted High School Soccer Field area to review possible interactions of the system with a proposed equalization basin in this area and a meter on the Mastick 42-inch sewer upstream of 2 bends to continue to review their backwater impacts on the system in relation to the downstream 24-inch sewer. The remainder of available meters were placed for additional flow characterization or wet weather tributary response. The locations of the 9 meters installed are shown on **Figure 2-2**.

Based upon the 2019 flow meter data collected since the end of March, it appears that the model calibration for the 2018 Elizabeth site is reasonable. The calibration for the 2018 McKenzie site may be low for peak events by approximately 0.5 mgd. Since the 2018 and 2019 meter data comparison for the Southern and Springvale sites appear similar, the McKenzie difference will mean that the Springvale calibration is conservative.

7.0 Conclusion

The study indicates that Alternatives 4 and 5 are two feasible alternatives for managing the capacity of the Southern Interceptor to the 10 year 6-hour design event. The two alternatives include multiple equalization basins or I/I removal with a 50% flow reduction. The equalization alternative is less expensive at $13.4 M, but has new continuing operational and maintenance costs and continuing wet weather volume treatment costs at the waste water treatment plant. Additionally, the system could be impacted by system age, which could create more I/I over time.

I/I removal has a much higher cost at $48.3 M and will involve working on private property. The advantage of this option is that there can actually be a return on investment due to decreased treatment costs as well as creating capacity for future development opportunities whether they be from within the City limits or from surrounding communities.

The City should review the alternatives and consider the policies required to achieve these alternatives to decide a chosen alternative or a possible combination of the two alternatives.

Depending on funding availability and schedule for the chosen alternative, the City may want to consider implementation of Alternative 1. Alternative 1 can have a localized impact of reducing the SSO and may be able to be implemented more quickly due to cost than the more regional impacts of Alternatives 4 and 5 which also reduce the SSO impact.

ADDITIONAL NOTE

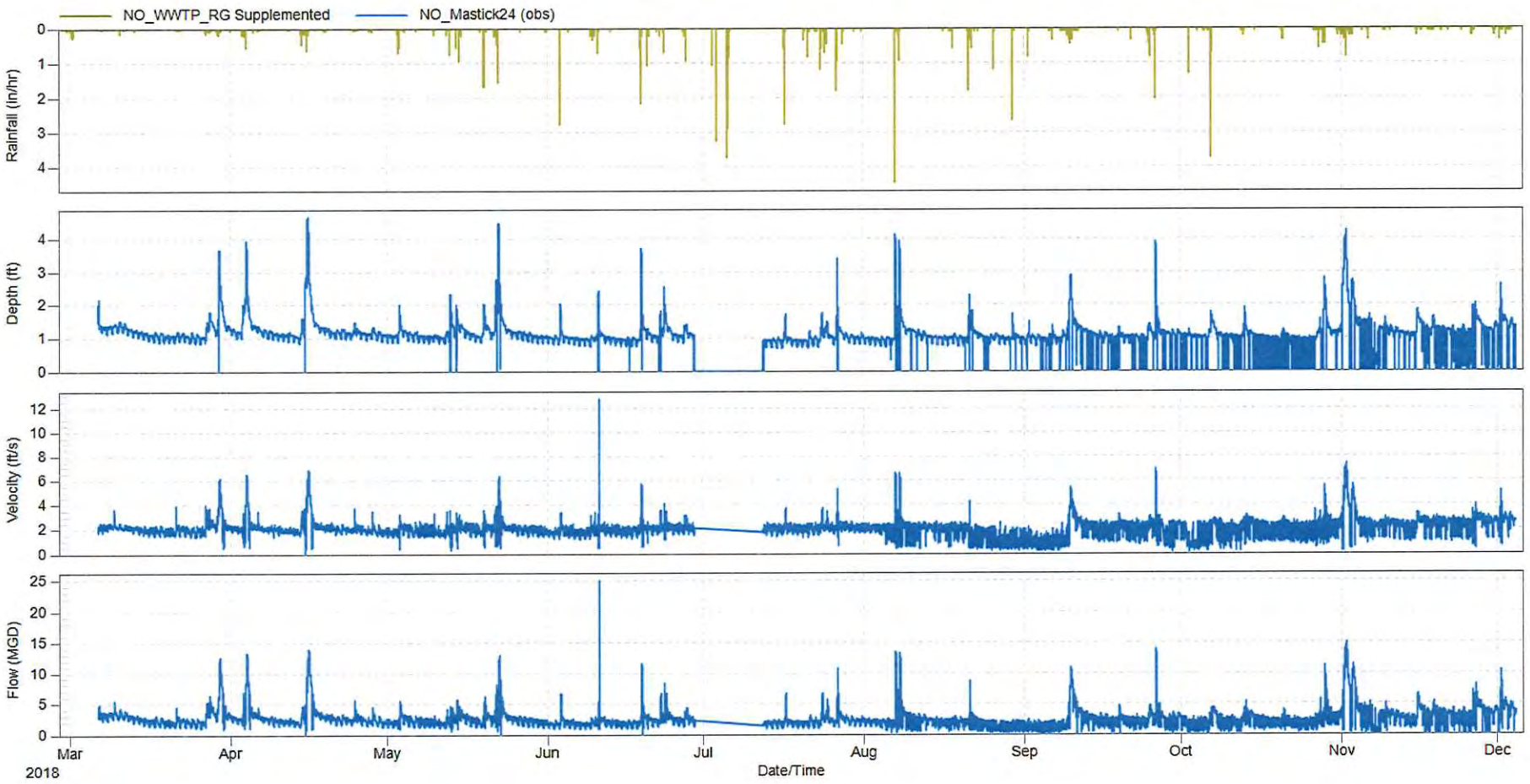
The City should continue to utilize meters to further investigate the metering issues and if the system blockage upstream of Bodecker Hill is a systemic issue or a single event. Televising this investigation area could also provide further insight into these issues.

Appendix A
Rainfall Data

City of North Olmstead Rainfall Event Summary					
Event	Date	Duration (h)	Maximum Rainfall (in/hr)	Mean Rainfall (in/hr)	Total Rainfall (in)
1	2/28/2018 11:45	2.25	0.08	0.04889	0.11
2	3/1/2018 6:30	11.5	0.24	0.1061	1.22
3	3/10/2018 13:15	0.75	0.28	0.1733	0.13
4	3/27/2018 2:45	7.5	0.16	0.05067	0.38
5	3/27/2018 16:00	11.25	0.12	0.03022	0.34
6	3/29/2018 6:00	22.5	0.2	0.04756	1.07
7	4/3/2018 2:15	23.25	0.36	0.05806	1.35
8	4/14/2018 13:15	4	0.28	0.0675	0.27
9	4/15/2018 2:00	27	0.4	0.07111	1.92
10	4/24/2018 13:15	20.75	0.08	0.01639	0.34
11	4/28/2018 1:15	7	0.16	0.03	0.21
12	5/3/2018 4:45	6.75	0.64	0.08	0.54
13	5/4/2018 14:15	0.25	0.4	0.4	0.1
14	5/10/2018 1:15	2	0.28	0.1	0.2
15	5/12/2018 13:15	2.25	0.24	0.07111	0.16
16	5/13/2018 3:30	4.75	0.68	0.1368	0.65
17	5/14/2018 9:15	3.5	0.44	0.1257	0.44
18	5/15/2018 8:25	0.75	0.24	0.1333	0.1
19	5/19/2018 13:40	7.75	2.76	0.07871	0.61
20	5/21/2018 18:55	6.92	0.36	0.1113	0.77
21	5/22/2018 8:20	4.75	3.96	0.2274	1.08
22	6/3/2018 12:00	0.5	2.04	0.76	0.38
23	6/10/2018 0:40	3.17	0.24	0.04737	0.15
24	6/10/2018 15:45	4.33	1.56	0.1477	0.64
25	6/18/2018 20:45	9.42	3.6	0.1965	1.85
26	6/22/2018 16:25	5.17	0.84	0.09484	0.49
27	6/23/2018 9:40	4.42	3.12	0.1585	0.7
28	6/27/2018 5:40	12.17	1.08	0.0411	0.5
29	7/5/2018 14:15	2.17	3.72	0.3923	0.85
30	7/5/2018 21:05	7.67	2.88	0.09	0.69
31	7/16/2018 17:50	5.58	2.76	0.1146	0.64
32	7/21/2018 7:00	6.08	0.84	0.06904	0.42
33	7/23/2018 14:05	7.92	1.2	0.07453	0.59
34	7/24/2018 19:15	2.5	0.48	0.076	0.19
35	7/26/2018 17:35	2.25	1.8	0.4667	1.05
36	7/31/2018 11:20	1.33	0.12	0.0825	0.11
37	8/6/2018 22:20	7.25	5.4	0.331	2.4
38	8/7/2018 16:05	4.33	2.28	0.2054	0.89
39	8/18/2018 5:05	0.33	1.2	0.72	0.24
40	8/21/2018 0:35	9.92	3	0.08067	0.8

City of North Olmstead Rainfall Event Summary					
Event	Date	Duration (h)	Maximum Rainfall (in/hr)	Mean Rainfall (in/hr)	Total Rainfall (in)
41	8/21/2018 15:15	4.67	0.96	0.08786	0.41
42	8/23/2018 0:45	0.75	0.36	0.16	0.12
43	8/25/2018 21:45	6.25	0.6	0.0192	0.12
44	8/29/2018 14:05	1.75	1.44	0.2514	0.44
45	9/1/2018 17:45	0.58	0.6	0.3429	0.2
46	9/5/2018 17:25	0.33	1.08	0.75	0.25
47	9/6/2018 4:30	4.17	0.36	0.0648	0.27
48	9/7/2018 23:50	11.33	0.12	0.02206	0.25
49	9/8/2018 17:30	39.83	0.24	0.03816	1.52
50	9/10/2018 18:00	16.5	0.24	0.01455	0.24
51	9/19/2018 2:15	0.5	2.76	1.12	0.56
52	9/21/2018 15:00	0.25	1.56	0.76	0.19
53	9/24/2018 16:45	11.42	0.48	0.05343	0.61
54	9/26/2018 1:35	5.58	2.04	0.2651	1.48
55	10/2/2018 14:05	0.25	1.32	0.96	0.24
56	10/6/2018 7:20	21.17	3.72	0.04299	0.91
57	10/12/2018 19:50	13.5	0.24	0.05111	0.69
58	10/20/2018 12:55	8.25	0.36	0.03879	0.32
59	10/26/2018 19:00	32.83	0.6	0.0198	0.65
60	10/28/2018 13:10	9.83	0.48	0.1058	1.04
61	10/31/2018 4:45	4.75	0.24	0.02105	0.1
62	10/31/2018 14:05	4.92	0.12	0.02441	0.12
63	10/31/2018 23:50	23.17	0.84	0.1079	2.5
64	11/2/2018 4:05	5.17	0.12	0.01935	0.1
65	11/2/2018 16:00	15.5	0.24	0.06387	0.99
66	11/5/2018 0:15	2.5	0.12	0.052	0.13
67	11/6/2018 1:20	5.67	0.36	0.02647	0.15
68	11/9/2018 6:35	3.67	0.12	0.06	0.22
69	11/15/2018 10:40	7.33	0.24	0.07364	0.54
70	11/16/2018 8:00	8.42	0.12	0.02614	0.22
71	11/18/2018 11:15	9.25	0.12	0.02703	0.25
72	11/20/2018 6:00	6.5	0.12	0.01692	0.11
73	11/24/2018 4:30	4.33	0.12	0.03	0.13
74	11/26/2018 3:30	14.42	0.24	0.04578	0.66
75	11/30/2018 13:30	4.5	0.24	0.06444	0.29
76	12/1/2018 13:00	4.58	0.36	0.08727	0.4

