



City of Sedalia, Missouri
Wastewater Master Plan
Project No. 128564

April 2022

City of Sedalia Wastewater Master Plan

prepared for

**City of Sedalia, Missouri
Wastewater Master Plan**

Project No. 128564

April 2022

prepared by

**Burns & McDonnell Engineering Company, Inc.
Kansas City, Missouri**

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LIST OF ABBREVIATIONS

<u>Abbreviation</u>	<u>Term/Phrase/Name</u>
Ac	Acres
ADDF	Average Daily Dry-Weather Flow
BMcD	Burns & McDonnell
City	City of Sedalia, Missouri
CIP	Capital Improvement Plan
d/D	Depth to Pipe Diameter Ratio
ft	Feet
Gal	Gallons
GIS	Geographic Information Systems
gpcd	Gallons per Capita per Day
In	Inches
In/hr	Inches per Hour
IDM	Inch Diameter Mile
I&I	Inflow and Infiltration
LF	Linear Feet
LoS	Level of Service
MG	Million Gallons
MGD	Million Gallons per Day
MIT	Minimum Inter-event Time
mg/l	Milligrams per Liter
NOAA	National Oceanic and Atmospheric Administration

<u>Abbreviation</u>	<u>Term/Phrase/Name</u>
Ac	Acres
NRCS	National Resources Conservation Service
PDDF	Peak Daily Dry-Weather Flow
ppm	Parts per Million
RPA	Return Period Analysis
SCS	Soil Conservation Service
USGS	United States Geological Survey
WSL	Water Surface Level
WWTP	Wastewater Treatment Plant

1.0 EXECUTIVE SUMMARY

1.1 Introduction

The City of Sedalia, Missouri (City) retained Burns & McDonnell (BMcD) to develop a wastewater master plan to understand existing and future conditions capacity limitations within the sanitary sewer system and to develop capital improvement projects. A hydraulic model was developed for this study that included sanitary sewers that are 4-inches in diameter and larger and incorporated smaller diameter pipes where needed to maintain hydraulic connectivity of the system and to support future buildout.

The purpose of this report is to provide the City with documentation of a sanitary sewer model development and a capital improvement plan. A rate study analysis will be completed under separate cover.

1.2 Flow and Rainfall Data Analysis

Flow and rainfall monitoring was performed in two 60-day periods to obtain system flow rates during both dry and wet weather conditions. A total of nine (9) flow meter data sets were collected for this study. Rainfall data was obtained from the City at three locations throughout the system for the wet weather flow analysis. Three rain gauge locations were utilized to capture spatial variation of rainfall.

A rainfall analysis was completed; all rainfall events were reviewed, and independent rainfall events were selected for hydraulic model calibration and verification purposes. Dry and wet weather flow analysis was completed to estimate average daily dry weather flow (ADDF) parameters and inflow and infiltration (I&I) rankings for each metered basin.

1.3 Data Gap Analysis

A data gap analysis was conducted to evaluate the quality and completeness of data provided by the City for the hydraulic model development. The purpose of the analysis was to provide the City with a list of missing data and recommendations to populate the data gaps.

The initial data provided by the City included information for manholes, pipes, and pump stations critical to model calibration. Gaps in network connectivity were discussed with the City and engineering judgment was used to make assumptions in locations where additional information was not available. The quality of the data available allows for confidence in our understanding of existing connectivity, pipe and manhole elevations, and operations of the sewer system.

1.4 Model Calibration and Verification

The model developed for this study includes all sanitary sewer mains represented in the City GIS that are 4-inches in diameter or larger. The model includes approximately 2,703 manholes; 750,000 linear feet (LF) of pipe; and 15 pump stations.

After the hydraulic model was constructed, calibration and verification were completed to demonstrate that the hydraulic model can adequately represent observed rainfall events for the purpose of this study.

1.4.1 Dry Weather Calibration

Dry weather flow calibration compares the observed ADDF developed for each meter catchment to model predicted dry weather flows and adjusts model loading to replicate the field measured data.

For this project, calibration targets for the model ADDF volume were within +/- 10-percent of the observed volume. Model results were also visually reviewed to demonstrate that the hydrograph predicted by the hydraulic model replicated, within a reasonable tolerance, the shape and magnitude of the metered data.

1.4.2 Wet Weather Calibration and Verification

Wet weather calibration was performed to correlate the hydrology in the hydraulic model to field measured rainfall response data during and after rainfall events. Calibration was followed by verification. Verification is used to verify that the model produces reasonable results for non-calibration rainfall events.

Hydraulic model results were compared against recorded flows to compare model predictions for overall hydrograph shape, peak wet weather flow, wet weather response volume, and the flow depth at each meter location. The hydraulic model calibration goal for instantaneous peak flow was -15-percent to +25-percent and -10-percent to + 20-percent for volume. The depth is considered calibrated when the simulated depth is within +/- 0.30 feet of the observed depth. The hydraulic model was calibrated and validated at each flow meter location for a range of rainfall events.

At each flow meter location, the hydraulic model was adjusted as necessary, within customary bounds and accepted industry practice, until the hydraulic model reasonably replicated the observed flow meter data. If the calculated percentage error for peak flow and volume was within calibration tolerances and the overall hydrograph shape was similar, the hydraulic model was considered calibrated at that flow meter location.

Although some percentage differences appear large, the absolute differences in peak flow or volume are small. This can occur when low flow, low depth, or high sensitivity causes a meter to fail calibration tolerances, but is not a cause for concern in these circumstances.

When possible, three (3) rain events are used to calibrate each meter and two (2) events are used to verify the model. However, due to limited rainfall events during the monitoring period, two (2) events were calibrated for most meters and one (1) event was used to verify the model.

In some cases, hydraulic model calibration to both peak flow rate and volume was not possible due to the flow meter data exhibiting flow irregularities, such as low flow conditions or velocity dropouts during the peak of the event, as noted for specific events. In these instances, the model was calibrated using depth data instead. The Table 1-1 provides a summary of the wet weather calibration.

Table 1-1: Wet Weather Calibration Summary

Meter	Calibration Event	Peak Flow Difference		Volume Difference		Depth Difference
		(Million Gallons Per Day)	(%)	(Million Gallons)	(%)	(ft)
CE1-30 ¹	4/28/2021	-	-	-	-	-0.05
	5/9/2021	-	-	-	-	-0.02
CE7-1	4/23/2021	-0.38	-11.2%	-0.21	-14.7%	-0.23
	4/28/2021	0.20	4.7%	-0.13	-3.3%	-0.17
CW1-3	5/16/2021	-0.34	-7.9%	-0.07	-1.2%	-0.66
	5/20/2021	-0.82	-43.6%	-0.89	-34.9%	-0.12
N3-5	4/28/2021	0.45	32.9%	<0.01	0.5%	0.25
	5/9/2021	<0.01	0.1%	-0.15	-15.4%	0.04
N1-31	6/11/2021	-0.13	-2.2%	-0.13	-9.7%	0.30
	6/25/2021	-0.03	-0.5%	0.06	1.1%	0.34
SEB3-6	6/11/2021	0.34	17.2%	0.08	19.1%	0.10
	6/28/2021	<0.01	0.9%	-0.10	-26.2%	-0.05
SEC3-20 ¹	8/12/2021	-	-	-	-	0.03
SED1-29 ¹	7/12/2021	-	-	-	-	<-0.01
	7/25/2021	-	-	-	-	-0.01
SWLST-25	6/24/2021	0.18	20.5%	0.07	17.1%	-
	7/15/2021	-0.10	-5.1%	-0.29	-17.6%	-
Notes:						
1. Supercritical flow. Calibrated to depth.						

Table 1-2 provides peak flow, volume, and depth differences for each verification rainfall event. Wet weather verification graphs comparing predicted and meter recorded flow can be found in Appendix D.

Table 1-2: Wet Weather Verification Summary

Meter	Verification Event	Peak Flow Difference		Volume Difference		Depth Difference
		(Million Gallons Per Day)	(%)	(Million Gallons)	(%)	(ft)
CE1-30 ¹	4/18/2021	-	-	-	-	-0.13
	5/20/2021	-	-	-	-	-0.06
CE7-1	4/8/2021	0.39	17.7%	0.29	39.5%	-0.10
	4/18/2021	-0.53	-22.3%	-0.06	-16.7%	-0.26
CW1-3	4/23/2021	-1.18	-47.1%	-0.31	-34.5%	0.15
	5/9/2021	0.06	1.5%	0.85	52.2%	-5.99
N3-5	4/8/2021	0.01	2.1%	0.03	8.3%	0.05
	4/9/2021	-0.49	-29.3%	0.21	19.7%	-0.03
N1-31	6/20/2021	0.15	8.7%	-0.08	-12.5%	0.17
	6/25/2021	1.07	18.1%	-0.05	-4.3%	0.51
SEB3-6	6/24/2021	1.34	54.1%	0.89	52.0%	0.24
SEC3-20 ¹	9/4/2021	-	-	-	-	0.08
SED1-29 ¹	7/15/2021	-	-	-	-	0.03
SWLST-25	6/25/2021	0.39	18.5%	0.27	14.8%	-
Notes:						
1. Supercritical flow. Calibrated to depth.						

The hydraulic model calibration and verification was found to reasonably reproduce metered sewer flow and volume results during rainfall events.

1.5 Existing Conditions Analysis

The existing conditions analysis consisted of identification of existing issues within the sanitary system for the selected assessment event. The evaluation was performed using the calibrated model and the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 2-year, 6-hour design rainfall event.

The existing system was evaluated under the design storm to predict the projected sanitary sewer pipes and manholes that have a reduced level of service. The level of service (LoS) of manholes within the sewer system were defined as the water surface level (WSL) less than three feet below manhole rim elevation. The conduit level of service is defined as the WSL exceeding the crown of pipe.

The existing system analysis indicated pump station capacity constraints and pipe capacity constraints within the sanitary sewer system. The pump station and pipe capacity constraints result in back water conditions upstream of constraints. The existing system analysis also indicated reduced level of service upstream of the equalization basins (EQ) due to inlet and outlet operations at the basins. Further, the existing

conditions analysis identified the reduced level of service in manholes and conduits with constraints and those adversely impacted by those constraints.

1.6 Capital Improvement Plan

Proposed planning level sewer improvements were developed as part of the Capital Improvement Plan (CIP). These improvements addressed reduced LoS in manholes and conduits for the existing and future conditions. The future conditions were evaluated in two tiers of Level of Service. The tiers are described as follows:

- Tier 1:
 - Projects triggered by manhole LoS (WSL less than 3 feet from grade).
- Tier 2:
 - Projects adjacent to Tier 1 CIPs to improve system hydraulics. Aimed at reducing the potential for basement backups.

The CIP considers estimates for future growth within the City limits and expansion outside of current City limits. Future growth includes industrial, commercial, and residential growth. Two phases of growth have been projected and considered in CIP development:

- Phase 1: Projected 5-year growth.
- Phase 2: Projected 20-year growth.

The proposed improvements were divided into 24 projects for the CIP. Note that CIP numbers are not in sequential order due to refinement of the CIP list. The following control measures were proposed for the various CIP projects to address LoS issues within the existing sanitary system.

- 30-percent I&I Reduction
- Relief Sewers
- Increased Pipe Capacity
- Increased Pumping Capacities

Projects were divided based on location within the overall sewer system and appropriate phasing of development. Table 1-3 summarizes the proposed CIP projects and the planning level opinion of probable costs associated with each project. Figure 1-1 and Figure 1-2 illustrate the location of the proposed projects based on the future development phase 1 and 2, respectively.