Appendix B
Flow Meter Data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2
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4
Depth (ft)
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2
3
4
Velocity (ft/s)
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2
4
6
8
10
12
Flow (MGD)
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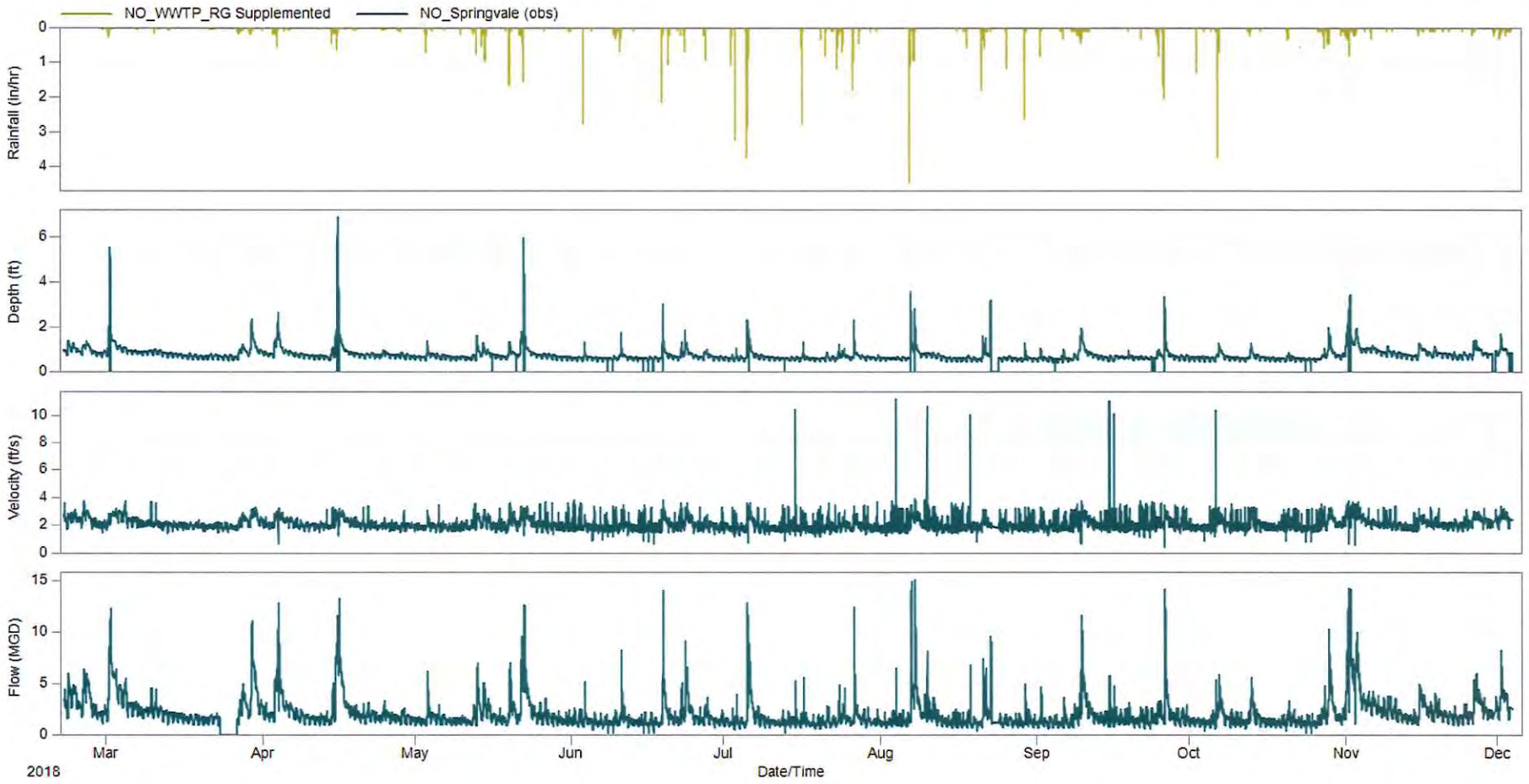
5
10
15
20
25
Mar
2018
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Date/Time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4
Depth (ft)
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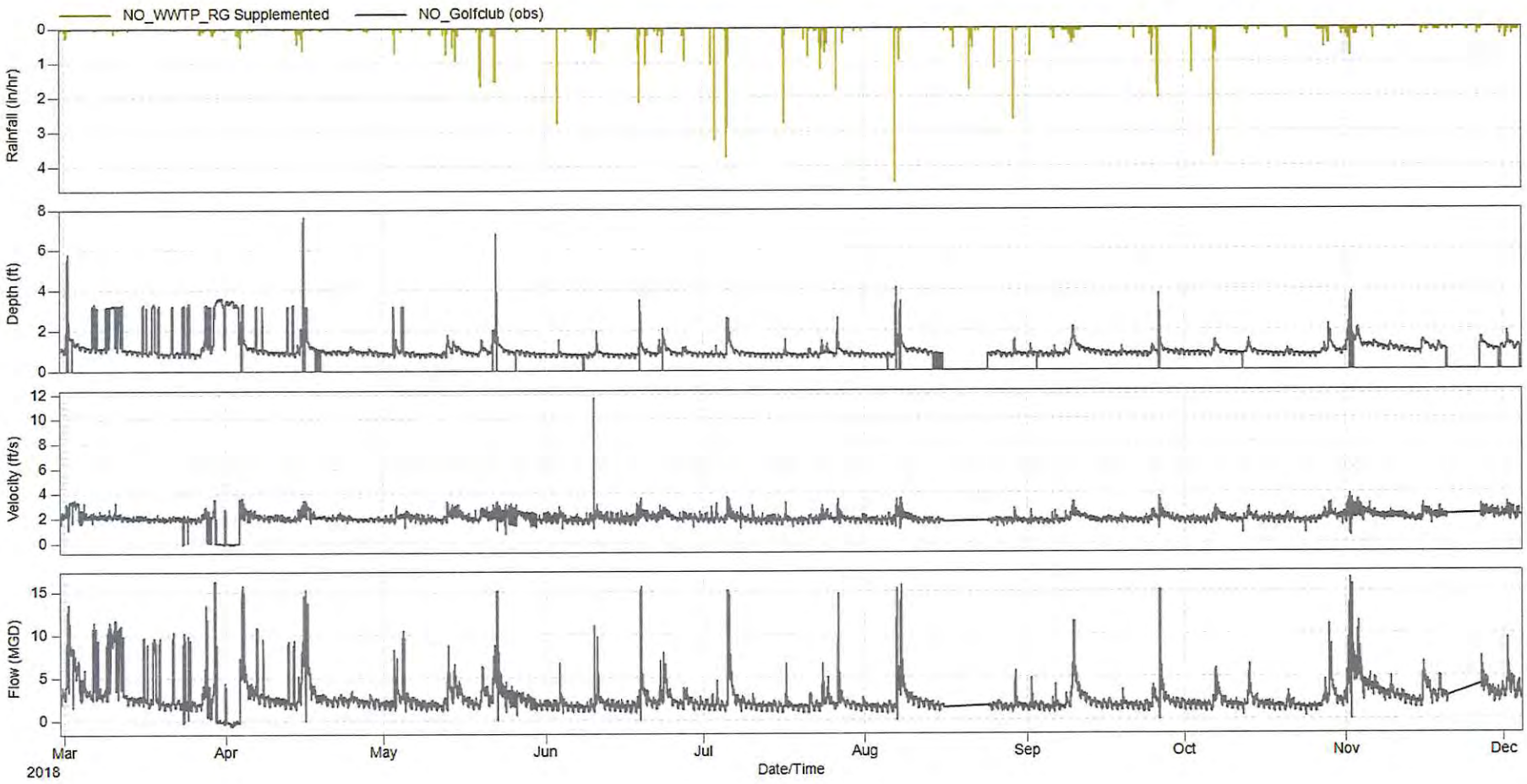
4
6
Velocity (ft/s)
0
2
4
6
8
10
Flow (MGD)
0
5
10
15
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
2018
Date/Time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Rainfall (in/hr)
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2
3
4
Depth (ft)
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2
4
6
8
Velocity (ft/s)
0
2
4
6
8
10
12
Flow (MGD)
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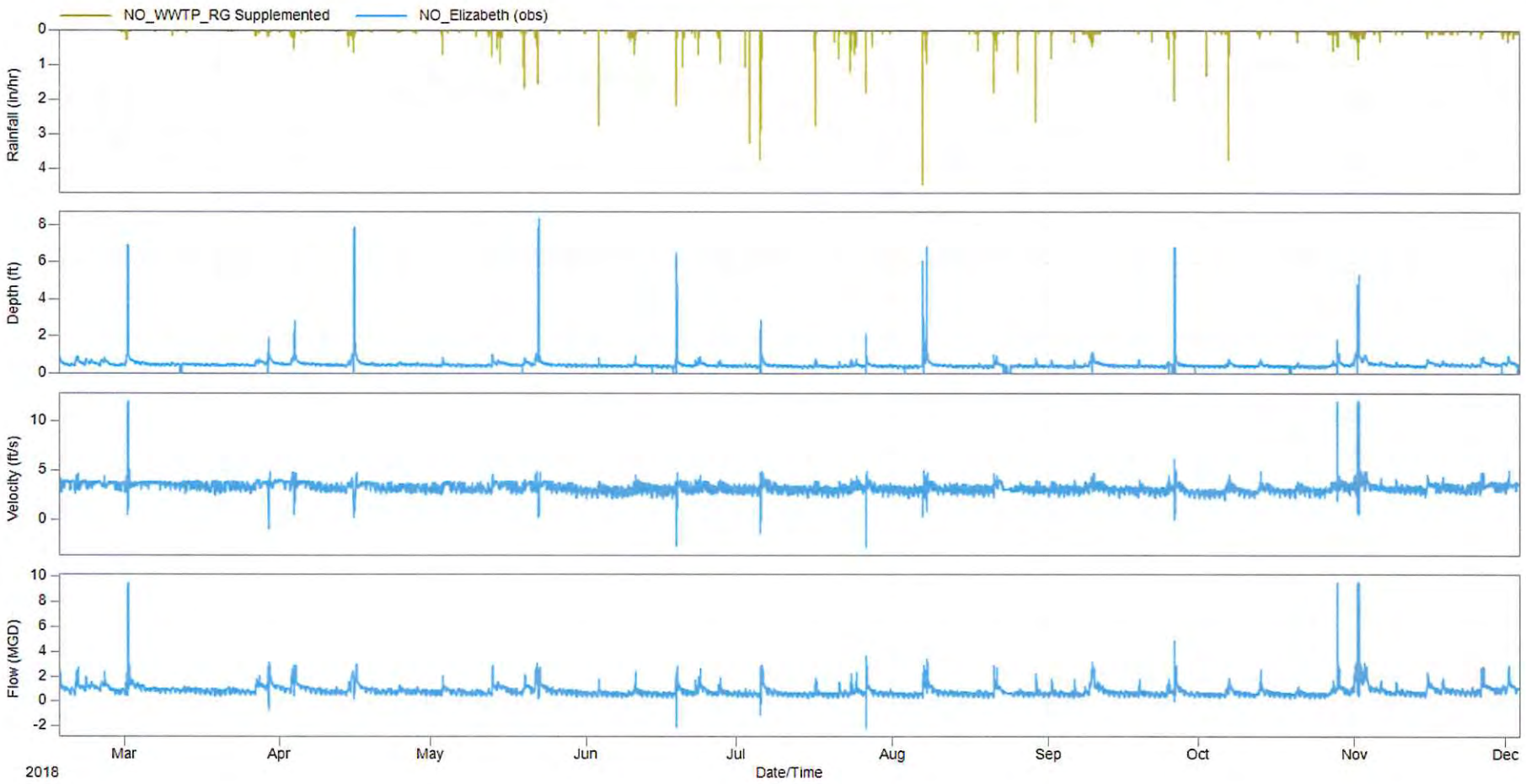
5
10
15
Mar
2018
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Date/Time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NO_Elizabeth (obs)
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6
8
Velocity (ft/s)
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5
10
Flow (MGD)
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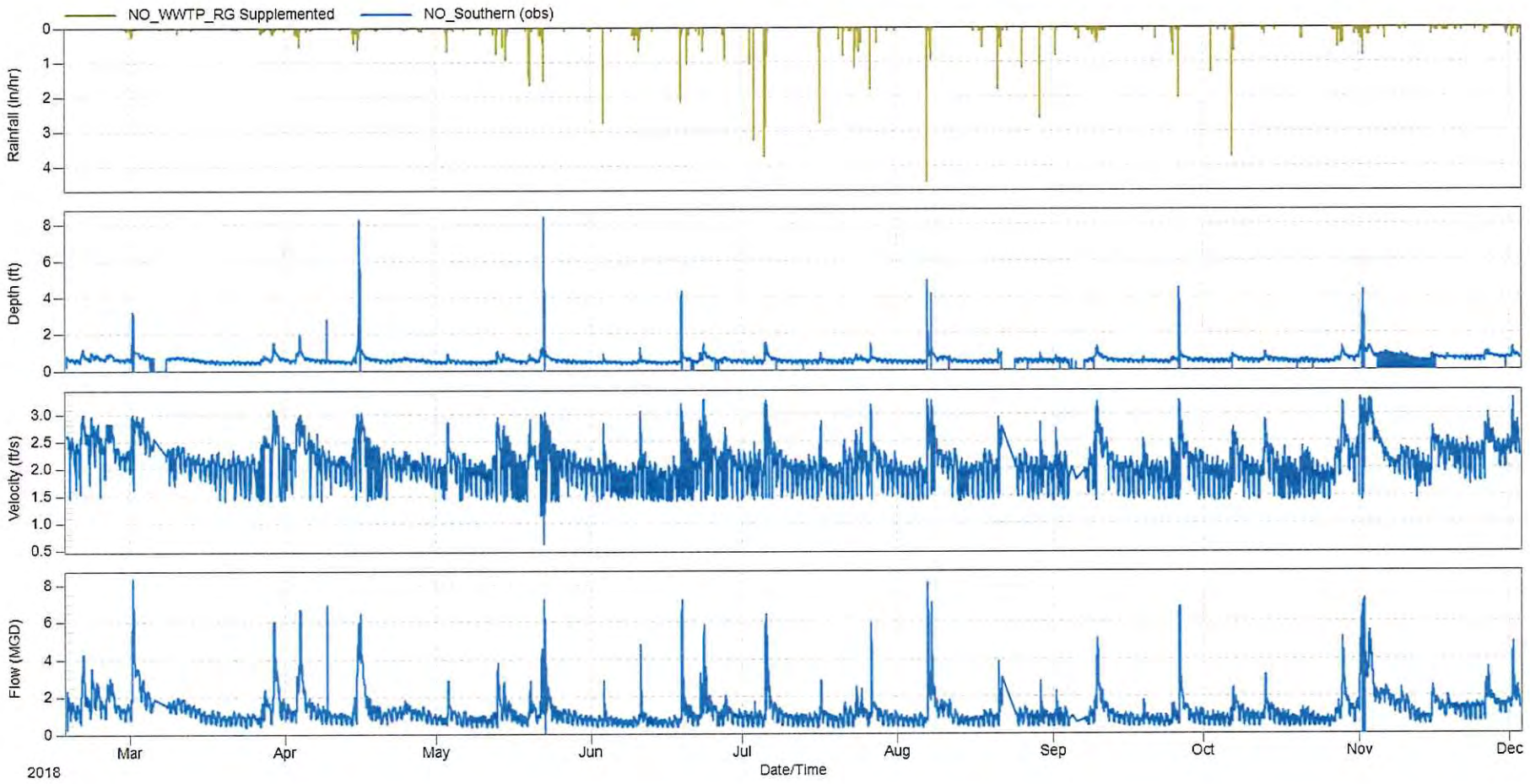
0
2
4
6
8
10
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
2018
Date/Time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Depth (ft)
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8
Velocity (ft/s)
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1.0
1.5
2.0
2.5
3.0
Flow (MGD)
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2
4
6
8
2018
Mar
Apr
May
Jun
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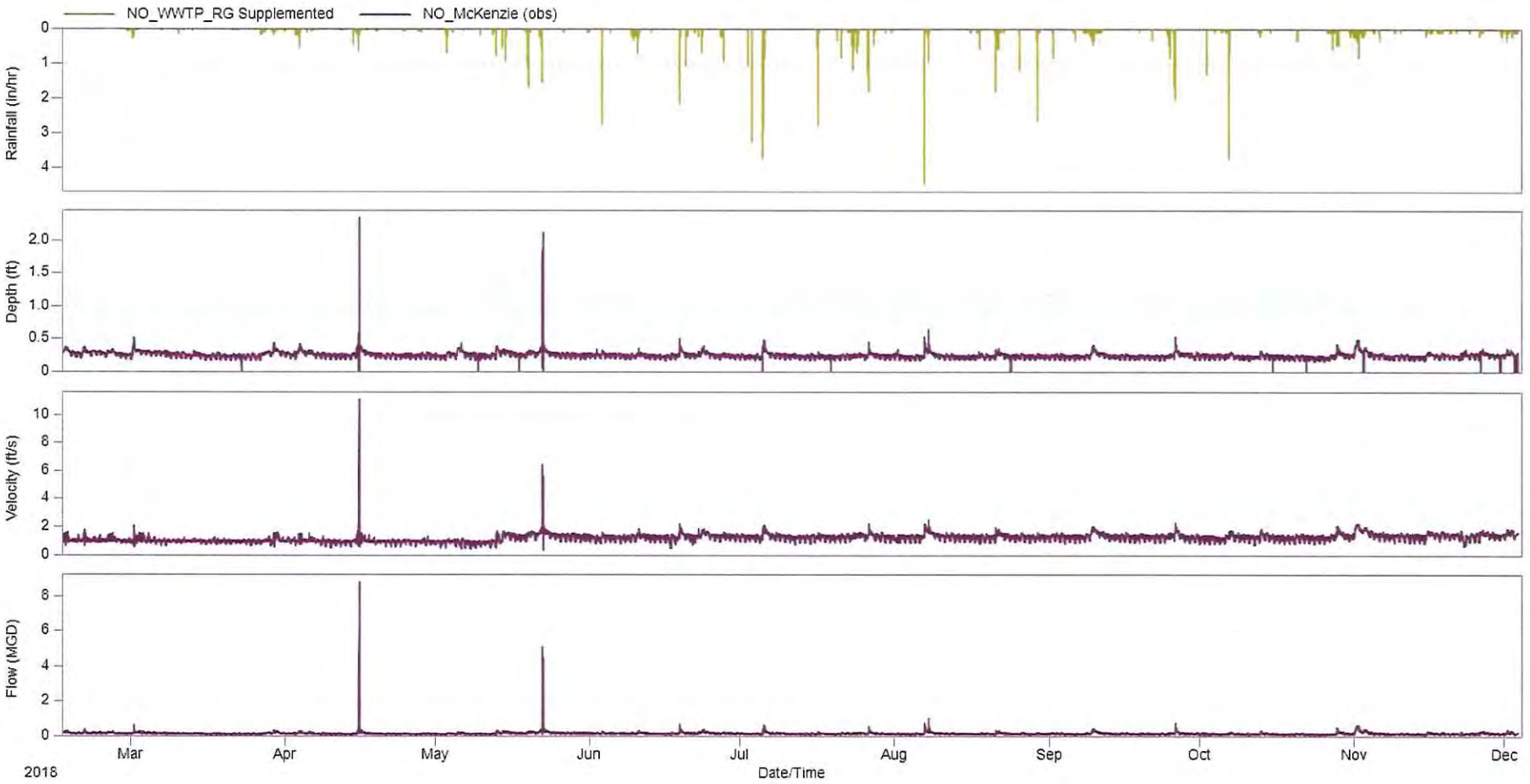
Aug
Sep
Oct
Nov
Dec
Date/Time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1.5
2.0
Velocity (ft/s)
0
2
4
6
8
10
Flow (MGD)
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4
6
8
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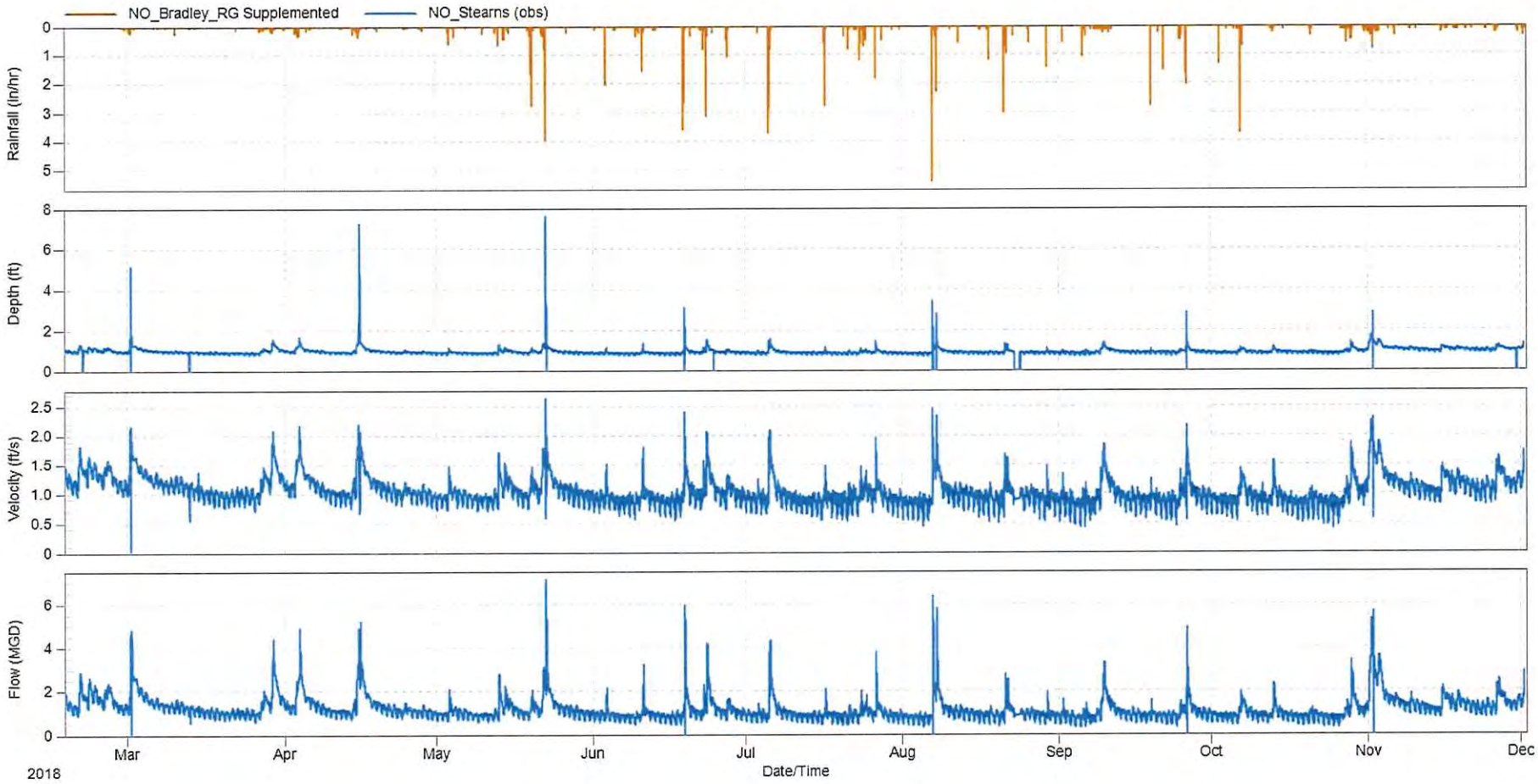