Table 1-3: CIP Project Summary

Phasing	CIP #	CIP Project	Opinion of Probable Construction Costs ²	Opinion of Probable Engineering Costs ³	Opinion of Probable Costs
Existing	1A	I&I Assessment & Reduction Program Development	-	\$250,000	\$250,000
	1B	I&I Assessment & Reduction - Basin CW1-3	\$3,670,000	\$551,000	\$4,221,000
	1C	I&I Assessment - Basin CE7-1	\$140,000	\$21,000	\$161,000
	1D	I&I Assessment - Basin CE1-30	\$170,000	\$26,000	\$196,000
	1E	I&I Assessment - Basin N1-31	\$160,000	\$24,000	\$184,000
	1F	I&I Assessment - Basin N3-5	\$60,000	\$9,000	\$69,000
	1G	I&I Assessment - Basin SEB3-6	\$80,000	\$12,000	\$92,000
	2	Main Street Lift Station Improvements	\$838,000	\$126,000	\$964,000
	3	Central Basin Additional Flow Metering	\$40,000	\$6,000	\$46,000
	6	Pelham Drive Relief Sewer	\$490,000	\$74,000	\$564,000
	8	Central Plant Relief Sewer & Weir Adjustment	\$1,810,000	\$272,000	\$2,082,000
	9	Industrial Road Relief Sewer	\$8,450,000	\$1,268,000	\$9,718,000
	10	Limit Ave and 7th Street Relief Sewer	\$1,230,000	\$185,000	\$1,415,000
	11	Main Street Relief Sewer	\$2,060,000	\$309,000	\$2,369,000
	12	5th Street to 11th Street Relief Sewer	\$6,080,000	\$912,000	\$6,992,000
	13	Clinton Road Relief Sewer	\$790,000	\$119,000	\$909,000
	14	32nd Street Relief Sewer	\$1,170,000	\$176,000	\$1,346,000
	15	Lamine Avenue Relief Sewer	\$4,170,000	\$626,000	\$4,796,000
	16	20th Street Relief Sewer	\$2,350,000	\$353,000	\$2,703,000
	17	E Broadway Boulevard Relief Sewer	\$520,000	\$78,000	\$598,000
	18	Missouri Press Metals Lift Station Improvements	\$1,100,000	\$165,000	\$1,265,000
	19	Marshall Avenue and E 14th Street Relief Sewer	\$1,480,000	\$222,000	\$1,702,000
	20	Harding Avenue Relief Sewer	\$1,070,000	\$161,000	\$1,231,000
	25	New North Plant Interceptor Sewer	\$5,400,000	\$810,000	\$6,210,000
	26	Future North Lift Station 1 and Gravity Sewer	\$15,680,000	\$2,352,000	\$18,032,000
	Phase 1	27	Future Northwest Lift Station 5 and Gravity Sewer	\$1,380,000	\$207,000
28		Future West Lift Station 4 and Gravity Sewer	\$1,350,000	\$203,000	\$1,553,000
29		Future Southwest Lift Station 3 and Gravity Sewer	\$2,010,000	\$302,000	\$2,312,000
Phase 2	30	Future North Lift Station 2 and Gravity Sewer	\$3,040,000	\$456,000	\$3,496,000
Future Phase	31	Future Central Plant Lift Station and Forcemain	\$10,460,000	\$1,569,000	\$12,029,000
1. Opinion of Probable Costs were developed based on 2022 unit costs.					
2. Opinion of Probable Construction Costs include a 20% contingency.					
3. Opinion of Probable Engineering Costs are assumed at 15% of Opinion of Probable Construction Costs.					

Figure 1-1: Phase 1 Capital Improvement Projects

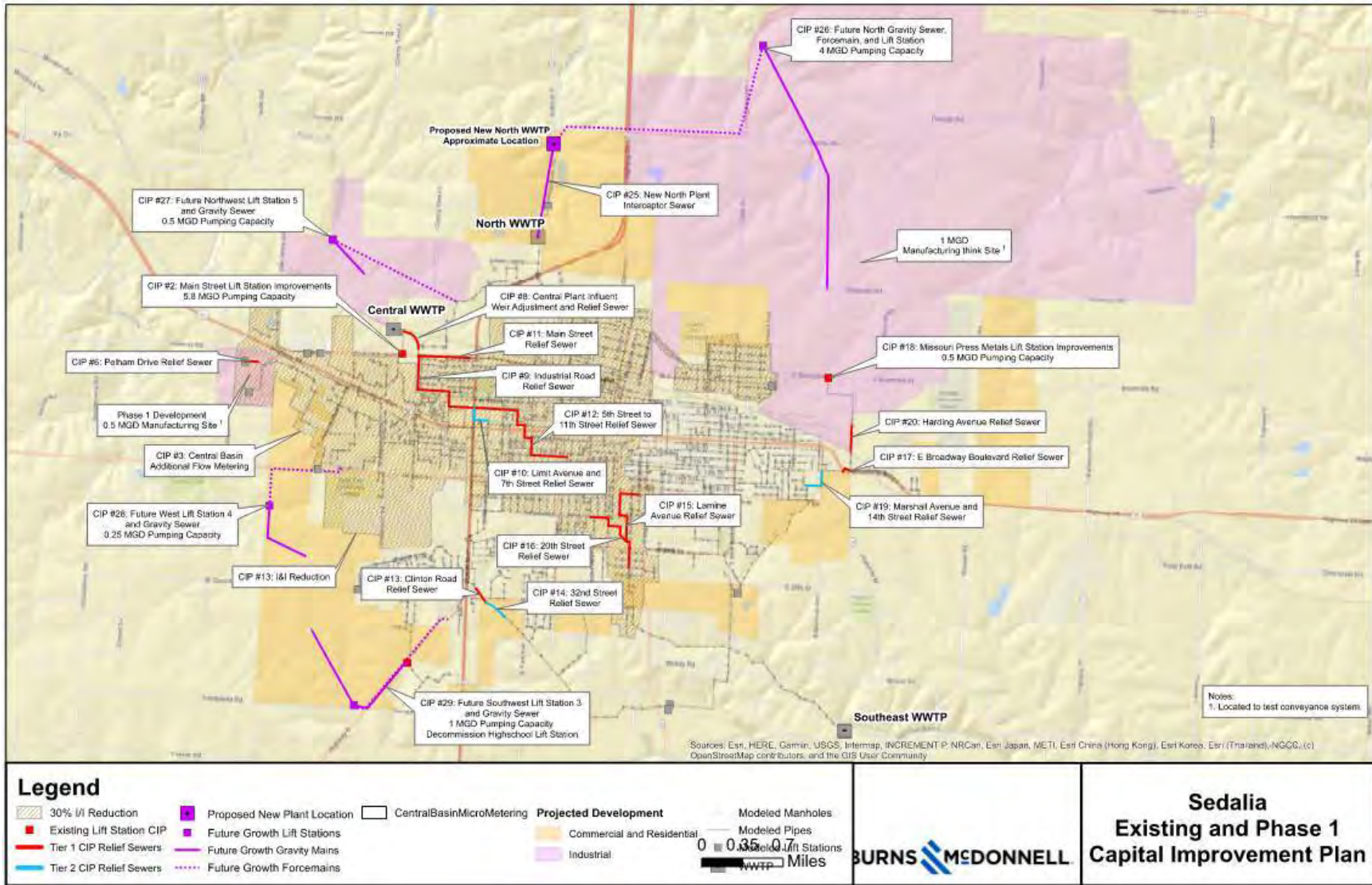
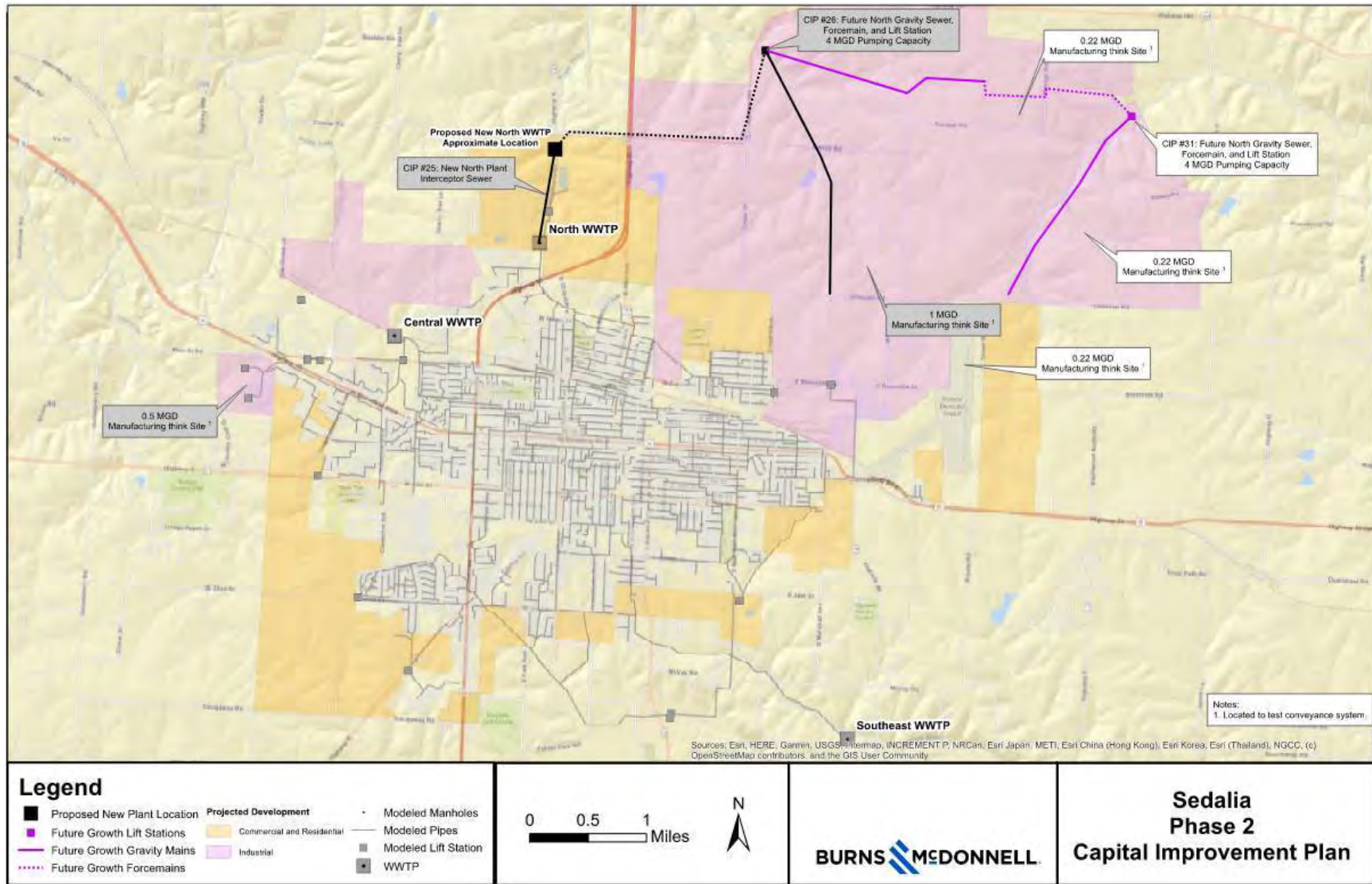


Figure 1-2: Phase 2 Capital Improvement Projects



2.0 INTRODUCTION

The City of Sedalia, Missouri (City) retained Burns & McDonnell (BMcD) to develop a wastewater master plan to understand existing and future conditions capacity limitations and proposed future improvements. The model used in the analysis includes the sanitary sewers that are 4-inches and larger. The results of the analysis were used to develop a Capital Improvement Plan for the existing and future system conditions.

The purpose of this report is to document dry and wet weather flow analysis and model development used in the evaluation of the existing system and Capital Improvement Plan. Specifically, this report summarizes the following components of the project:

- Flow and Rainfall Data Analysis
- Data Gap Analysis
- Model Development
- Model Calibration and Verification
- Existing Conditions Analysis
- Future Conditions Development
- Development of CIP Projects
- Phasing of CIP Projects
- Opinion of Probable Cost

3.0 FLOW AND RAINFALL DATA ANALYSIS

Flow and rainfall monitoring were performed to establish a relationship between precipitation (rainfall intensity and rainfall volume) and system flow rates.

3.1 Flow and Rainfall Monitoring Locations

Nine (9) flow meter locations were selected to monitor flows throughout the entire collection system. Flow monitoring site information is summarized in Table 3-1 and shown in Figure 3-1. Flow meters were installed in the incoming pipes of the manholes for best hydraulic conditions. Flow Meters were installed in multiple phases. Rain gauge locations are provided in Table 3-2 and shown in Figure 3-1.