Nov
Dec
Date/Time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Flow (MGD)
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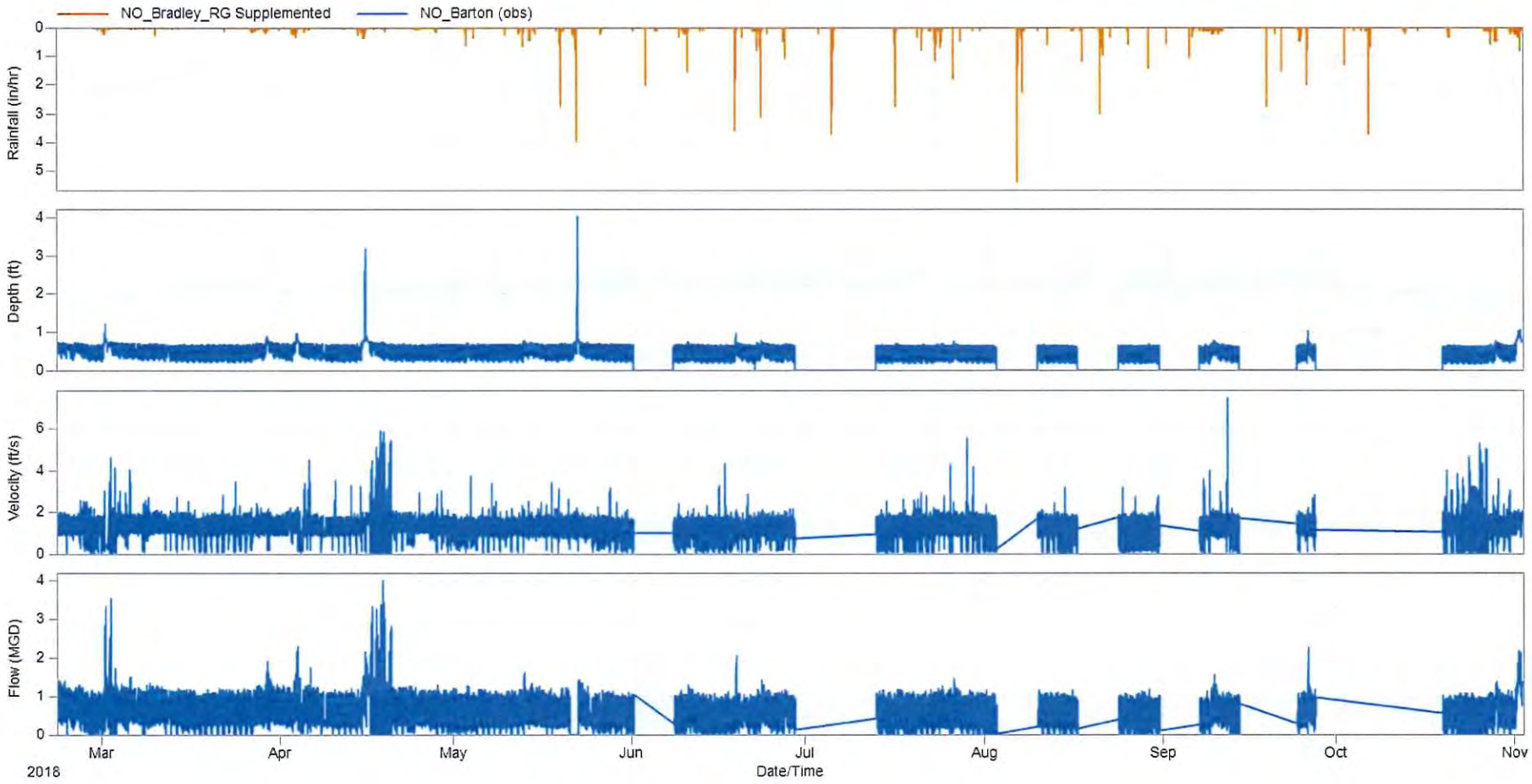
Aug
Sep
Oct
Nov
Dec
Date/Time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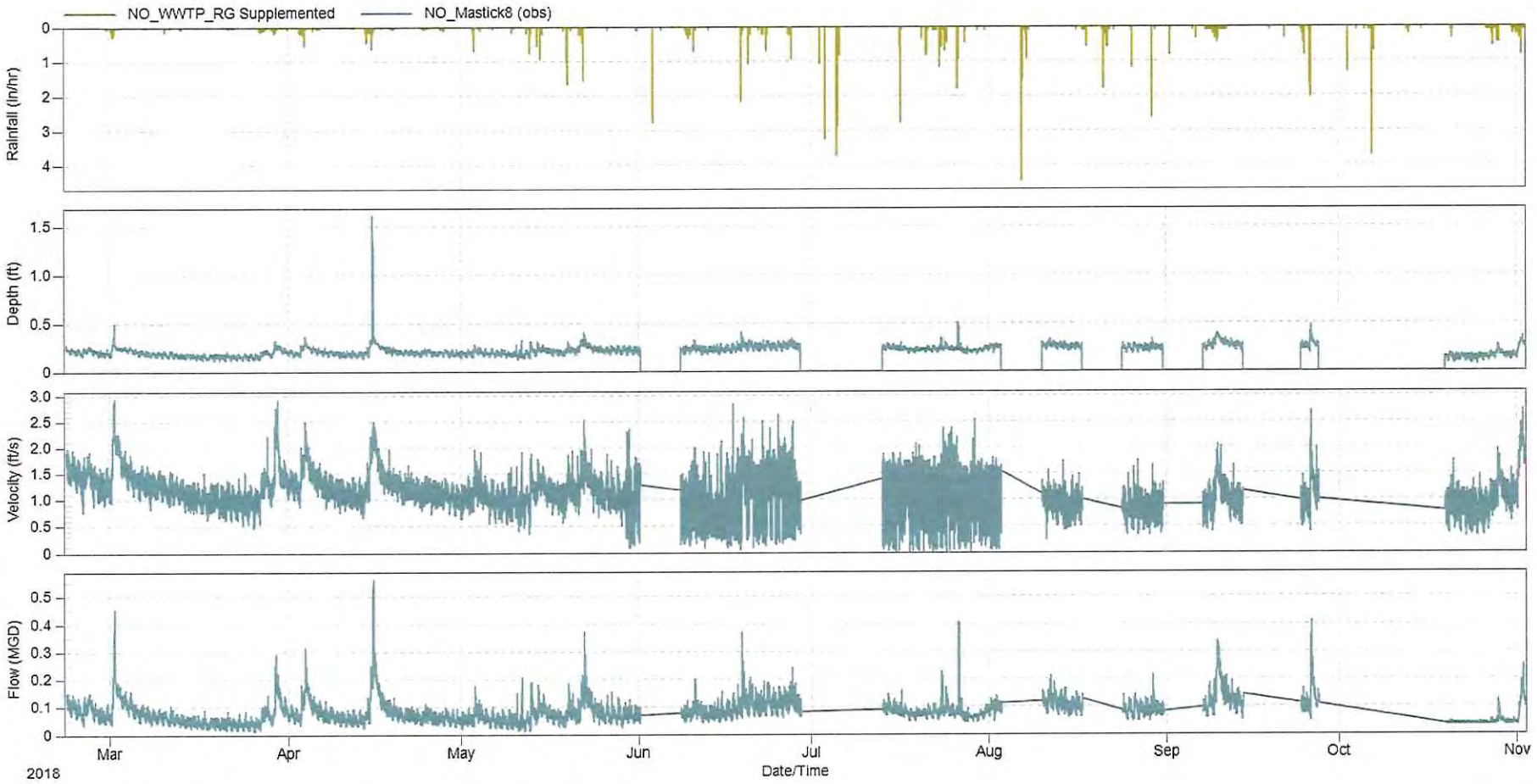
Oct
Nov
2018
Date/Time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NO_Mastick8 (obs)
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2
3
4
Depth (ft)
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0.5
1.0
1.5
Velocity (ft/s)
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0.5
1.0
1.5
2.0
2.5
3.0
Flow (MGD)
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0.4
0.5
Mar
Apr
May
Jun
Jul
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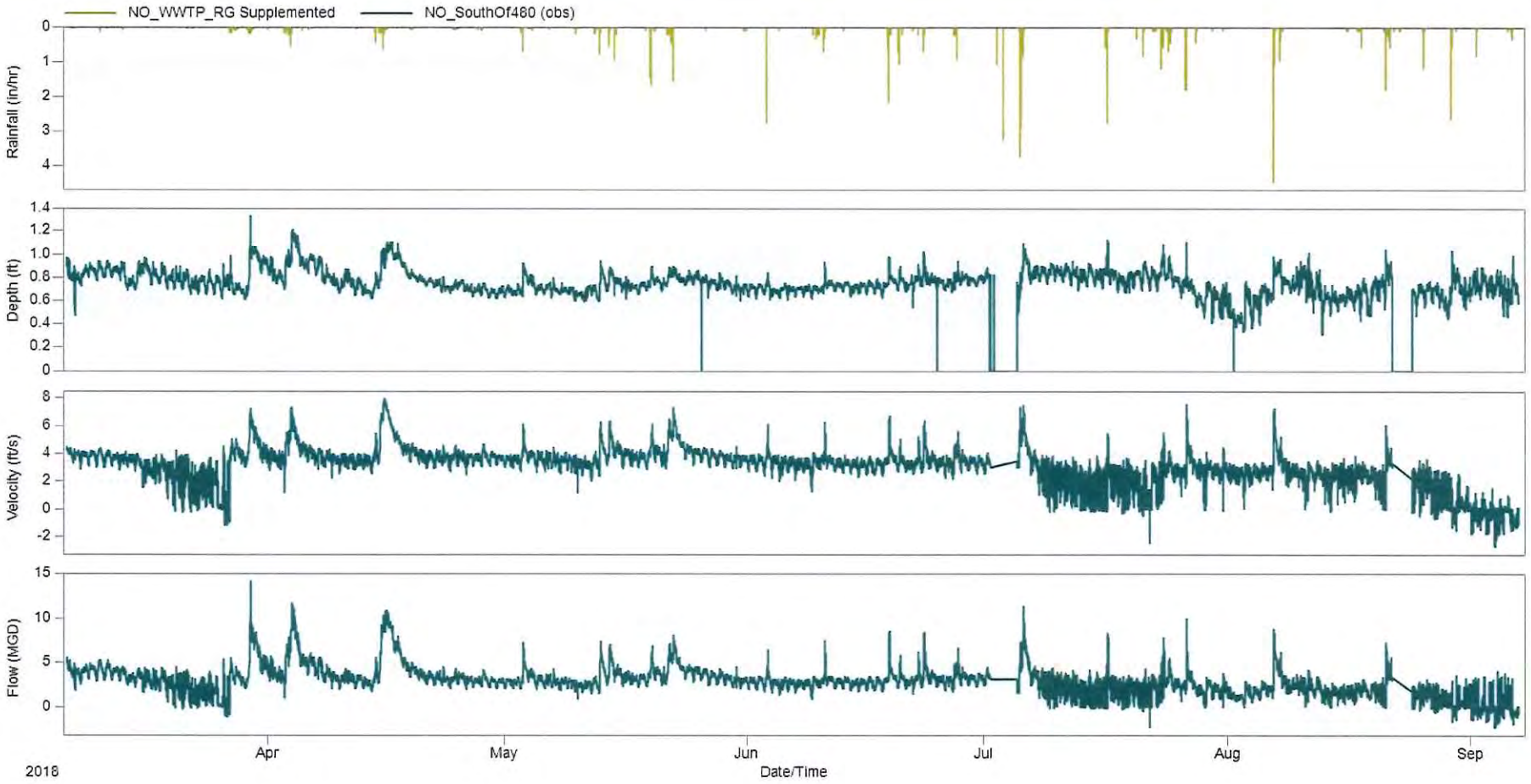
Nov
2018
Date/Time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3
4
Depth (ft)
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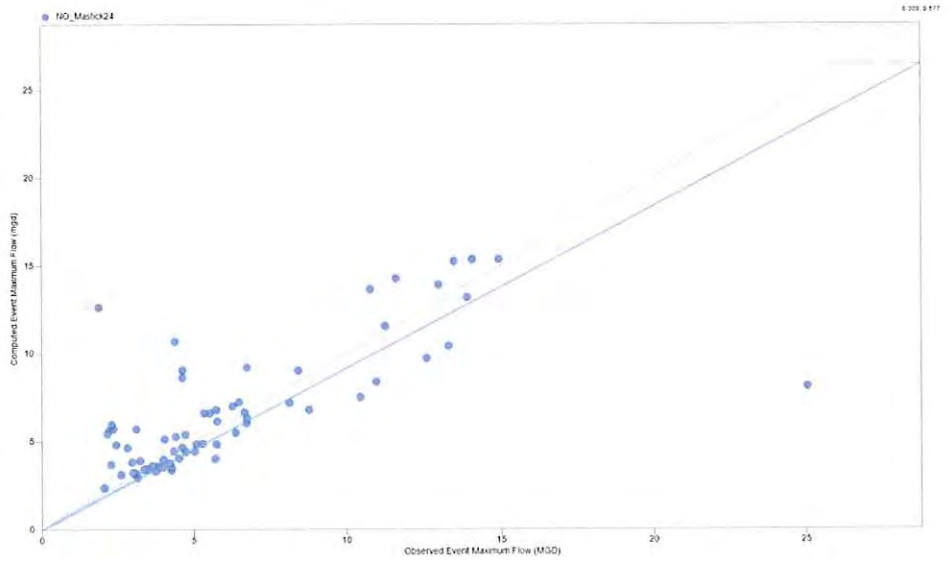
1.2
1.0
0.8
0.6
0.4
0.2
0
Velocity (ft/s)
8
6
4
2
0
-2
Flow (MGD)
15
10
5
0
2018
Apr
May
Jun
Jul
Aug
Sep
Date/Time

CITY OF NORTH OLMSTED
SOUTH INTERCEPTOR STUDY

Appendix C
Calibration Data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Observed Event Maximum Flow (MGD)
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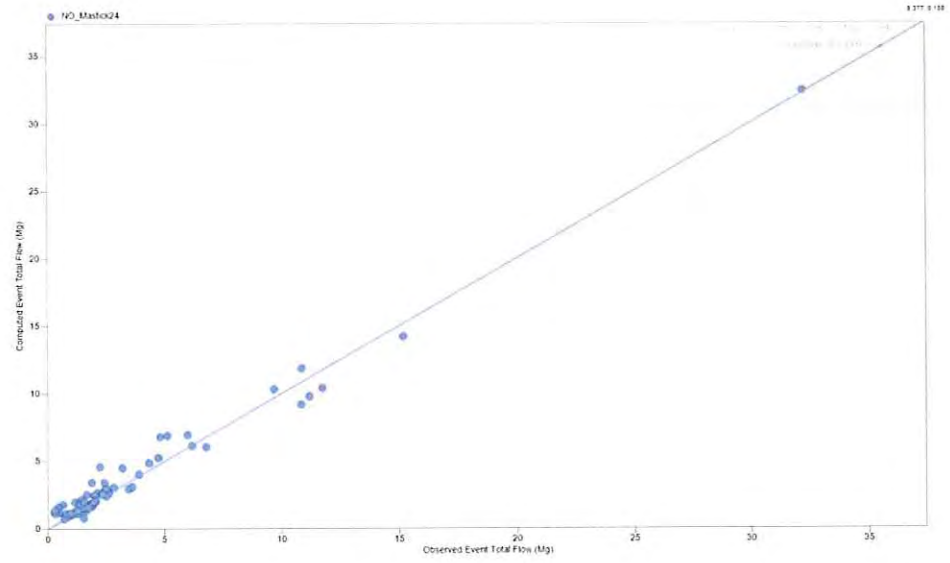
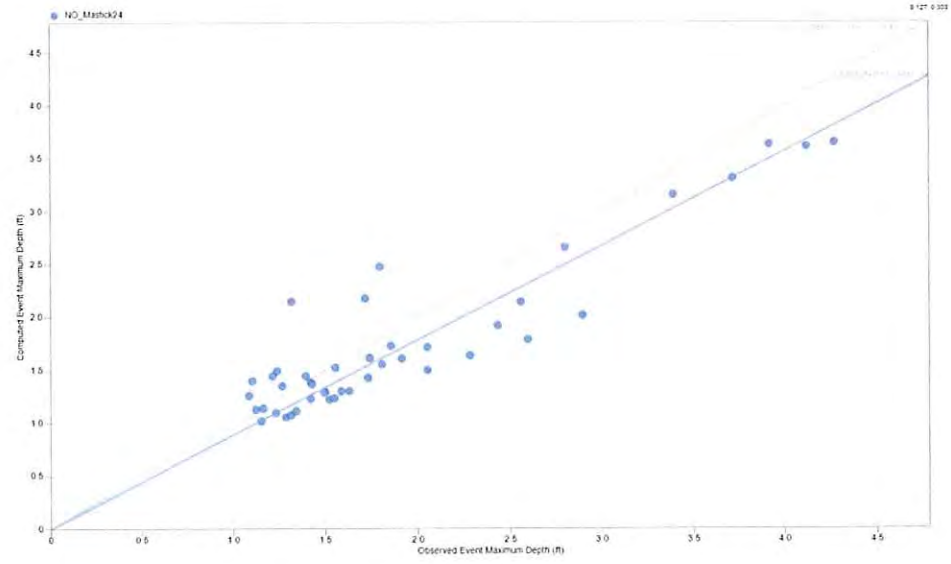
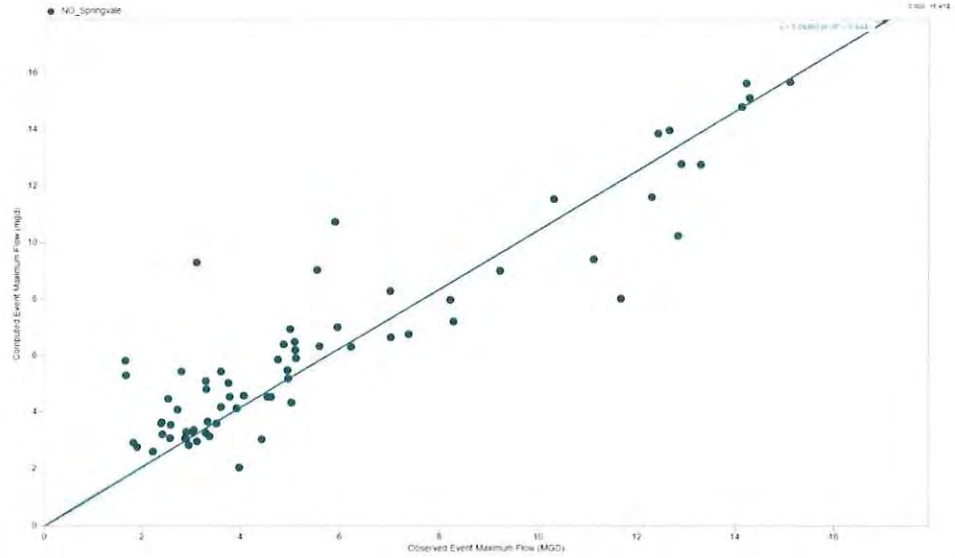
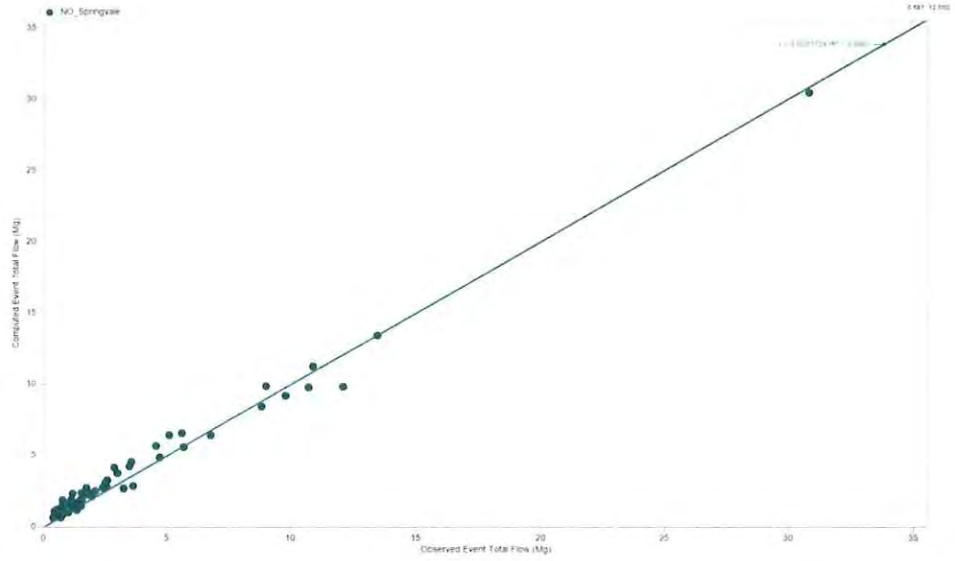
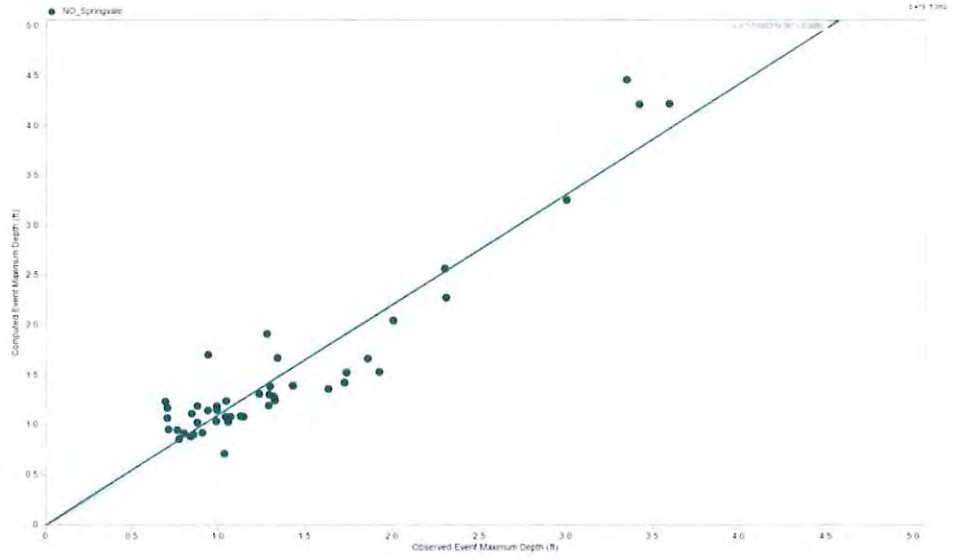
10
15
20
25

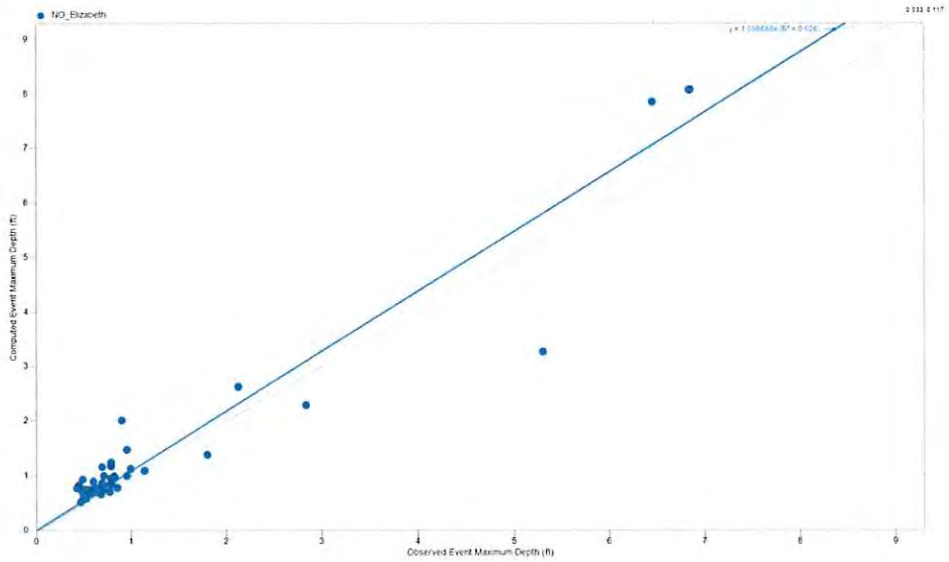

NO_Mashck24
Computed Event Total Flow (Mg)
Observed Event Total Flow (Mg)
0
5
10
15
20
25
30
35

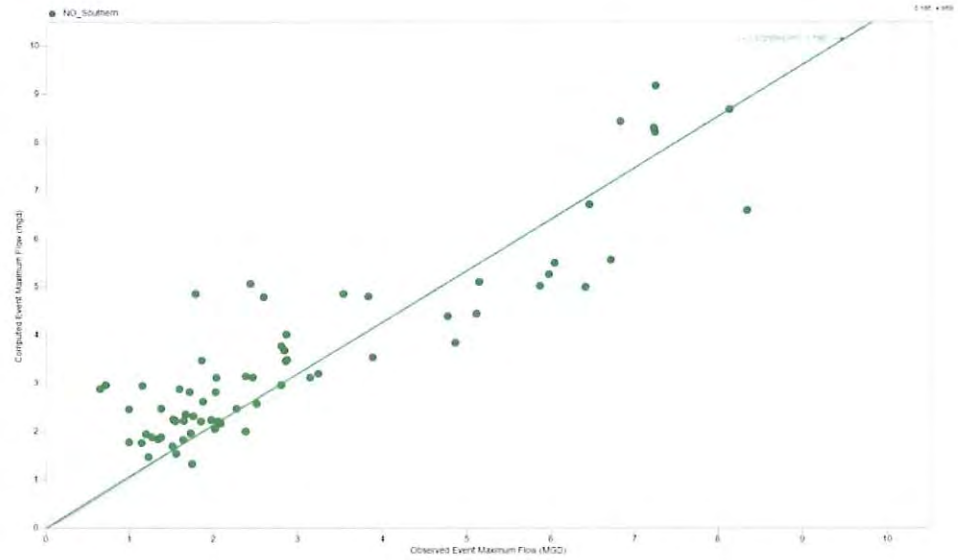

NO_Mashck24
Computed Event Maximum Depth (ft)
Observed Event Maximum Depth (ft)
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5

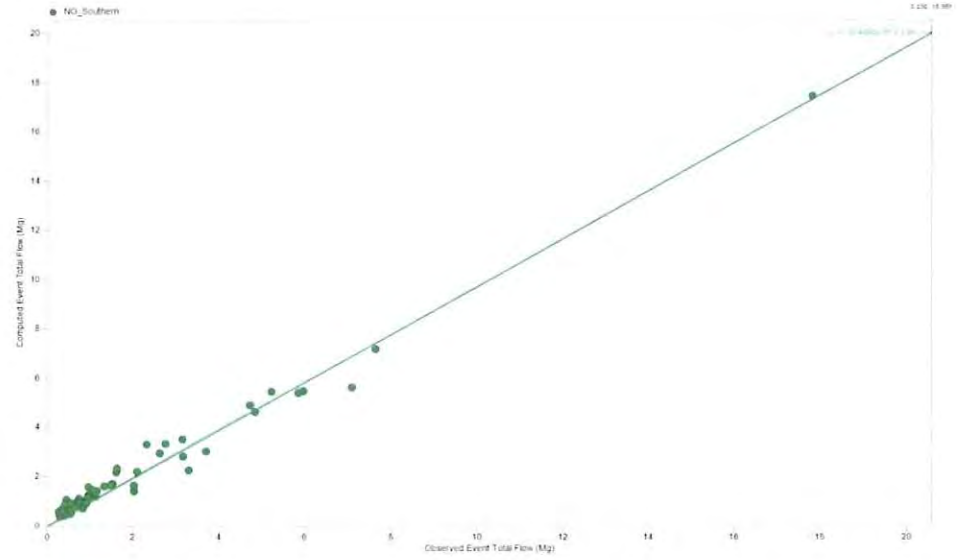

NO_Springvale
Computed Event Maximum Flow (mgd)
Observed Event Maximum Flow (MGD)
0
2
4
6
8
10
12
14
16

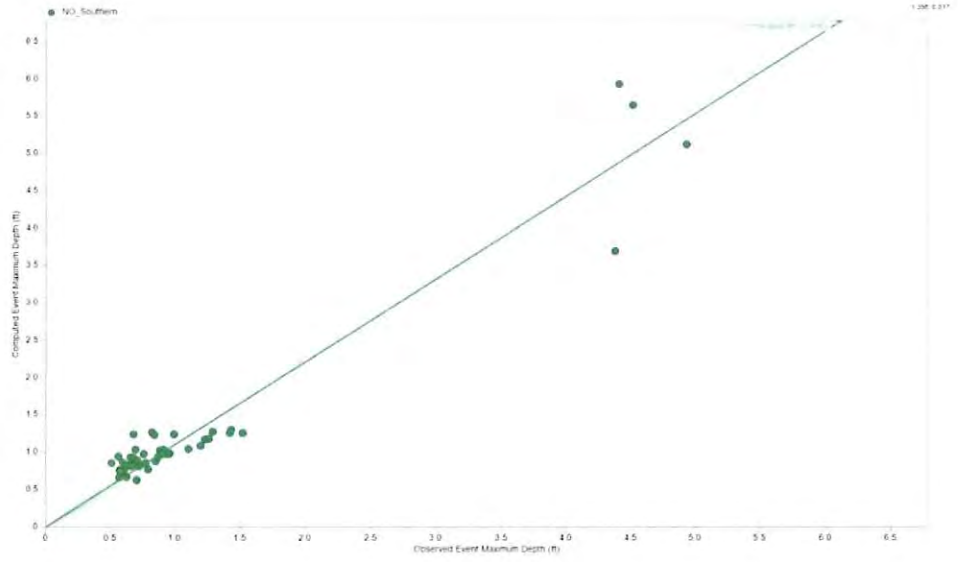

NO_Springvale
Computed Event Total Flow (Mg)
Observed Event Total Flow (Mg)
0
5
10
15
20
25
30
35