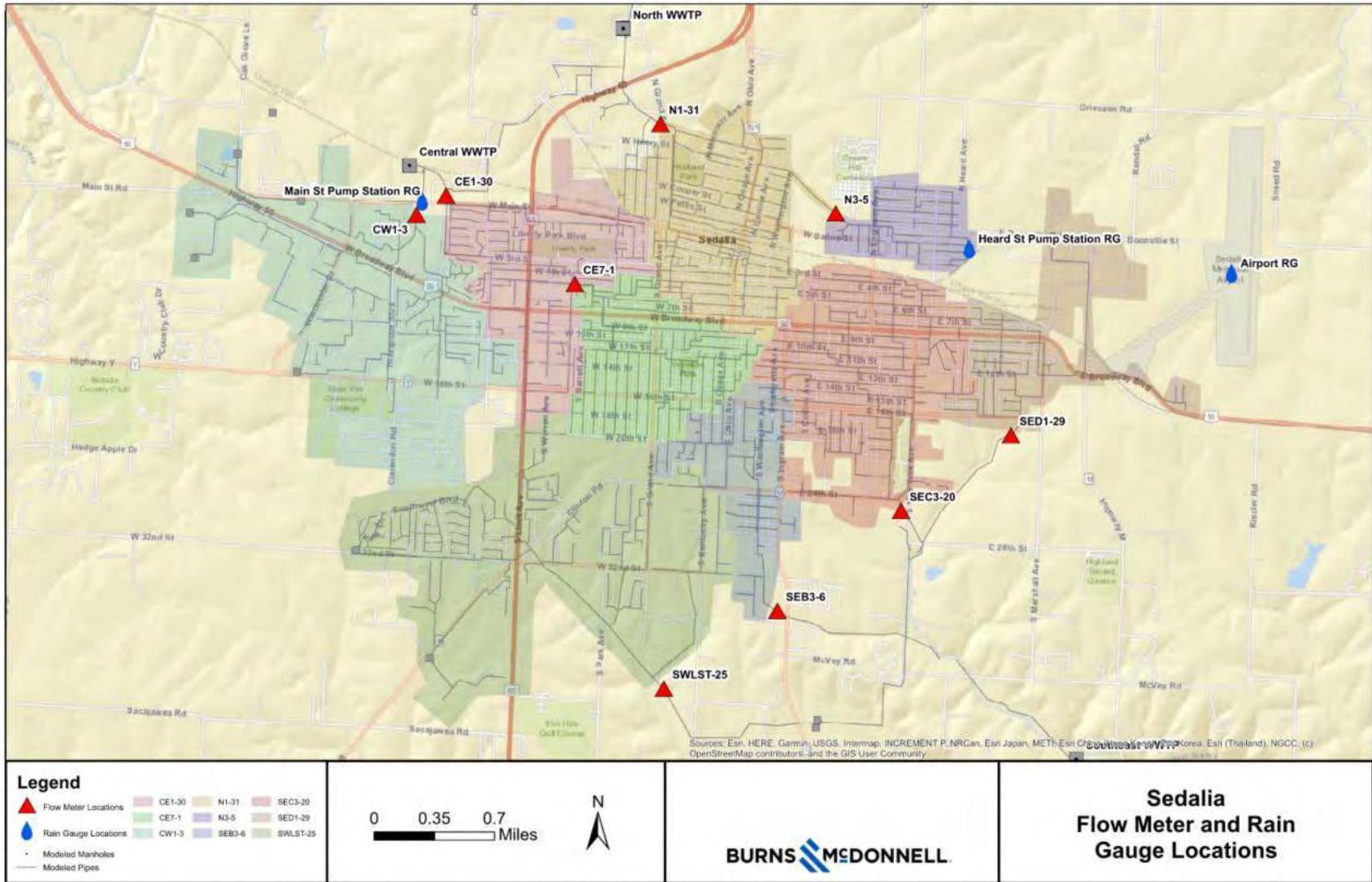
Table 3-1: Flow Meter Locations

Flow Meter ID	Pipe Diameter (inches)	Start Date	End Date
CE1-30	27	4/6/2021	5/25/2021
CE7-1	18	4/1/2021	5/26/2021
CW1-3	21	3/26/2021	5/26/2021
N3-5	12	3/24/2021	5/26/2021
N1-31	30	5/26/2021	9/14/2021
SEB3-6	18	6/2/2021	7/31/2021
SEC3-20	36	6/7/2021	7/7/2021
SED1-29	24	5/26/2021	9/14/2021
SWLST-25	24	6/2/2021	9/14/2021

Table 3-2: Rain Gauge Locations

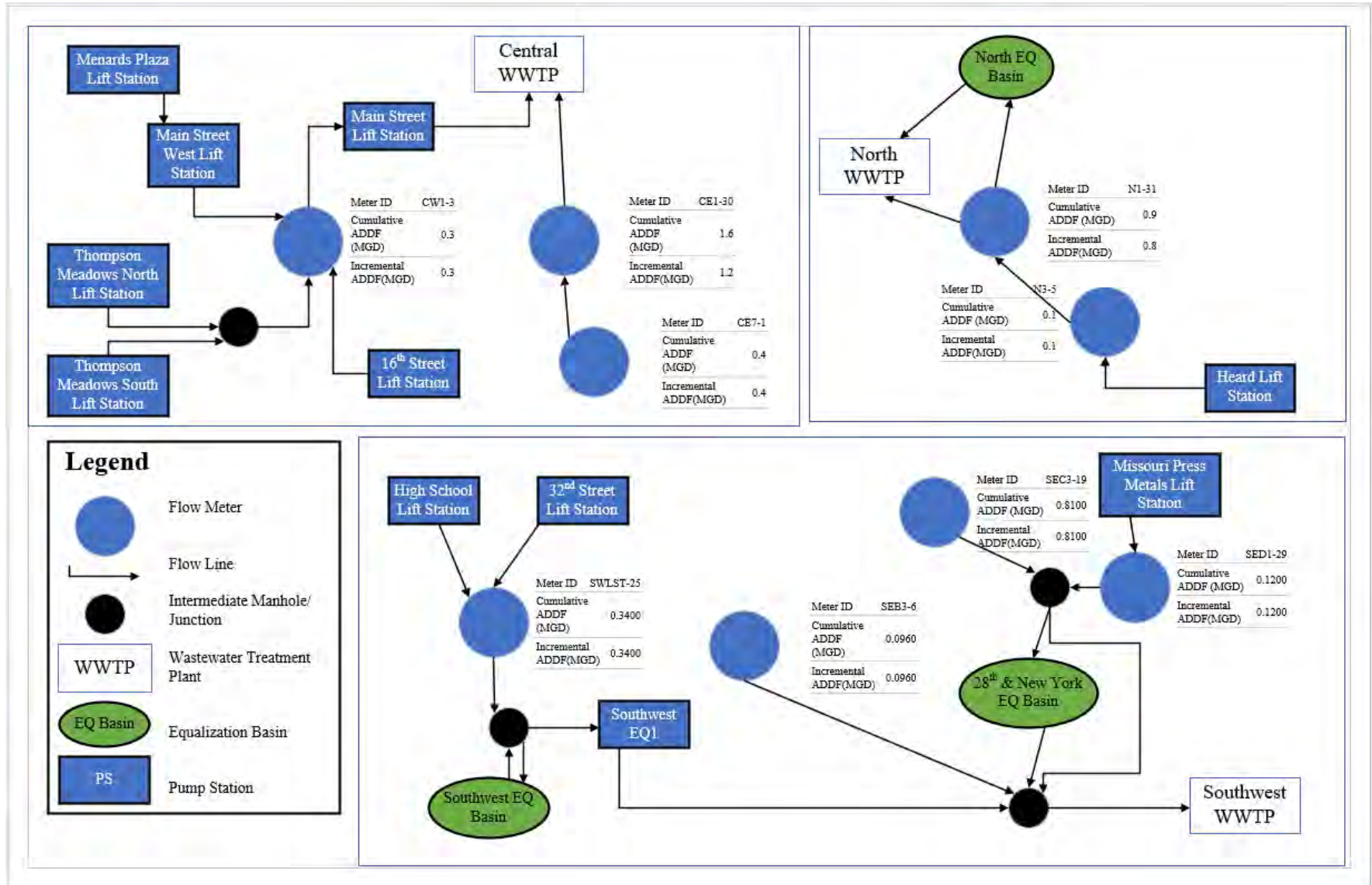
Rain Gauge ID	Location
Main Street	Main Street Pump Station 2919 W Main St
Airport	Sedalia Airport 1900 E Boonville St
Heard	Heard Street Pump Station 311 N Heard Ave

Figure 3-1: Flow Meter and Rain Gauge Locations



A flow meter schematic showing the relationship between flow meters is presented in Figure 3-2. This illustrates the cumulative and incremental ADDF as the flow is routed through the system for a typical weekday. The meter data represents the cumulative flow observed at the meter location. The incremental volume per meter represents volume produced by the individual meter catchment, excluding the flow coming from upstream meters.

Figure 3-2: Weekday ADDF Schematic



3.2 Meter Catchments

Meter catchments represent the tributary area draining to flow meters. Flow meter locations were established throughout the collection system to provide information about system flow that can be used to support hydraulic model calibration. In addition, the flow data for meter catchments was used to rank the catchments for I&I based on the estimates of wet-weather flow volume. A total of nine (9) flow metering basins have been delineated. Flow meter ID and associated area in acres are provided in Table 3-3.

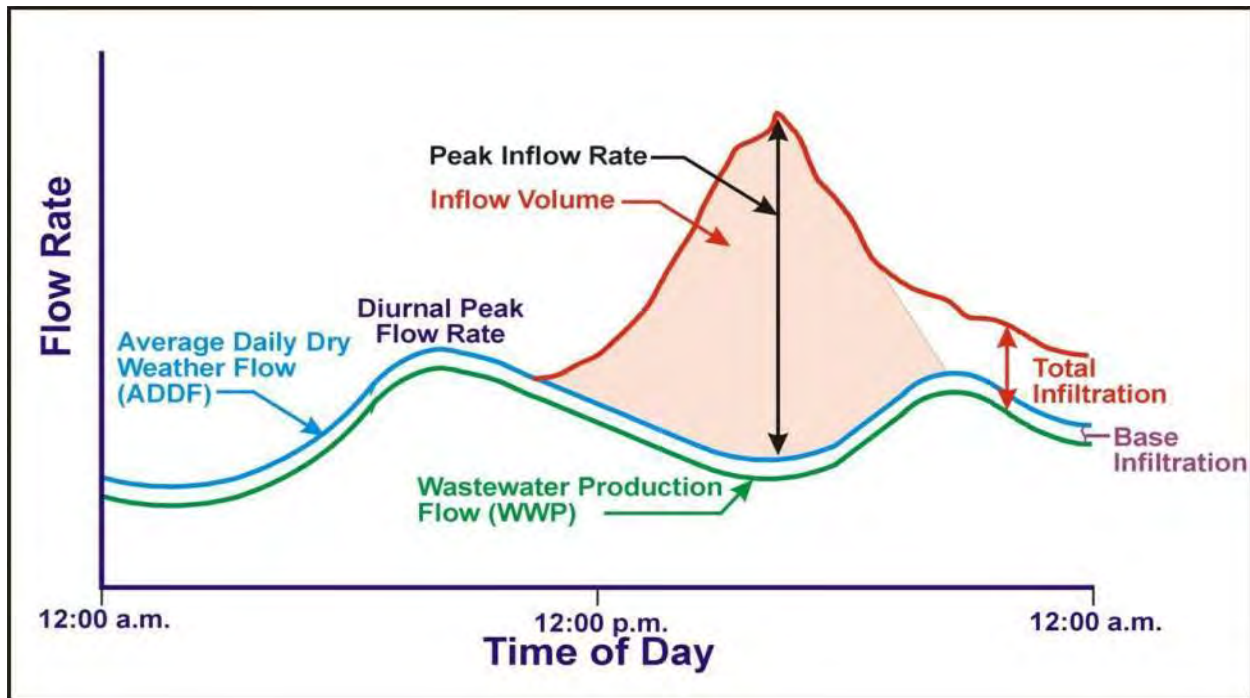
Table 3-3: Meter Catchment Area

Flow Meter ID	Incremental Area (acres)
CE7-1	475.66
CE1-30	590.98
CW1-3	1351.23
N3-5	220.85
N1-31	659.03
SEB3-6	358.99
SEC3-20	815.44
SED1-29	494.22
SWLST-25	1531.66

3.3 Flow Analysis

Flow analyses were performed to deconstruct flow meter hydrographs into the various components of sanitary sewer system flows. Average daily dry-weather flow (ADDF), peak daily dry-weather flow (PDDF), and I&I were estimated through the analyses described in the following sections. Figure 3-3 illustrates the flow components of a typical sanitary flow response hydrograph.