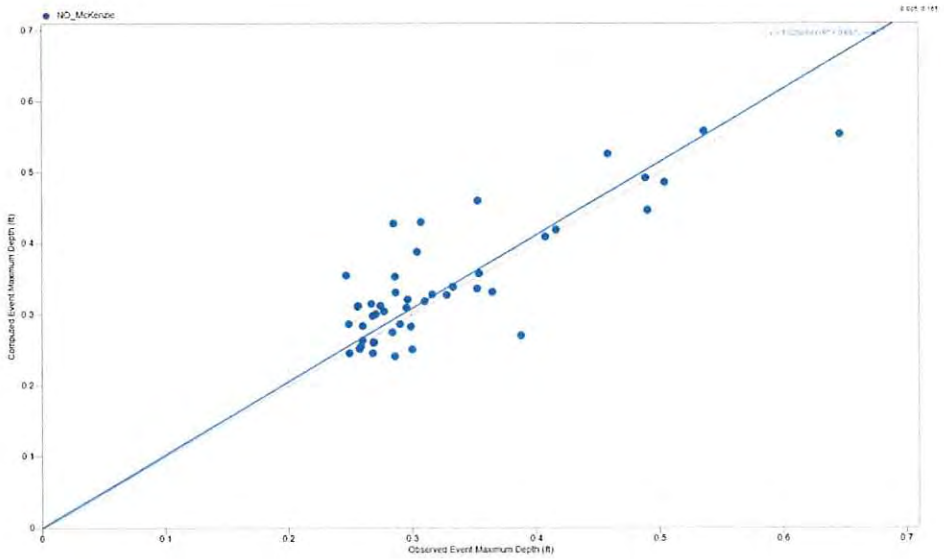

NO_Springvale
Computed Event Maximum Depth (ft)
Observed Event Maximum Depth (ft)
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0

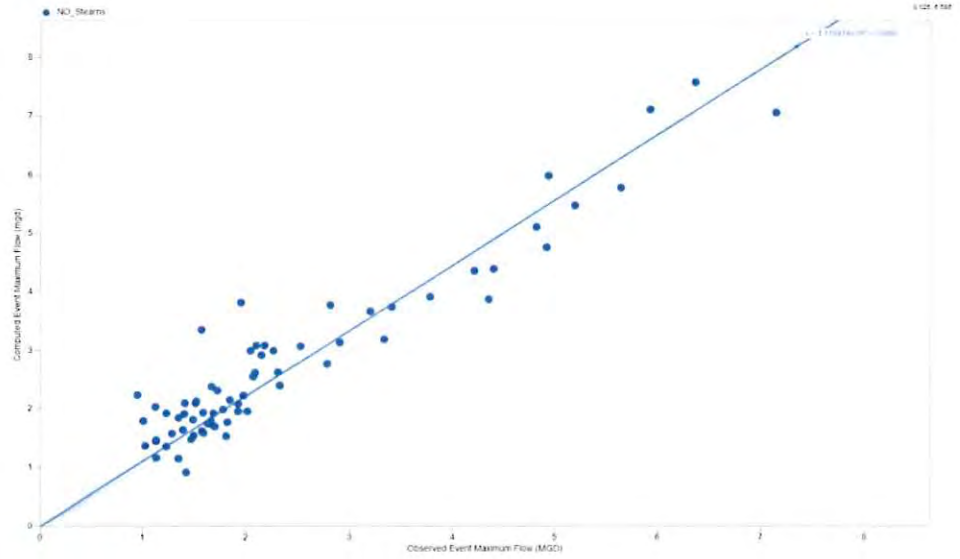

NO_Elizabeth
Computed Event Maximum Depth (ft)
Observed Event Maximum Depth (ft)
0
1
2
3
4
5
6
7
8
9

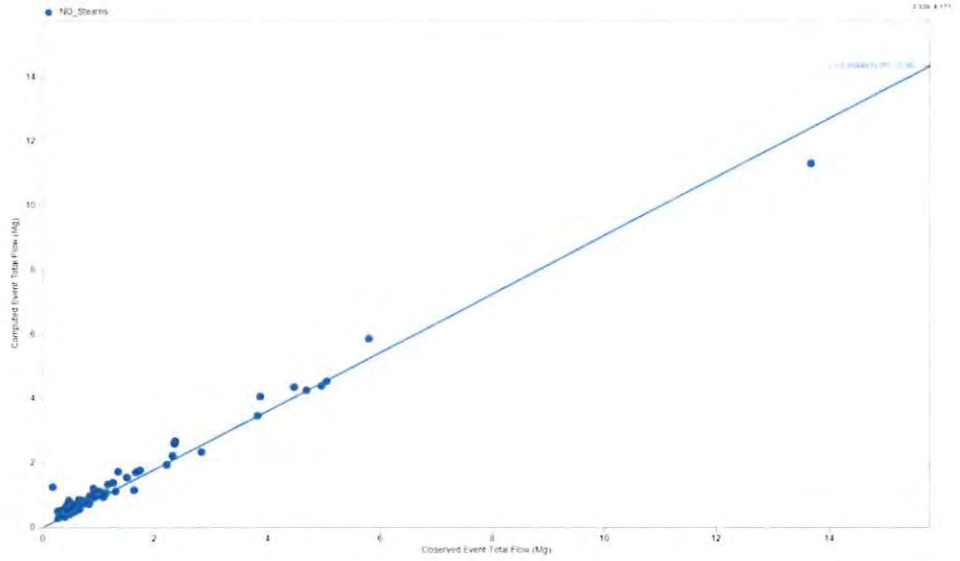

NO_Southern
Computed Event Maximum Flow (mgd)
Observed Event Maximum Flow (MGD)

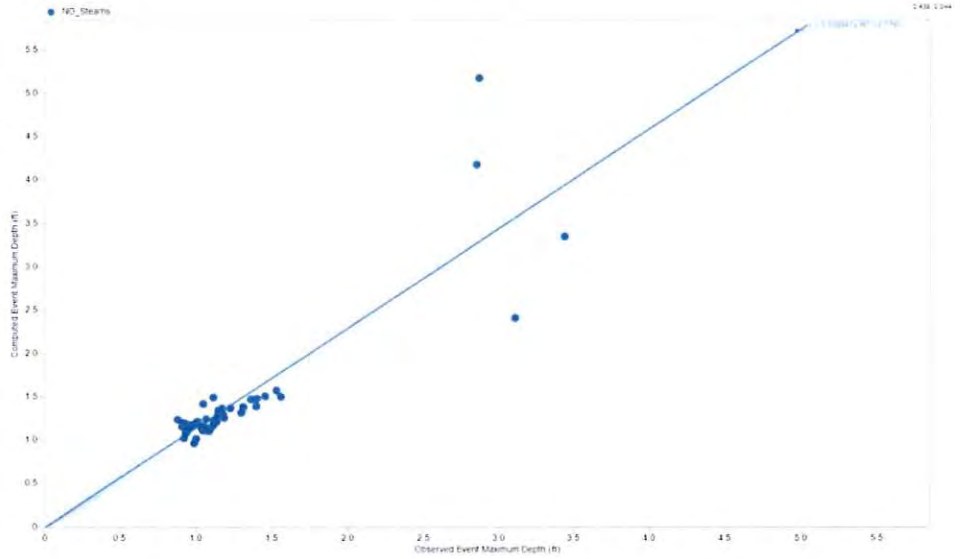

NO_Southern
Computed Event Total Flow (Mg)
Observed Event Total Flow (Mg)

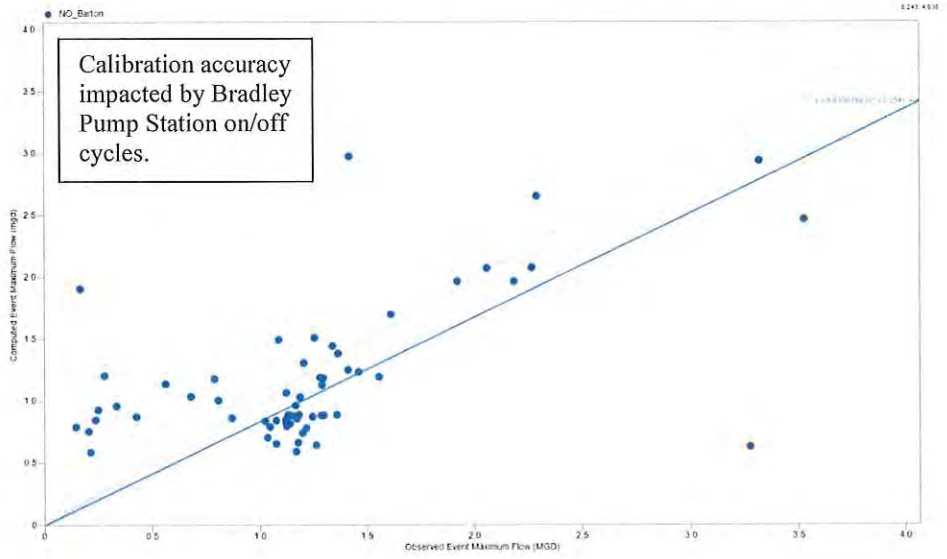

NO_Southern
Computed Event Maximum Depth (ft)
Observed Event Maximum Depth (ft)

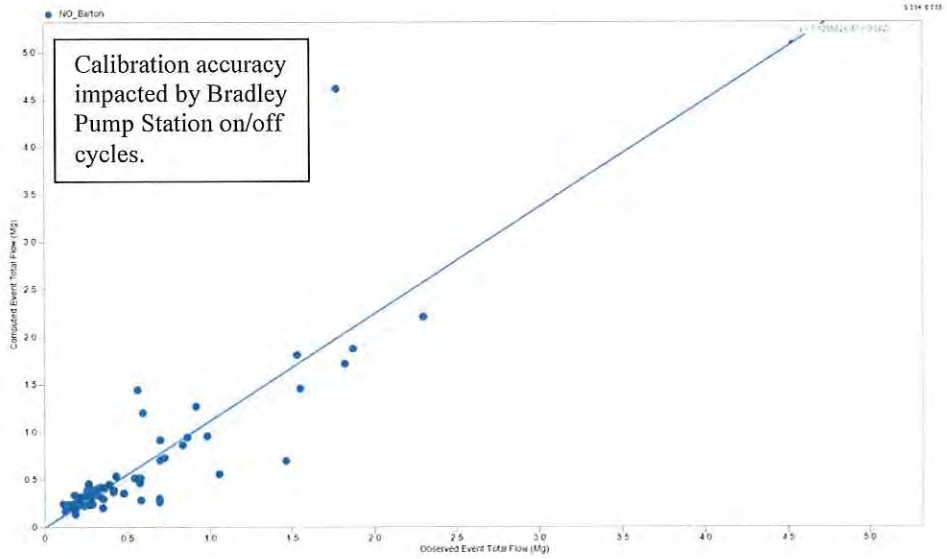

NO_McKenzie
Computed Event Maximum Depth (ft)
Observed Event Maximum Depth (ft)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7

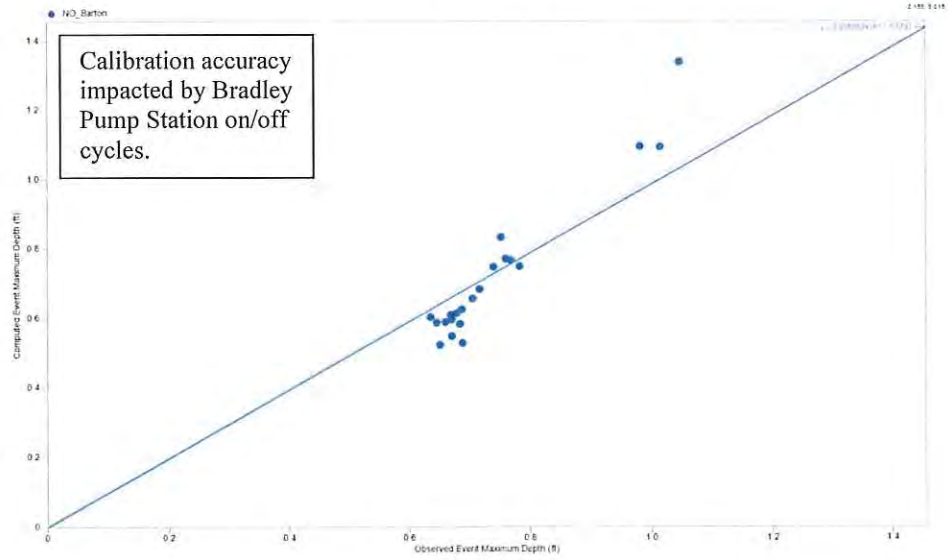

NO_Storms
Computed Event Maximum Flow (mgd)
Observed Event Maximum Flow (MGD)

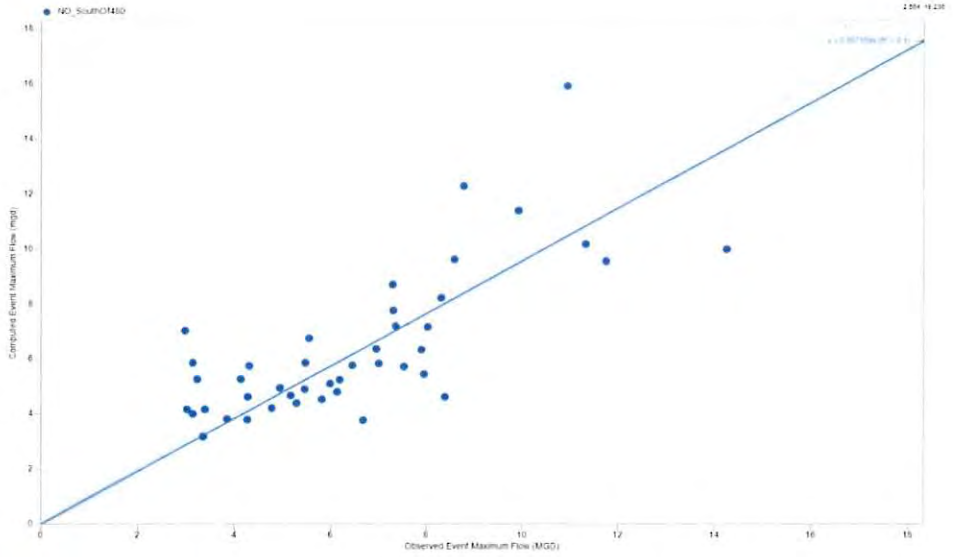

NO_Storms
Computed Event Total Flow (Mg)
Observed Event Total Flow (Mg)

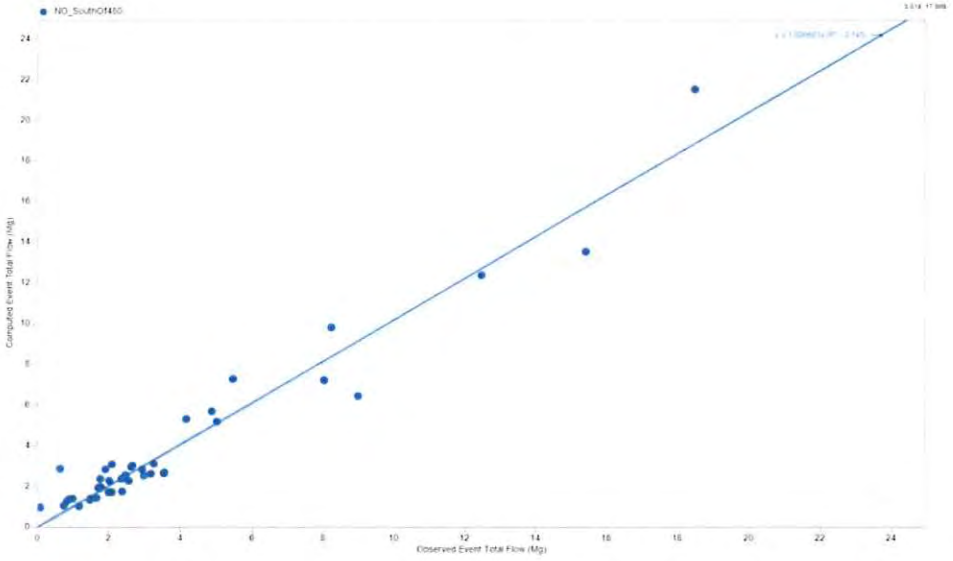

NO_Storms
Computed Event Maximum Depth (ft)
Observed Event Maximum Depth (ft)

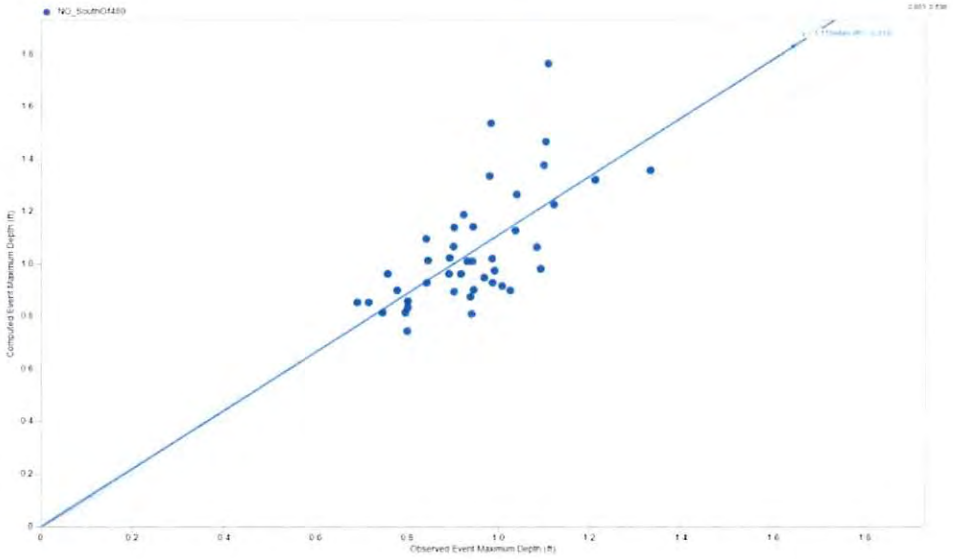

NO_Barton
Calibration accuracy impacted by Bradley Pump Station on/off cycles.
Computed Event Maximum Flow (mgd)
Observed Event Maximum Flow (MGD)


NO_Barton
Calibration accuracy impacted by Bradley Pump Station on/off cycles.
Computed Event Total Flow (Mg)
Observed Event Total Flow (Mg)


NO_Barton
Calibration accuracy impacted by Bradley Pump Station on/off cycles.
Computed Event Maximum Depth (ft)
Observed Event Maximum Depth (ft)


NO_SouthOf480
Computed Event Maximum Flow (mgd)
Observed Event Maximum Flow (MGD)


NO_SouthOf480
Computed Event Total Flow (Mg)
Observed Event Total Flow (Mg)


NO_SouthOf480
Computed Event Maximum Depth (ft)
Observed Event Maximum Depth (ft)

CITY OF NORTH OLMSTED
SOUTH INTERCEPTOR STUDY

Appendix D

2019 Flow Meter Data Technical Memo

Memorandum

To: Brian Blum, WWTP Superintendent

From: Chris Rybak

Subject: 2019 Flow Meter Data Review

Date: June 21, 2019

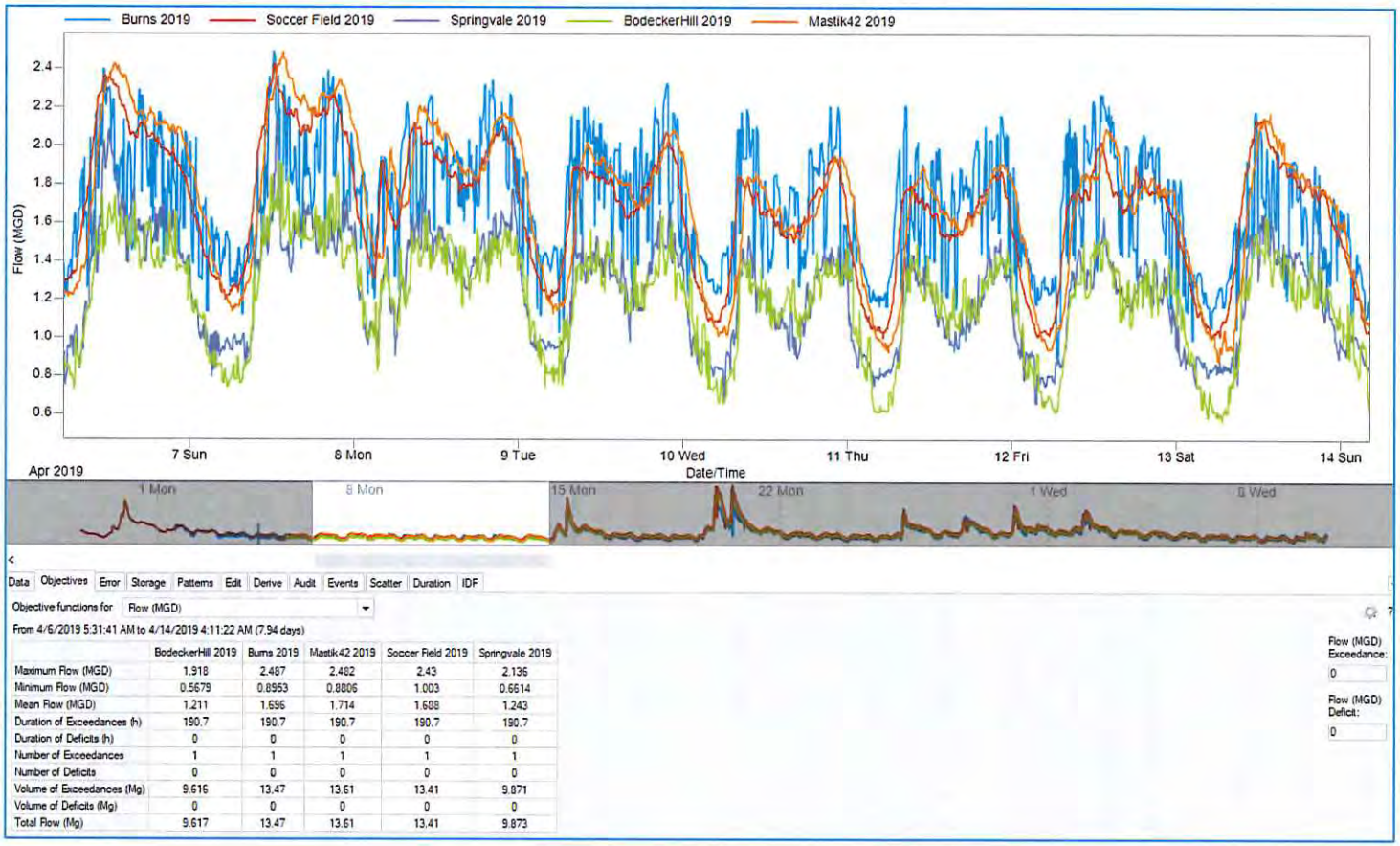
Based upon the 2019 flow meter data collected since the end of March, it appears that the model calibration for the 2018 Elizabeth site is reasonable. The calibration for the 2018 McKenzie site may be low for peak events by approximately 0.5 mgd. Since the 2018 and 2019 meter data comparison for the Southern and Springvale sites appear similar, the McKenzie difference will mean that the Springvale calibration is conservative.

The new meters between Southern and Springvale and the additional meters at Bodecker Hill and Mastick have identified a more significant issue. The Burns and Soccer Field sites are relatively close together as are the Springvale and Bodecker Hill sites. It is assumed that the sites that are relatively close together have similar flows during dry weather conditions. As shown in **Figure 1**, the dry weather flows for each group are similar. However, what is a concern is the decrease in the downstream flows. The average day flow decreases from approximately 1.6 mgd to 1.1 mgd with a loss of 0.5 mgd.

Further review of six wet weather events indicates increasing flows and volumes from Burns to the Soccer Field sites and also from Springvale to Bodecker Hill. It should be noted that none of these events were large enough to surcharge the sewer pipe depth. Again there is a decrease in volume between the Soccer Field and downstream Springvale site. The volume decrease equates to a loss of approximately 0.5 mgd. The largest 2019 rainfall flow event data is provided in **Figure 2**.