Figure 3-3: Typical Hydrograph Flow Components



3.3.1 Dry Weather Analysis

Dry weather flow is defined as the flow present in the sanitary sewer without the influence of rainfall. The ADDF includes wastewater production and infiltration present during dry, low groundwater conditions. Dry weather flow does not include inflow or groundwater infiltration due to rainfall. For this study, a dry weather day assumes no rainfall that day or the previous day. Additionally, the flow hydrographs were reviewed to exclude days with residual high groundwater entering the system from a previous rain event.

Typically, between 3 and 6 representative dry weather days were selected and averaged to estimate ADDF. Due to a minimal variance between weekday and weekend ADDF curves, only the weekday ADDF curves for each flow meter are presented in this section. Fridays were excluded from both weekend and weekday ADDF curves. The weekday and weekend ADDF curves are provided in Appendix A.

PDDF is the average of maximum one-hour flow rate for observed dry weather days. The PDDF was estimated by averaging the maximum one-hour flow rate for the selected dry weather days at each meter. The dry weather peaking factor was then estimated as the ratio of PDDF to ADDF.

Minimum night-time flow is the lowest 3-hour flow and typically occurs between 2 AM and 5 AM. The average minimum night-time flow was estimated by averaging the lowest 3-hour flow rate for the selected dry weather days.

A summary of the flow parameters (ADDF, PDDF, peaking factor, and average minimum night-time flow) for each flow meter is provided in Table 3-4 for weekdays. Cumulative flow is the flow observed at the meter location and incremental flow per meter represents volume produced by the individual meter catchment, excluding the flow coming from upstream meters. As shown in the table, the cumulative ADDF and PDDF increases downstream through the system. The peaking factor decreases moving downstream since peak flows are attenuated as the time of concentration increases.

Table 3-4: Dry Weather Flow Parameters Summary - Weekdays

Flow Meter ID	Average Min. Night-Time Flow (Million Gallons per Day)	Incremental ADDF (Million Gallons per Day)	Cumulative ADDF (Million Gallons per Day)	Incremental PDDF (Million Gallons per Day)	Cumulative PDDF (Million Gallons per Day)	Peaking Factor
CE1-30	1.37	1.20	1.60	1.30	1.80	1.14
CE7-1	0.37	-	0.40	-	0.52	1.17
CW1-3	0.23	-	0.40	-	0.87	2.34
N3-5	0.04	-	0.05	-	0.08	1.45
N1-31	0.62	0.70	0.75	0.84	0.92	1.23
SEB3-6	0.07	-	0.10	-	0.12	1.24
SEC3-20	0.28	-	0.81	-	1.16	1.45
SED1-29	0.06	-	0.12	-	0.22	1.91
SWLST-25	0.22	-	0.34	-	0.49	1.43

3.3.2 Rainfall Analysis

The minimum inter-event time (MIT) is the minimum number of dry hours between rainfall events. MIT is defined such that system flows have returned (or substantially returned) to dry weather flow conditions. In this way, rainfall events are isolated and there is a clearer relationship between rainfall and wet weather flow. For this project, discrete rainfall events were selected based on an MIT of 12 hours. Where system flows did not substantially return to dry weather flow conditions between rainfall events, rainfall events were combined for I&I analysis, described in the following sections.

Not all rainfall events observed were suitable for use in flow analysis. A rainfall must be large enough to produce a noticeable flow response in the sewer system (as measured by the flow meter) to be used in the analysis, but small enough that the sewer system does not become surcharged. This is because peak wet weather flow response to rainfall is desired, and this cannot be estimated during restricted, hydraulic surcharge conditions.

Rainfall events are measured at three rain gauges, summarized below in Table 3-5.

Table 3-5: Rain Gauge Locations

Rain Gauge	Total Depth (inches)	Duration
Main Street PS Rain Gauge	27.4	3/1/2021 through 9/15/2021
Sedalia Airport Rain Gauge	29.1	3/1/2021 through 9/15/2021
Heard Street PS Rain Gauge	24.2	3/1/2021 through 9/15/2021

3.3.2.1 Rainfall Event Selection

The entire flow meter and rain gauge record was reviewed to identify rainfall events that produced an appreciable response in the flow meter. Not all rainfall events are suitable for hydraulic model calibration and verification. The rainfall event should be large enough to produce a response in all (or most) flow meters but small enough that the sewer system does not become surcharged. It is typical for rainfall events representing small (<0.5 inches), medium (0.5 to 1.5 inches), and large (>1.5 inches) events to be selected for calibration and verification at each metered location. It should be noted that all observed rain events during the monitoring period were smaller than the 2-year, 6-hour design storm. Periods with substantial inconsistent or erratic flow and/or rainfall data are avoided.

Events selected for calibration and verification were characterized by total rainfall depth, duration, and peak hourly intensity. Table 3-6 provides a summary of the selected rainfall events.

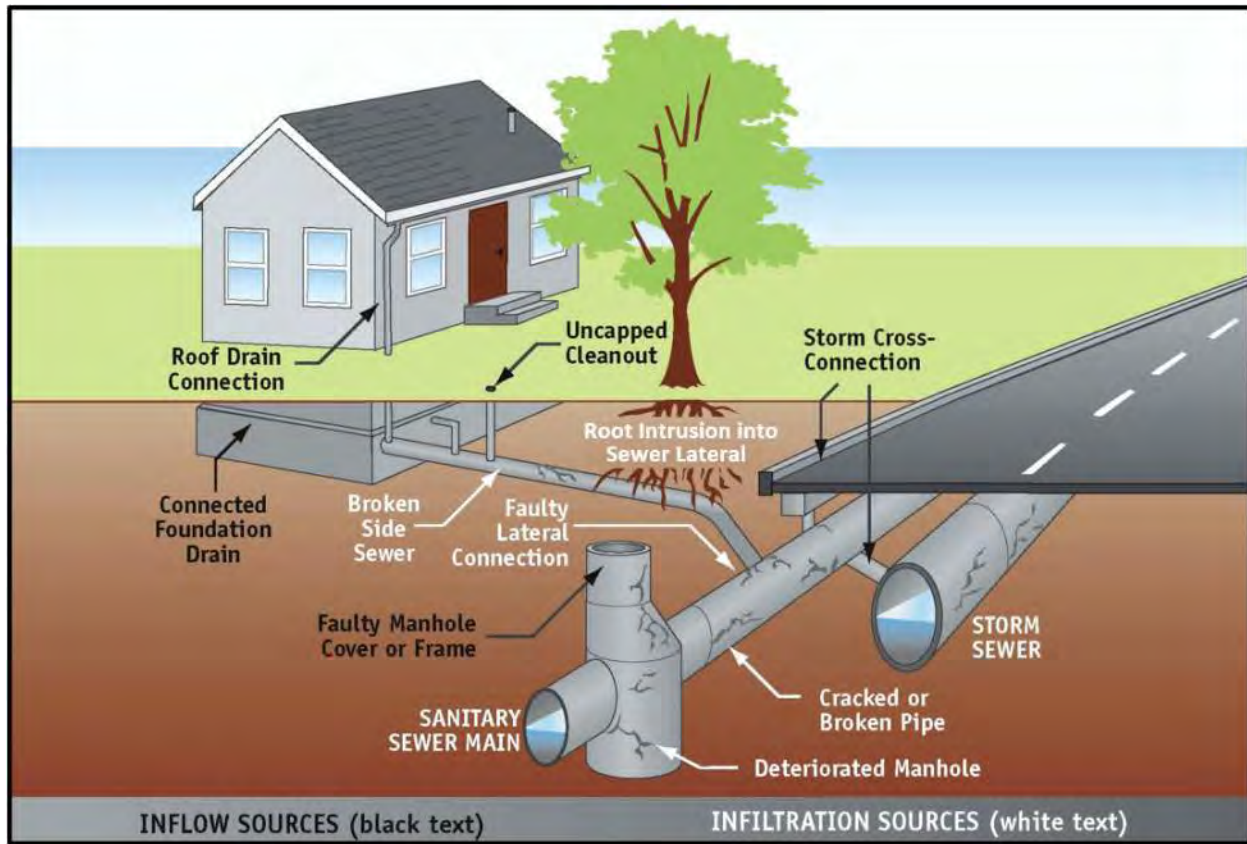
Table 3-6: Rainfall Event Summary

Rainfall Event	Rain Gauge	Total Rainfall Depth (inches)	Duration (hours)	Hourly Peak Intensity (inches/hour)
4/8/2021	Main Street	0.18	11.75	0.06
	Airport	0.30	11.30	0.10
4/9/2021	Airport	1.60	18.30	0.40
4/18/2021	Main Street	0.23	5.50	0.15
4/23/2021	Main Street	0.15	2.25	0.12
4/28/2021	Main Street	1.15	5.75	0.59
	Airport	1.60	15.30	1.20
5/9/2021	Main Street	1.17	4.50	0.70
	Airport	1.20	10.30	0.80
5/16/2021	Main Street	2.18	44.25	0.81
5/20/2021	Main Street	0.47	20.00	0.15
6/11/2021	Main Street	0.69	2.75	0.49
	Airport	0.70	3.25	0.50
6/20/2021	Main Street	0.36	4.75	0.23
6/24/2021	Main Street	2.83	71.25	0.67
	Airport	3.10	44.30	1.00
6/25/2021	Airport	0.10	3.25	0.03
6/28/2021	Airport	0.40	20.30	0.10
7/12/2021	Airport	0.50	10.30	0.30
7/15/2021	Airport	1.00	12.30	0.80
7/25/2021	Airport	0.30	0.25	0.30
8/12/2021	Airport	0.60	15.30	0.40
9/4/2021	Airport	0.70	0.25	0.02

3.3.3 Inflow and Infiltration (I&I) Assessment

Sanitary sewer systems are impacted by I&I that enters the system through cracks in the pipes and manholes or unintended stormwater connections (e.g., downspouts, sump pumps, etc.). I&I into the sanitary sewer system is identified by an increase in sewer flow during and after a rainfall event. Figure 3-4 illustrates typical I&I sources for sanitary sewer systems.

Figure 3-4: Typical I&I Sources



Source: King County Department of Natural Resources and Parks

Inflow is observed as a fast flow reaction, corresponding with, or immediately following the peak of the rainfall event. Infiltration is observed as a slower flow reaction. Infiltration is represented in the tail of the wet weather flow reaction and is higher than average dry weather flow. The inflow and infiltration flow components of a typical hydrograph are illustrated in Figure 3-3. The flow reaction to each storm event was analyzed to divide the inflow and infiltration volumes between the fast and slow reactions.

The I&I volumes identified from the flow monitoring data were evaluated and ranked. Three (3) I&I volume to rainfall indices were evaluated to eliminate biases in the data to specific meter catchment characteristics, such as large meter catchment area or high lineal feet of pipe. The three indices evaluated were:

- I&I volume (gal) / Rainfall (in)
- I&I volume (gal) / Inch Diameter Mile (IDM) / Rainfall (in)
- I&I volume (gal) / Meter Catchment Area (acres) / Rainfall (in)

These indices were evaluated for each of the selected flow meter catchments and ranked.

Total wet-weather volume was estimated by taking total wet weather flow and subtracting the ADDF, integrated over time during wet weather. Inflow and infiltration were estimated using the same process.

A linear regression relationship between the volume of total I&I versus total rainfall depth was assumed for the selected rainfall events. A linear regression relationship between separate inflow and infiltration versus rainfall depth was also assumed for selected rainfall events. The linear regression obtained from observed data was extrapolated to the rainfall depth corresponding to design storm, as defined in Section 6.1. This provides an estimate of I&I in the sewer system, the purpose of the I&I estimate is to normalize the comparison of meter catchments.

The 5-year, 6-hour event was selected as the assessment storm for these analyses. Normalization of the I&I indices allow comparison of I&I rates across different meter catchments and across different flow monitoring periods. The projected volumes of total I&I, inflow, and infiltration versus rainfall depth are provided in Appendix B. The 5-year, 6-hour event is shown in these volume projections and was used to predict the existing level of service in the system.

3.3.3.1 I&I Volume per Inch of Rainfall Index

A relationship between I&I volume and rainfall was estimated for each meter. Total I&I, inflow and infiltration volumes were estimated and ranked based on incremental volume.

The meter data represents the cumulative flow observed at the meter location. The incremental volume per meter represents I&I volume produced by the individual meter catchment, excluding the flow coming from upstream meters. Incremental volumes were also evaluated for inflow and infiltration. Meter catchments with high incremental volumes for inflow indicate surface runoff entering the system through manhole covers, exposed broken pipe, defective pipe joints, cross connections between storm sewers and sanitary sewers, and/or illicit connections of downspouts, area drains, sanitary cleanouts, or storm inlets.

Meter catchments with high incremental infiltration volume indicate groundwater is entering a sewer system through broken pipes and sanitary service laterals, defective pipe joints, or illegal connections of foundation drains. Table 3-7, Table 3-8, and Table 3-9 summarize the volume of I&I, inflow, and infiltration per inch of rainfall.

Table 3-7: Volume of I&I per Inch of Rainfall

Flow Meter ID	Volume I&I		Volume I&I		Rank ²
	(Thousand Gallons) ¹		(Thousand Gallons per Inch of Rainfall)		
	Cumulative	Incremental	Cumulative	Incremental	
CE7-1	6,000	-	1813	-	2
CE1-30	12,700	6,700	3837	2024	1
CW1-3	3,600	-	1088	-	4
N3-5	1,700	-	514	-	7
N1-31	7,700	6,000	2326	1813	2
SEB3-6	1,700	-	514	-	7
SEC3-20	3,400	-	1027	-	5
SED1-29	1,500	-	453	-	9
SWLST-25	1,800	-	544	-	6
Notes:					
1. Rainfall depth based on NOAA 5-year, 6-hour event of 3.31 inches.					
2. Basin ranking based on volume generated from 1-inch of rainfall.					

Table 3-8: Volume of Inflow per Inch of Rainfall

Flow Meter ID	Volume Inflow		Volume Inflow		Rank ²
	(Thousand Gallons) ¹		(Thousand Gallons per Inch of Rainfall)		
	Cumulative	Incremental	Cumulative	Incremental	
CE7-1	3,000	-	906	-	3
CE1-30	6,300	3,300	1903	997	2
CW1-3	2,400	-	725	-	4
N3-5	1,000	-	302	-	7
N1-31	4,700	3,700	1420	1118	1
SEB3-6	1,000	-	302	-	7
SEC3-20	2,200	-	665	-	5
SED1-29	500	-	151	-	9
SWLST-25	1,200	-	363	-	6
Notes:					
1. Rainfall depth based on NOAA 5-year, 6-hour event of 3.31 inches.					
2. Basin ranking based on volume generated from 1-inch of rainfall.					

Table 3-9: Volume of Infiltration per Inch of Rainfall

Flow Meter ID	Volume Infiltration		Volume Infiltration		Rank ²
	(Thousand Gallons) ¹		(Thousand Gallons per Inch of Rainfall)		
	Cumulative	Incremental	Cumulative	Incremental	
CE7-1	3,200	-	967	-	2
CE1-30	6,600	3,400	1994	1027	1
CW1-3	1,200	-	363	-	5
N3-5	700	-	211	-	7
N1-31	3,100	2,400	937	725	3
SEB3-6	700	-	211	-	7
SEC3-20	1,300	-	393	-	4
SED1-29	500	-	151	-	9
SWLST-25	1,100	-	332	-	6
Notes:					
1. Rainfall depth based on NOAA 5-year, 6-hour event of 3.31 inches.					
2. Basin ranking based on volume generated from 1-inch of rainfall.					

3.3.3.2 Inch Diameter Mile (IDM) Index

Inch-diameter-mile (IDM) is an indicator of pipe area that can receive infiltration. It is estimated as the product of pipe size in inches and length in miles. IDM for a meter catchment is the sum of IDM values for all pipes in the meter catchment. The relationship between the amount of I&I observed, and the size and length of pipes was evaluated using IDM. The volume of I&I per IDM per inch of rainfall was estimated and ranked for each selected rainfall event. Table 3-10, Table 3-11, and Table 3-12 summarize the volume of I&I, inflow, and infiltration per IDM per inch of rainfall.

Table 3-10: Volume of I&I per IDM per Inch of Rainfall

Flow Meter ID	IDM	Incremental Volume I&I (Thousand Gallons) ¹	Volume I&I per IDM per Inch of Rainfall	Rank ²
CE7-1	152	6,000	39	2
CE1-30	142	6,700	47	1
CW1-3	125	3,600	29	4
N3-5	46	1,700	37	3
N1-31	227	6,000	26	5
SEB3-6	72	1,7000	24	6
SEC3-20	243	3,400	14	8
SED1-29	68	1,500	22	7
SWLST-25	179	1,800	10	9

Notes:

1. Rainfall depth based on NOAA 5-year, 6-hour event of 3.31 inches.
2. Basin ranking based on volume generated from 1-inch of rainfall.

Table 3-11: Volume of Inflow per IDM per Inch of Rainfall

Flow Meter ID	IDM	Incremental Volume Inflow (Thousand Gallons) ¹	Volume Inflow per IDM per Inch of Rainfall	Rank ²
CE7-1	152	3,000	20	3
CE1-30	142	3,300	23	1
CW1-3	125	2,400	19	4
N3-5	46	1,000	22	2
N1-31	227	3,700	16	5
SEB3-6	72	1,000	14	6
SEC3-20	243	2,200	9	7
SED1-29	68	500	7	8
SWLST-25	179	1,200	7	9

Notes:

1. Rainfall depth based on NOAA 5-year, 6-hour event of 3.31 inches.
2. Basin ranking based on volume generated from 1-inch of rainfall.

Table 3-12: Volume of Infiltration per IDM per Inch of Rainfall

Flow Meter ID	IDM	Incremental Volume Infiltration (Thousand Gallons) ¹	Volume Infiltration per IDM per Inch of Rainfall	Rank ²
CE7-1	152	3,200	21	2
CE1-30	142	3,400	24	1
CW1-3	125	1,200	10	6
N3-5	46	700	15	3
N1-31	227	2,400	11	4
SEB3-6	72	700	10	5
SEC3-20	243	1,300	5	9
SED1-29	68	500	7	7
SWLST-25	179	1,100	6	8

Notes:

1. Rainfall depth based on NOAA 5-year, 6-hour event of 3.31 inches.
2. Basin ranking based on volume generated from 1-inch of rainfall.

3.3.3.3 Area Index

The volume of I&I is impacted by the surface area of each meter catchment. The surface area of the subcatchments is used to normalize the volume of I&I, inflow, and infiltration to eliminate any biases due to an abnormally large meter catchment. The amount of surface area of each meter catchment represents the upper ceiling volume of surface runoff available to enter the sewer system. I&I volume per acre per inch of rainfall was estimated and ranked for each selected rainfall event. Table 3-13, Table 3-14, and Table 3-15 summarize volume of I&I, inflow, and infiltration per acre per inch of rainfall.

Table 3-13: Volume of I&I per Acre per Inch of Rainfall

Flow Meter ID	Area	Incremental Volume I&I (Thousand Gallons) ¹	Volume I&I per Acre per Inch of Rainfall	Rank ²
CE7-1	591	6,000	10	2
CE1-30	476	6,700	14	1
CW1-3	1351	3,600	3	8
N3-5	221	1,700	8	4
N1-31	659	6,000	9	3
SEB3-6	359	1,700	5	5
SEC3-20	815	3,400	4	6
SED1-29	494	1,500	3	7
SWLST-25	1532	1,800	1	9

Notes:

1. Rainfall depth based on NOAA 5-year, 6-hour event of 3.31 inches.
2. Basin ranking based on volume generated from 1-inch of rainfall.

Table 3-14: Volume of Inflow per Acre per Inch of Rainfall

Flow Meter ID	Area	Incremental Volume Inflow (Thousand Gallons) ¹	Volume Inflow per Acre per Inch of Rainfall	Rank ²
CE7-1	591	3,000	5	3
CE1-30	476	3,000	7	1
CW1-3	1351	2,400	2	7
N3-5	221	1,000	5	4
N1-31	659	3,700	6	2
SEB3-6	359	1,000	3	5
SEC3-20	815	2,200	3	6
SED1-29	494	500	1	8
SWLST-25	1532	1,200	1	9

Notes:

1. Rainfall depth based on NOAA 5-year, 6-hour event of 3.31 inches.
2. Basin ranking based on volume generated from 1-inch of rainfall.

Table 3-15: Volume of Infiltration per Acre per Inch of Rainfall

Flow Meter ID	Area	Incremental Volume Infiltration (Thousand Gallons) ¹	Volume Infiltration per Acre Inch of Rainfall	Rank ²
CE7-1	591	3,200	5	2
CE1-30	476	3,400	7	1
CW1-3	1351	1,200	1	8
N3-5	221	700	3	4
N1-31	659	2,400	4	3
SEB3-6	359	700	2	5
SEC3-20	815	1,300	2	6
SED1-29	494	500	1	7
SWLST-25	1532	1,100	1	9
Notes:				
1. Rainfall depth based on NOAA 5-year, 6-hour event of 3.31 inches.				
2. Basin ranking based on volume generated from 1-inch of rainfall.				

3.3.3.4 Summary of I&I Assessment

Based on the indices of total I&I volume, inflow volume, and infiltration volume, the individual rankings were accumulated and ranked in descending order to produce an overall rank for each meter catchment. These are outlined in Table 3-16 on the following page. The columns “Combined I&I Rankings,” “Combined Inflow Rankings,” and “Combined Infiltration Rankings” are the accumulated rankings of the three indices for evaluation. Multiple I&I analysis indices were evaluated to decrease the sensitivity of the meter catchment I&I rankings to specific meter catchment characteristics. For example, flow meter CE7-1 has individual rankings of 2, 2, and 2 for volume of I&I per inch of rainfall (Table 3-7), volume of I&I per inch of rainfall per IDM (Table 3-10), and volume of I&I per inch of rainfall per acre (Table 3-13), respectively. The sum of these rankings is 6, or the value in column 2; see Table 3-16 sum of I&I Index Rankings. The overall meter catchment total rankings were developed by summing the total I&I, the total inflow, and the total infiltration (as shown Table 3-16 in Overall Total). This allowed for final ranking of each meter catchment.

Table 3-16: Overall Rankings

Flow Meter ID	Sum of I&I Index Rankings	Sum of Inflow Index Rankings	Sum of Infiltration Index Rankings	Overall Total	Overall Ranking
				(I&I + Inflow + Infiltration)	
CE7-1	2+2+2=6	3+3+3=9	2+2+2=6	21	2
CE1-30	1+1+1=3	2+1+1=4	1+1+1=3	10	1
CW1-3	4+4+8=16	4+4+7=15	5+6+8=19	50	5
N3-5	7+3+4=14	7+2+4=13	7+3+4=14	41	4
N1-31	2+5+3=10	1+5+2=8	3+4+3=10	28	3
SEB3-6	7+6+5=18	7+6+5=18	7+5+5=17	53	6
SEC3-20	5+8+6=19	5+7+6=18	4+9+6=19	56	7
SED1-29	9+7+7=23	9+8+8=25	9+7+7=23	71	8
SWLST-25	6+9+9=24	6+9+9=24	6+8+9=23	71	8

4.0 MODEL BUILD

The hydraulic model represents the sanitary sewer system tributary to each wastewater treatment plant: North, Central, and Southeast. Sewer data used to create the model was provided by the City in the form of a GIS dataset.