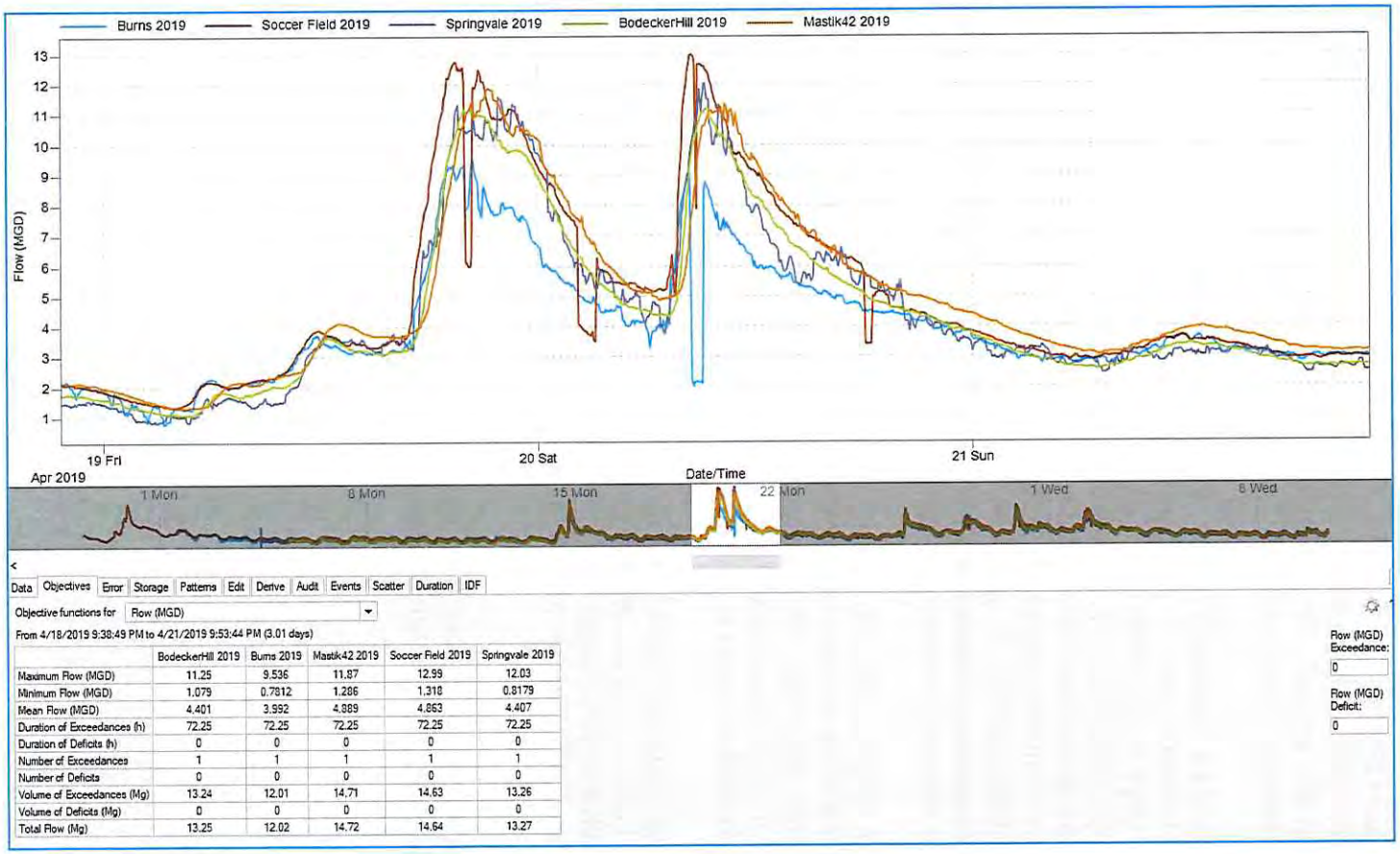
The discovery of this information initiated additional review of the 2018 Golf Club site data. This meter was originally assumed to be operating out of specifications due to its age, dissimilarity to upstream and downstream flow meter patterns and having flows greater than the Springvale flow meter. The meters were moved around to different sites for the 2019 monitoring program. The 2018 Golf Club site was not re-used for the 2019 program and was sent in for testing. The manufacturer did make repairs to the meter due to operating out of specifications. The 2019 program currently indicates similar flow at Springvale to the 2018 program utilizing a different meter. The Golf Club site was re-evaluated based upon possible flow loss in comparison to the Springvale site. The review determined a flow loss of approximately 0.3 mgd during dry weather and wet weather flows. It is currently unknown if the Golf Club meter was reading high, low, or just erroneous. Depending on the reason, this meter is assumed to have been within the loss area since the loss is less than the 0.5 mgd value from the 2019 data. Further metering locations will need to be identified to investigate this issue.

Graph 1
2019 Dry Weather Flow Loss Review



	BodeckerHill 2019	Burns 2019	Mastik42 2019	Soccer Field 2019	Springvale 2019
Maximum Flow (MGD)	1.918	2.487	2.482	2.43	2.136
Minimum Flow (MGD)	0.5679	0.8953	0.8806	1.003	0.6614
Mean Flow (MGD)	1.211	1.696	1.714	1.698	1.243
Duration of Exceedances (h)	190.7	190.7	190.7	190.7	190.7
Duration of Deficits (h)	0	0	0	0	0
Number of Exceedances	1	1	1	1	1
Number of Deficits	0	0	0	0	0
Volume of Exceedances (Mg)	9.616	13.47	13.61	13.41	9.871
Volume of Deficits (Mg)	0	0	0	0	0
Total Flow (Mg)	9.617	13.47	13.61	13.41	9.873

Graph 2
2019 Wet Weather Flow Loss Review



	BodeckerHill 2019	Burns 2019	Mastik42 2019	Soccer Field 2019	Springvale 2019
Maximum Flow (MGD)	11.25	9.536	11.87	12.99	12.03
Minimum Flow (MGD)	1.079	0.7812	1.286	1.318	0.8179
Mean Flow (MGD)	4.401	3.992	4.889	4.863	4.407
Duration of Exceedances (h)	72.25	72.25	72.25	72.25	72.25
Duration of Deficits (h)	0	0	0	0	0
Number of Exceedances	1	1	1	1	1
Number of Deficits	0	0	0	0	0
Volume of Exceedances (Mg)	13.24	12.01	14.71	14.63	13.26
Volume of Deficits (Mg)	0	0	0	0	0
Total Flow (Mg)	13.25	12.02	14.72	14.64	13.27

The loss of water from the system may not have an overall impact on the 2018 model calibration due to the major impacts on the system from wet weather flows being upstream of the Bodecker Hill SSO. At this time, it appears to be more impactful to the dry weather flows of the Mastick24 flow meter service area. There may be additional dry weather flow to consider in the model. There could be some impact on peak flow conditions, but anything appears to be minimized by backwater impacts of bends at manholes.

The issue could be related to meter programming error or flow loss from the sewer. A verification of sewer size by physical measurement should occur at all the sites to eliminate an error in pipe size or shape programmed into the meter. If flow is lost from the sewer, it is assumed that the flows are being conveyed by the sewer trench. Much of the trench may actually be constructed in sandstone. Trench conveyance can have the opportunity to re-enter the sewer as infiltration further downstream or may follow any other trench for a utility passing under the sanitary sewer. There is a storm sewer that passes under the interceptor on the west side of Columbia Road a short distance upstream of the Bodecker Hill metering site that may require investigation.

The City regularly samples the river upstream and downstream of the WWTP. It is assumed that if there is flow loss from the interceptor it would register in these sampling locations. The City has not currently identified this as an issue. A more detailed investigation through flow metering, dye testing, televising and stream/outfall sampling could be performed to assist in resolving this issue.

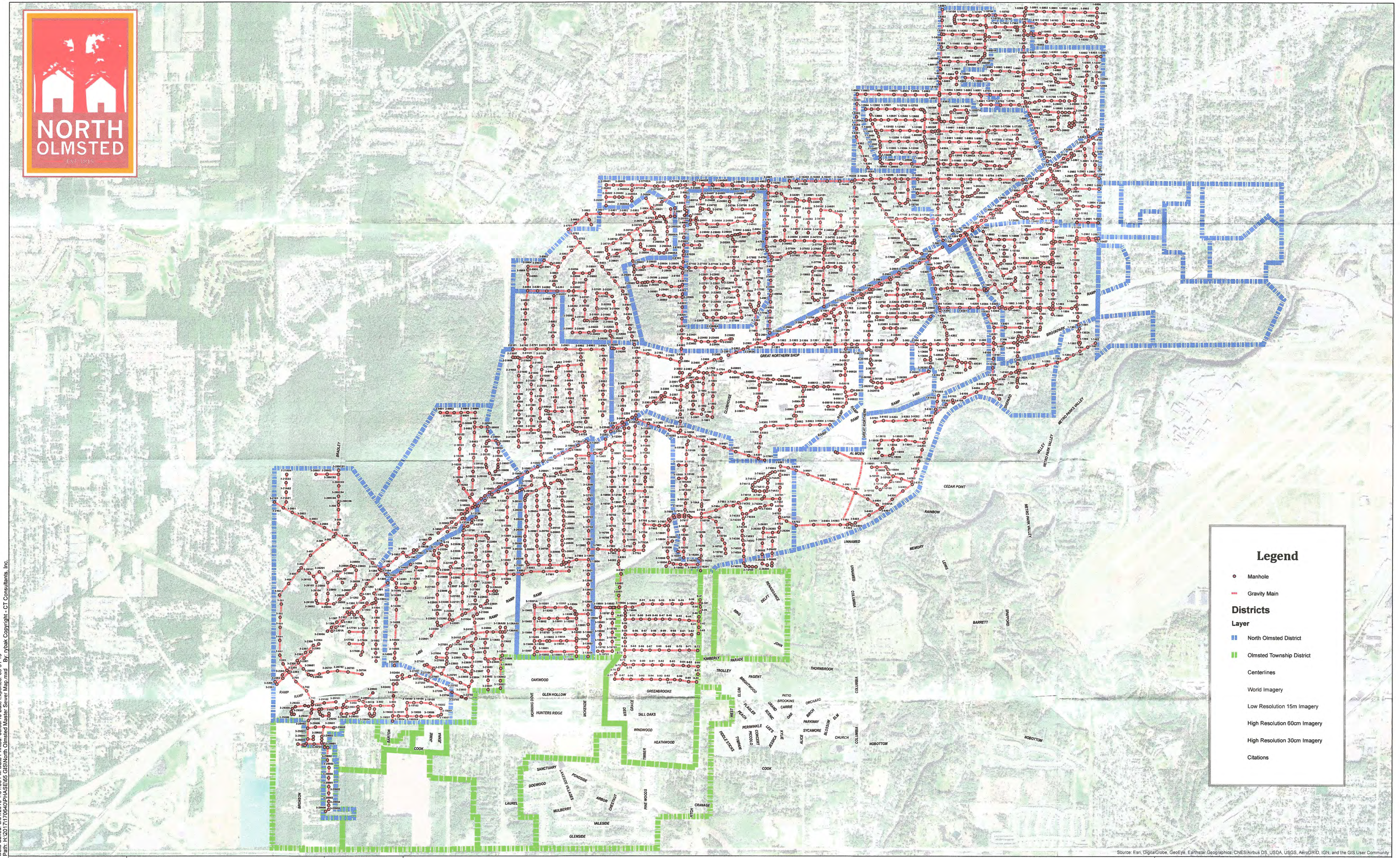
It should additionally be noted that the 2019 metering program has not experienced any spring metering flow depths similar to the 2018 program. The 2019 depths are more consistent with the 2018 program from June to December. Therefore, the assumption that there was a blockage in the interceptor during April and May of the 2018 program remains.

These findings were communicated to City staff so that they could move or add meters to the current metering program to isolate the problem area.

CITY OF NORTH OLMSTED
SOUTH INTERCEPTOR STUDY

Figures

NORTH OLMSTED
EST. 1916



Legend

- Manhole
- Gravity Main

Districts

Layer

- North Olmsted District
- Olmsted Township District
- Centerlines
- World Imagery
- Low Resolution 15m Imagery
- High Resolution 60cm Imagery
- High Resolution 30cm Imagery
- Citations

Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

Date Saved: 5/31/2019 1:51:58 PM Date Printed: 06/20/19 Date Exported: 05.31.19
Path: H:\20\7\17\90400\PHASE\05 GIS\North Olmsted Master Sewer Map.mxd By: rybak Copyright - CT Consultants, Inc.

CT consultants — engineers architects planners

0 2,500 5,000
Feet
1 inch = 1,250 feet

N W E S

City Of North Olmsted
Sanitary Sewer Map

Figure 2-1

NORTH OLMSTED
EST. 1815

NO_MacKenzie15
NO_SouthernDr30
NO_MacKenzie8
NO_StearnsRd30
NO_Barton27
NO_SoccerField
NO_Burns
NO_NorthOf480
NO_Elizabeth
NO_MacKenzie
NO_GolfClub
NO_Springvale36
NO_BodeckerHill
NO_Mastick24
NO_Mastick8
NO_SouthOf480_36
NO_Mastick42

Legend

Flow Meters

- 2018
- 2018/2019
- 2019
- Manhole
- Interceptor
- Gravity Main
- North Olmsted District
- Olmsted Township District
- Bradley PSService Area
- Golf Club Service Area
- Mastick Service Area
- Stearns Service Area
- Southern Service Area

Date Saved: 5/31/2019 1:54:14 PM Date Printed: 05/30/19 Date Exported: 05 31 19
Path: R:\CO_CUYAHOGA\North Olmsted\North Olmsted Interceptor Map.mxd By: rybak Copyright - CT Consultants, Inc.

Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

CT consultants engineers architects planners

0 1,400 2,800
Feet
1 inch = 700 feet

N
W E
S

City Of North Olmsted
South Interceptor Sanitary Study Area

Figure 2-2