The model contributing area was divided into 900 subcatchments. The subcatchments represent the areas that contribute flow to a single sanitary sewer system manhole. These subcatchments were then aggregated to flow meter catchments. Meter catchments identify the sewer system area that is tributary to each flow meter. Population data was obtained from population estimates of the 2020 US Census and used to estimate an average population density for the subcatchments. Table 4-1 outlines the distribution of the population across the meter catchments.

Table 4-1: Population Distribution by Meter Catchment

Meter Catchment	Population Estimates
CE1-30	2,872
CE7-1	2,200
CW1-3	3,211
N3-5	870
N1-31	2,858
SEB3-6	1,269
SEC3-20	3,384
SED1-29	1,469
SWLST-25	3,579
Total	21,713

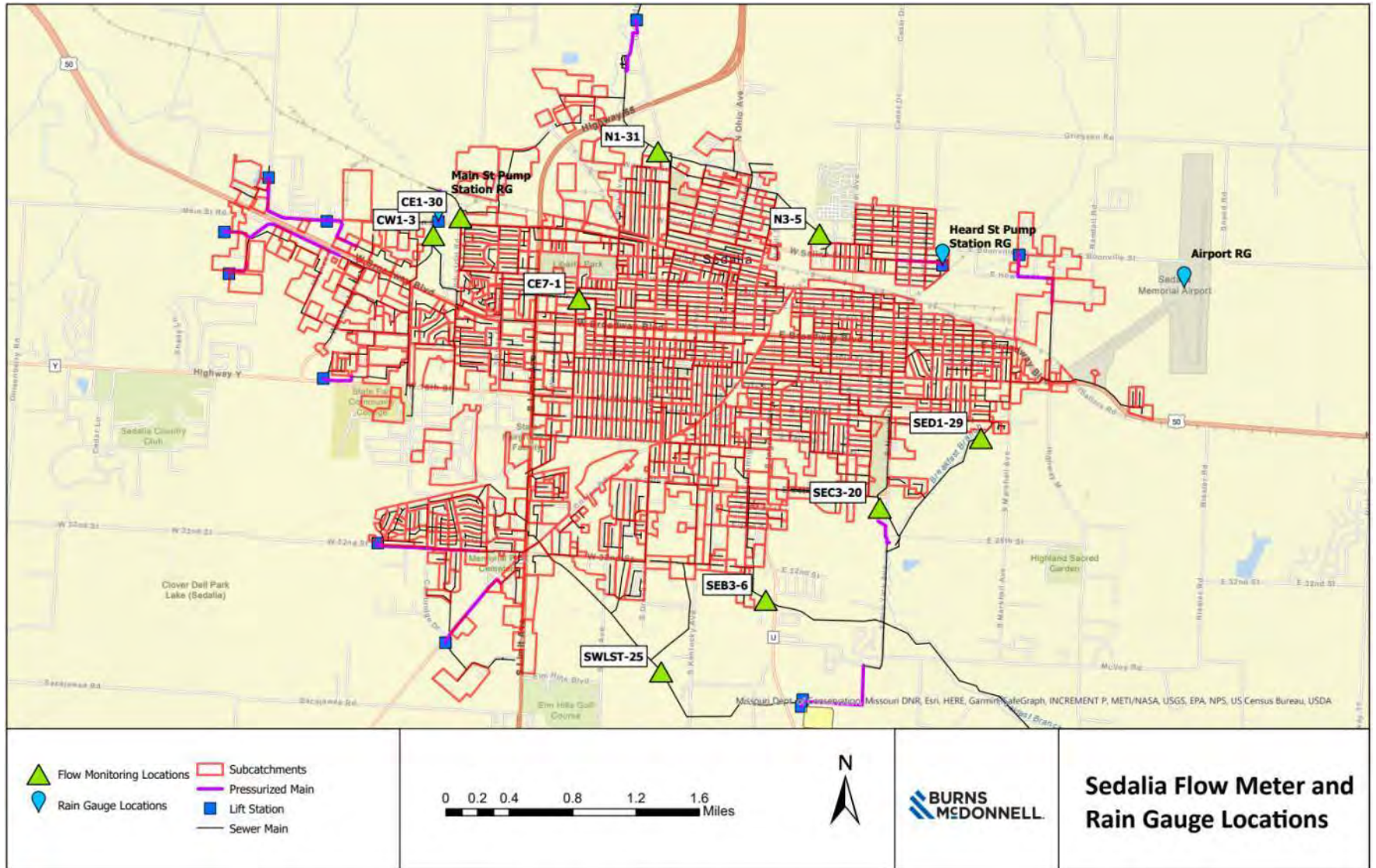
4.1 Model Coverage

The calibrated model encompasses sanitary sewers 4-inches and larger. Sewer data was provided by the City in the form of a GIS dataset. Table 4-2 shows a summary of the model inventory. Modeled pump stations are shown in Figure 4-1.

Table 4-2: Model Inventory

Inventory Item	Quantity – GIS Data	Quantity – Model
No. of Manholes	2,703	2,703
No. of Pipes	2,705	2,713
Total Length of Pipe (LF)	733,000	715,500
Length of 4”-6” Pipe (LF)	3,000	2,700
Length of 8” Pipe (LF)	440,900	429,800
Length of 10”-12” Pipe (LF)	158,400	154,000
Length of 14”-21” Pipe (LF)	75,100	73,900
Length of 24”-27” Pipe (LF)	40,900	40,900
Length of 30”-48” Pipe (LF)	14,700	14,700
Total Length of Force Main (3”-30”) (LF)	29,300	30,200
System Pump Stations	15	15
No. of Subcatchments	-	861
Area of Subcatchments (acres)	4,700	4,700

Figure 4-1: Modeled Pump Stations and Subcatchments



5.0 MODEL CALIBRATION AND VERIFICATION

Regular model calibration is required to demonstrate that the hydraulic model can reasonably replicate the flow for observed rainfall events. Model verification enumerates model accuracy. The calibrated model can then be used for system analysis.

5.1 Dry Weather Calibration

Dry weather flow calibration estimates the ADDF and daily pattern for each meter catchment.

ADDF profiles were estimated for each meter catchment during flow analysis. Each ADDF profile was normalized, the average of the hourly factors was set to one. Those diurnal factors, multiplied by the estimated ADDF are used by the hydraulic model to represent dry weather. The ADDF was estimated as the product of contributing population and flow per capita.

5.1.1 Dry Weather Calibration Process

A percentage error was calculated at each flow meter location by comparing measured flow meter data against hydraulic model predictions. The model was considered calibrated at a meter location if the following calibration criteria was met:

- Model predicted volume was within +/- 10-percent of the meter recorded volume; and
- Model predicted hydrograph visually followed shape and magnitude of the meter recorded hydrograph.

Each meter location was evaluated based on an initial assumption of 100 gallons per capita per day (gpcd). For simulations that were outside the prescribed level of accuracy, the per capita flow was adjusted to achieve the desired level of calibration. Once modified, the model simulation was performed again, and simulation output was compared to the flow monitoring data. This iterative process was repeated until suitable model accuracy was achieved.

5.1.2 Dry Weather Calibration Results

Modeled flows were calibrated at nine (9) flow meter locations for dry weather flow pattern and volume. Due to effects from pump stations and data errors, the model often was unable to accurately predict peak flows. Table 5-1 provides a summary of dry weather flow calibration. Dry weather calibration graphs can be found in Appendix C.

Table 5-1: Dry Weather Calibration Summary

Meter	Volume Difference (absolute values)	
	(%)	(Million Gallons)
CE1-30	3.0%	0.127
CE7-1	2.7%	0.020
CW1-3	1.2%	0.011
N3-5	4.9%	0.005
N1-31	1.5%	0.053
SEB3-6	2.6%	0.009
SEC3-20	4.5%	0.087
SED1-29	2.2%	0.007
SWLST-25	7.8%	0.121

5.2 Wet Weather Calibration and Verification

Wet weather calibration was performed to estimate the hydraulic parameters using field measured data during and after rainfall events.

Calibration was followed by a verification process in which the accuracy of the model was enumerated using independent rainfall events. Verification is used to verify that the model produces reasonable results for non-calibration rainfall events.

5.2.1 Wet Weather Calibration and Verification Process

Hydraulic model results were compared against flow meter recorded peak flow, volume, and overall hydrograph shape. For a range of events, if the calculated percentage error for peak flow and volume was within calibration tolerances (described in later sections) and the overall hydrograph shape was similar, the model was considered calibrated. Otherwise, the inputs and characteristics in the model were adjusted. For simulations that were outside the prescribed level of accuracy, the contributing area, subcatchment width, subcatchment slope, and the hydrologic flow routing values were adjusted.

The model simulation was run, and simulation output was compared to the flow monitoring data again. This iterative process was repeated until further model adjustment becomes impractical.

Once calibrated to wet weather conditions, simulations were performed to verify the model. Using rainfall events which were independent of the calibration events allowed for demonstration that the hydraulic model was verified across multiple rainfall types. This process determined that the hydraulic model is reasonably calibrated and does not favor any specific type of rainfall event. Subject to this, the verification process was similar to the calibration process.

5.2.2 Wet Weather Calibration and Verification Results

Hydraulic model results were compared against recorded values for overall shape, peak flow, volume, and depth at each meter location. The hydraulic model calibration tolerance at each flow meter location for peak flow was -15-percent to +25-percent; for flow volume the calibration tolerance was -10-percent to + 20-percent. The depth is considered calibrated when the simulated depth is within +/- 0.30 feet of the observed depth. Although some percentage differences appear large, the absolute differences in peak flow or volume are small. This can occur when low flow, low depth, or high sensitivity cause a meter to fail calibration tolerances but is not a cause for concern in these circumstances.

Ideally three rain events would be used to calibrate each meter and two events would be used to verify the model. In some cases, hydraulic model calibration to both peak flow rate and volume was not feasible due to the recorded flow meter data exhibiting peak flow abnormalities, such as surcharging or velocity dropouts during the peak of the event, as noted for specific events. In these instances, depth was able to be calibrated. Wet weather calibration graphs illustrating predicted and meter recorded flow and depth can be found in Appendix D. Table 5-2 provides a summary of the wet weather calibration.

Table 5-2: Wet Weather Calibration Summary

Meter	Calibration Event	Peak Flow Difference		Volume Difference		Depth Difference
		(Million Gallons per Day)	(%)	(Million Gallons)	(%)	(ft)
CE1-30 ¹	4/28/2021	-	-	-	-	-0.05
	5/9/2021	-	-	-	-	-0.02
CE7-1	4/23/2021	-0.38	-11.2%	-0.21	-14.7%	-0.23
	4/28/2021	0.20	4.7%	-0.13	-3.3%	-0.17
CW1-3	5/16/2021	-0.34	-7.9%	-0.07	-1.2%	-0.66
	5/20/2021	-0.82	-43.6%	-0.89	-34.9%	-0.12
N3-5	4/28/2021	0.45	32.9%	<0.01	0.5%	0.25
	5/9/2021	<0.01	0.1%	-0.15	-15.4%	0.04
N1-31	6/11/2021	-0.13	-2.2%	-0.13	-9.7%	0.30
	6/25/2021	-0.03	-0.5%	0.06	1.1%	0.34
SEB3-6	6/11/2021	0.34	17.2%	0.08	19.1%	0.10
	6/28/2021	<0.01	0.9%	-0.10	-26.2%	-0.05
SEC3-20 ¹	8/12/2021	-	-	-	-	0.03
SED1-29 ¹	7/12/2021	-	-	-	-	<-0.01
	7/25/2021	-	-	-	-	-0.01
SWLST-25	6/24/2021	0.18	20.5%	0.07	17.1%	-
	7/15/2021	-0.10	-5.1%	-0.29	-17.6%	-
Notes:						
1. Supercritical flow. Calibrated to depth.						

Table 5-3 provides peak and volume results for each verification rainfall event. Wet weather verification graphs comparing predicted, and meter recorded flow can be found in Appendix D.

Table 5-3: Wet Weather Verification Summary

Meter	Verification Event	Peak Flow Difference		Volume Difference		Depth Difference
		(Million Gallons per Day)	(%)	(Million Gallons)	(%)	(ft)
CE1-30 ¹	4/18/2021	-	-	-	-	-0.13
	5/20/2021	-	-	-	-	-0.06
CE7-1	4/8/2021	0.39	17.7%	0.29	39.5%	-0.10
	4/18/2021	-0.53	-22.3%	-0.06	-16.7%	-0.26
CW1-3	4/23/2021	-1.18	-47.1%	-0.31	-34.5%	0.15
	5/9/2021	0.06	1.5%	0.85	52.2%	-5.99
N3-5	4/8/2021	0.01	2.1%	0.03	8.3%	0.05
	4/9/2021	-0.49	-29.3%	0.21	19.7%	-0.03
N1-31	6/20/2021	0.15	8.7%	-0.08	-12.5%	0.17
	6/25/2021	1.07	18.1%	-0.05	-4.3%	0.51
SEB3-6	6/24/2021	1.34	54.1%	0.89	52.0%	0.24
SEC3-20 ¹	9/4/2021	-	-	-	-	0.08
SED1-29 ¹	7/15/2021	-	-	-	-	0.03
SWLST-25	6/25/2021	0.39	18.5%	0.27	14.8%	-
Notes:						
1. Supercritical flow. Calibrated to depth.						

5.3 Model Calibration and Verification Conclusion

Dry weather calibration, wet weather calibration, and wet weather verification have been completed for the hydraulic model using data from the 2021 flow monitoring. During the calibration and verification process, an effort was made to obtain as many events as possible within the peak flow, volume, and depth prescribed calibration tolerances. It was also of importance for the predicted flow hydrographs to match the overall shape and timing of the observed data.

The hydraulic model calibration and verification was found to reasonably reproduce sewer flow and volume results during rainfall events.

6.0 EXISTING CONDITIONS ANALYSIS

The existing conditions analysis consisted of a return period analysis (RPA) and identification of existing issues within the sanitary system for the selected assessment event. The evaluation was performed using the calibrated model that was developed as part of this project, as discussed in Section 5.0.

The RPA evaluated the existing system under various return periods for the assessment storm to predict the projected sanitary sewer pipes and manholes with a reduced level of service. The storms evaluated used the 2-year, 5-year, and 10-year return intervals. Upon review of the RPA and discussions with the City, the 2-year, 6-hour design event was selected as the assessment storm for further analysis.

6.1 Description of Design Storm

Peak flows are largely impacted by the hydrology of the sewer system. The highest peak flows seen in sanitary sewers are typically caused by short, high intensity storm events. These events are typical of the spring and summer seasons and are recommended for sewer sizing and system evaluation purposes. Existing conditions were evaluated using the results of a design storm with those seasonal characteristics. Two main components were considered when preparing the design storm: rainfall distribution and return period.

6.1.1 Selection of Rainfall Distribution

Typical rainfall distribution varies by geographic location. The National Oceanic and Atmospheric Administration (NOAA) Atlas 14 developed standard synthetic hyetographs for different regions of the United States. The City of Sedalia is located within Volume 8, Region 3.

A 6-hour storm duration was selected for the project. With each storm duration, there are four hyetographs provided for each region, for a storm peak intensity in each quartile of the storms' duration. Table 6-1 displays the frequency of storms data within each region and quartile for varying durations.

Table 6-1: Table A.5.1 from NOAA Atlas 14

Table A.5.1. Total number of precipitation cases and number (and percent) of cases in each quartile for selected durations for each climate region: North Plains (1), Western Colorado (2), South Plains (3), and Mississippi Valley (4). Region 4 in this volume includes stations from region 1 of Volume 9.

Duration	Region	All cases	First quartile cases	Second quartile cases	Third quartile cases	Fourth quartile cases
6-hour	1	8,828	3,967 (45%)	2,547 (29%)	1,554 (17%)	760 (9%)
	2	1,300	755 (58%)	271 (21%)	178 (14%)	96 (7%)
	3	8,903	4,232 (48%)	2,619 (29%)	1,392 (16%)	660 (7%)
	4	9,142	3,050 (33%)	2,829 (31%)	2,087 (23%)	1,176 (13%)
12-hour	1	9,010	4,593 (51%)	2,110 (23%)	1,505 (17%)	802 (9%)
	2	1,356	710 (52%)	283 (21%)	215 (16%)	148 (11%)
	3	9,097	5,128 (56%)	1,988 (22%)	1,272 (14%)	709 (8%)
	4	9,631	3,519 (36%)	2,476 (26%)	2,203 (23%)	1,433 (15%)
24-hour	1	8,370	4,170 (50%)	1,765 (21%)	1,378 (16%)	1,057 (13%)
	2	1,025	503 (49%)	206 (20%)	155 (15%)	161 (16%)
	3	8,635	4,503 (52%)	1,527 (18%)	1,466 (17%)	1,139 (13%)
	4	9,325	3,316 (36%)	2,278 (24%)	2,171 (23%)	1,560 (17%)
96-hour	1	8,415	3,990 (47%)	1,551 (18%)	1,389 (17%)	1,485 (18%)
	2	1,134	542 (48%)	228 (20%)	188 (16%)	176 (16%)
	3	8,653	4,055 (47%)	1,720 (20%)	1,463 (17%)	1,415 (16%)
	4	8,908	3,696 (41%)	1,962 (22%)	1,653 (19%)	1,597 (18%)

From the Precipitation Frequency Data Server, regional temporal distribution data are available in a tabular form for a selected location under the "Supplementary information" tab or through the temporal distribution web page (http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_temporal.html). For 6-, 12- and 24-hour durations, temporal distribution data are provided in 0.5-hour increments and for 96-hour duration in hourly increments.

For a 6-hour duration in Region 3, the first-quartile hyetograph represented the majority of the storm events. For this reason, the first-quartile hyetograph was recommended to the City as the easement storm.

A percent exceedance within Region 3, first quartile was also evaluated. NOAA Atlas 14 has developed hyetographs based on 10-percent exceedance probability up to a 90-percent exceedance probability. The 50-percent probability exceedance was used for this study since it represents the median hyetograph (NOAA Atlas 14, Volume 8, Version 2, 2013).

To summarize, the synthetic hyetograph selected for the analysis was the NOAA Atlas 14, Volume 8, Region 3, 6-hour duration, first quartile, 50th-percent exceedance.

6.1.2 Selection of Return Period and Rainfall Depth

The hyetograph described above can be applied for storm events for any return period. NOAA Atlas 14 provides point precipitation depth frequency estimates for the contiguous United States. This data source

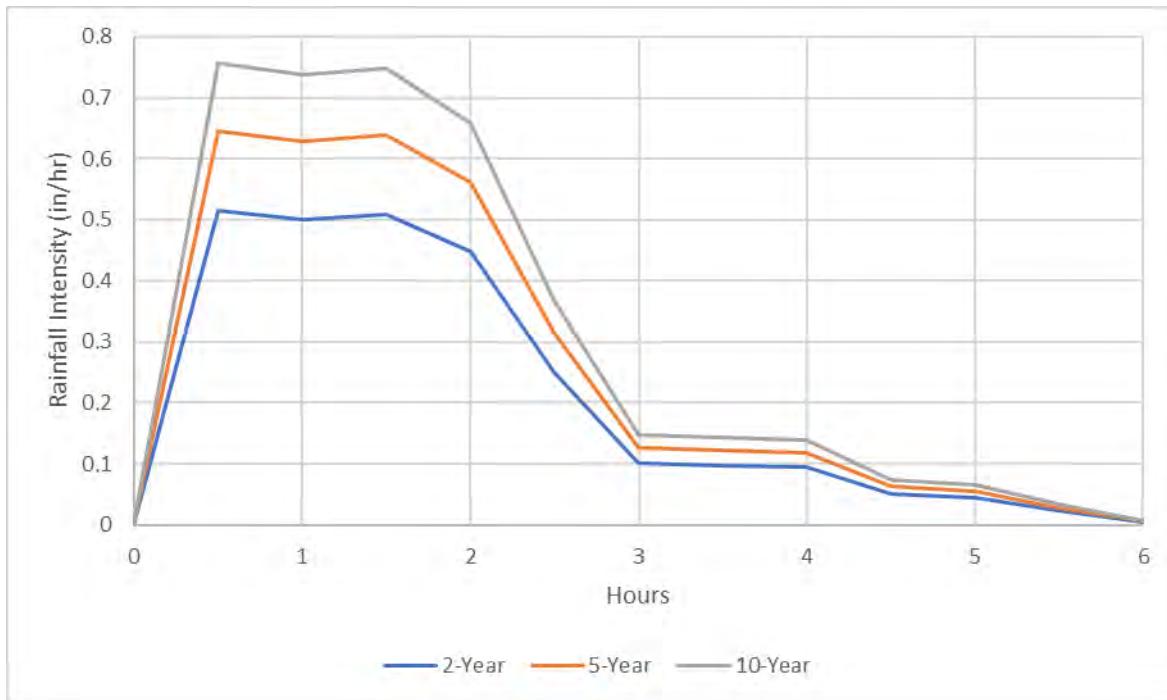
uses storm duration and return period to estimate a total rainfall depth for that storm at any chosen location. The total rainfall depths for the selected return period at Sedalia, Missouri are summarized in Table 6-2.

Table 6-2: Rainfall Depths for 6-Hour Duration

Return Period	Total Rainfall Depth (inches)
2-Year	2.64
5-Year	3.31
10-Year	3.88

The hyetographs for the 2-year, 5-year, and 10-year rainfall events in Sedalia, Missouri based on the NOAA hyetograph are shown in Figure 6-1.

Figure 6-1: Design Storm Hyetograph for Sedalia, Missouri



6.2 Return Period Analysis

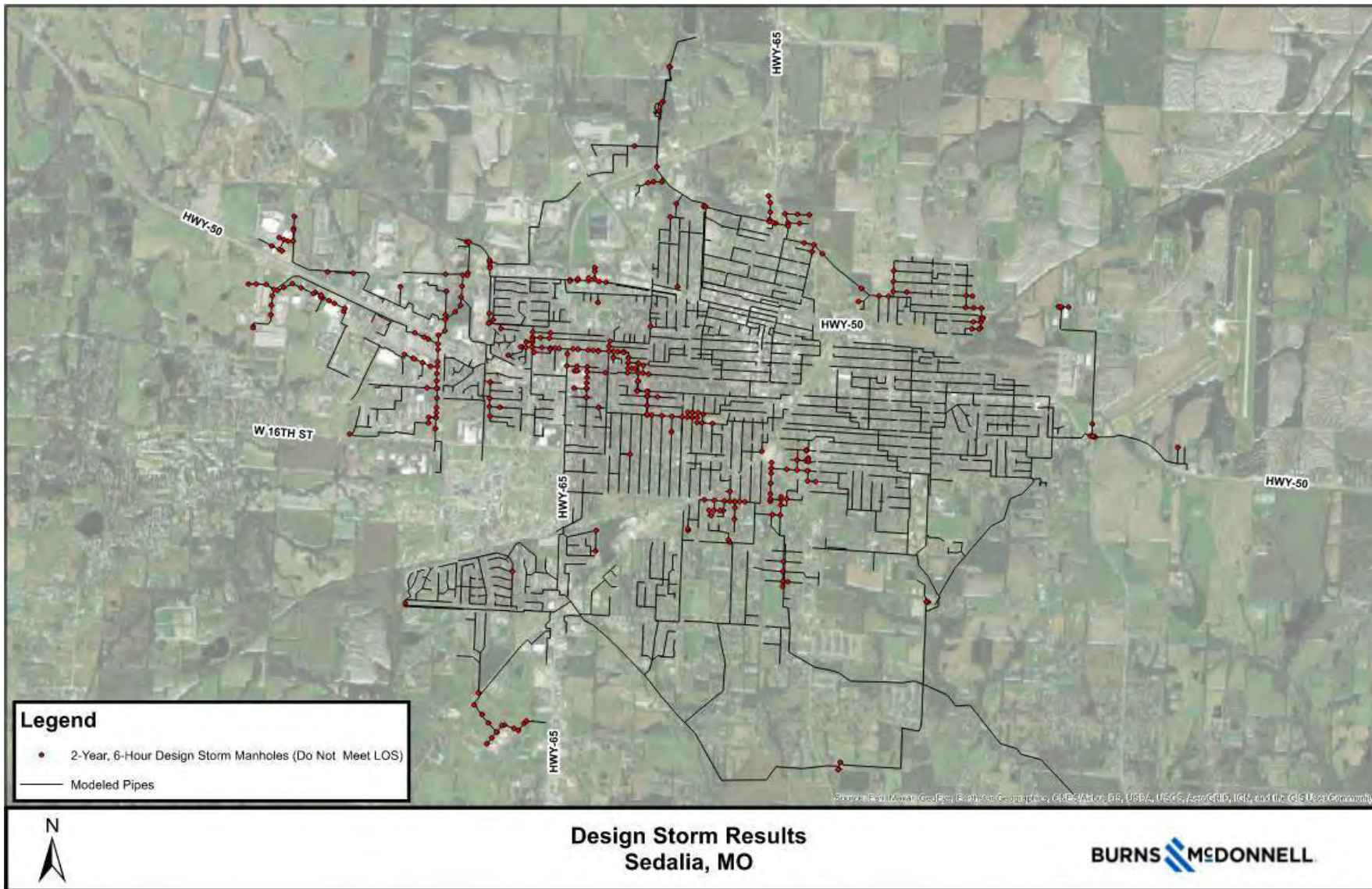
The calibrated sanitary sewer model was used for the RPA. Following industry standards, the analysis evaluated the existing system for the NOAA 2-year, 5-year, and 10-year storm events. The storm event return interval represented the proposed level of service used to analyze the existing sewer system level of risk. The hydraulic model was used to identify areas of manholes and conduits that cannot provide the desired level of service for each of the return intervals (risk levels).

6.2.1 Manholes with Reduced Level of Service

Level of service at the manhole represents the maximum allowable water surface level predicted by the model in the system during the design storm. The freeboard criteria from the maximum water surface level to the manhole rim elevation, was selected to be three (3) feet below rim elevation. The manholes that fail the level of service do not predict a release of sewage at this risk level. However, the model predicts that the manhole is near or at a release of untreated sewage. However, the three (3) feet below rim elevation criteria allows for a factor of safety to minimize the release of untreated sewage.

A manhole would be considered a reduced level of service structure if the model predicted that the maximum water surface elevation is three (3) feet or less below rim elevation. This can occur as a result of deteriorated or broken pipes, equipment failure, or system overload. A reduced level of service largely occurs at manholes that backup due to downstream pipe reaches that are also at a reduced level of service (i.e., downstream bottlenecks). Table 6-2 shows the model predicted manhole level of service for a 2-year event.

Figure 6-2: Existing Conditions Return Period Analysis Manhole Level of Service



In the absence of corroboratory maintenance and/or work order history, it would be prudent to place these on a “Watchlist” to confirm these locations exhibit a reduced level of service.

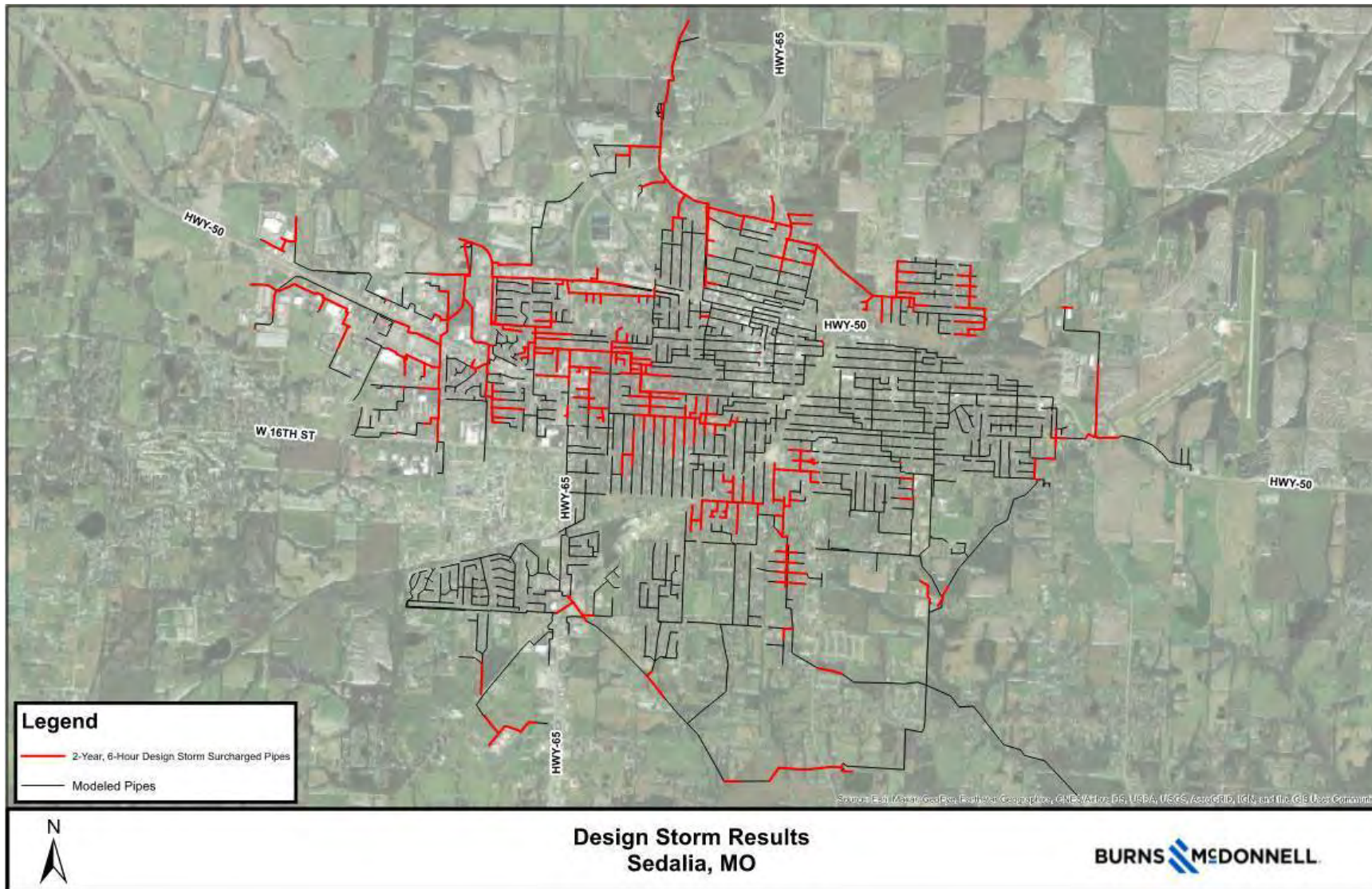
These manholes with a reduced level of service are located upstream form hydraulic restrictions. Future sewer capacity enhancements in these areas would increase the level of service and prevent manhole level of service failures.

6.2.2 Conduits with Reduced Level of Service

A conduit level of service is defined in terms of the maximum instantaneous water depth in relationship to the pipe diameter, the depth to diameter ratio (d/D) for the design storm. The range of potential criteria are from pipe half full to 8-ft below grade, based on hydraulic grade line in the manhole. The conduit level of service selected requires that the water surface elevation not exceed the crown elevation of the pipe.

An adequate level of service for conduits is achieved when the water surface elevation stays below the crown elevation of the pipe. When the water level exceeds the crown of the conduit at any point during the model simulation, the conduit no longer meets level of service criteria and is performing at a reduced level of service. The reduced level of service can be caused by restricted capacity at that pipe or downstream restrictions. Figure 6-3 shows the model predicted conduit level of service.

Figure 6-3: Existing Condition Return Period Analysis Conduit Level of Service



Common causes of a reduced level of service within a sanitary system can be a result of undersized pipes and/or sections of gravity pipes with minimal slopes, flow constrictions, pump station constriction, and areas of excessive infiltration and inflow (I&I).

6.3 Existing System Deficiencies

Based on the return period analysis and discussions with the City, the 2-year, 6-hour event was selected as the design event for this report. This event was the design level of service and used to identify specific deficiencies within the sanitary system. This design event was also used for the development of the Capital Improvement Plan (CIP) as discussed in Section 8.0.

The existing system analysis indicated pump station constrictions and flow constrictions within the sanitary sewer system. The reduced level of service identified in manholes and conduits were a result of these constrictions. The pump stations with constraints identified are summarized in the following CIP sections.

The reduced 2-year level of service at the remaining conduits were caused by under capacity pipes, or downstream pipes that were under capacity. Pipes that are under capacity were a result of I&I, undersized pipe diameters, flat slopes, or negative sloped pipes.

7.0 FUTURE CONDITIONS

A future conditions analysis was performed for the 2-year, 6-hour assessment storm. The system was evaluated for two phases of development in conjunction with the City's comprehensive planning process:

- Phase 1 is based on a projected 5-year development.
- Phase 2 is based on a projected 20-year development.

Similar to the existing system analysis, the system was evaluated under these future conditions to predict which areas would not meet the required level of service in these planning horizons. Two tiers of level of service were evaluated:

- Tier 1 projects were considered to achieve a manhole level of service of 3 feet below rim elevation freeboard.
- Tier 2 projects were aimed at reducing the potential for basement backups; when possible, aimed at maintaining 8' of freeboard below rim.

7.1 Phase 1

The calibrated hydraulic model was used as a basis for the Phase 1 development network. Projected future developments for the next five years developed with and reviewed by City staff, in conjunction with the City's comprehensive planning process, were added to the model along with the projected population. The locations of the specific residential development and manufacturing sites were selected to evaluate the conveyance system. The development, except the specific large water user sites, reflect the average industrial conditions of the existing system, which is typically warehouse and industrial. The Phase 1 projected development is summarized below:

- Residential and Commercial
 - 0.167 people per acre; and
 - 14,000 acres of residential development through the existing and future areas of the collection system
- Industrial
 - 69 gallons per day per acre as a base industrial loading; and
 - Development of 14,000 acres of industrial development through the existing and future areas of the collection system; and
 - 3 anticipated large water users or manufacturing sites (with sewer loading ranging from 0.4 MGD to 0.8 MGD)

7.2 Phase 2

The calibrated hydraulic model was used as a basis for the Phase 2 development network. Projected future developments for the next twenty years developed with and reviewed by City staff, in conjunction with the City's comprehensive planning process, were added to the model along with the projected population. The locations of the specific residential development and manufacturing sites were selected to evaluate the conveyance system. The development, except the specific large water user sites, reflect the average industrial conditions of the existing system, which is typically warehouse and industrial. The Phase 2 projected development is summarized below:

- Residential and Commercial
 - 0.486 people per acre; and
 - 14,000 acres of residential development through the existing and future areas of the collection system
- Industrial
 - 69 gallons per day per acre as a base industrial loading; and
 - Development of 14,000 acres of industrial development through the existing and future areas of the collection system; and
 - 6 anticipated large water users or manufacturing sites (with sewer loading ranging from 0.4 MGD to 0.8 MGD)

8.0 CAPITAL IMPROVEMENT PLAN

Proposed planning level sewer improvements were developed as part of the capital improvement plan (CIP). These improvements addressed manholes and conduits that indicated a reduced level of service during the existing conditions analysis.

The proposed improvements were organized into 24 projects. Numbering of CIP projects is inconsistent due to revisions to the preliminary CIP list of projects.

8.1 Control Measures and Assumptions

The following control measures were considered for the CIP projects to address deficiencies within the existing sanitary system.

- 30-percent I&I Reduction
- Increased Pipe Diameters
- Increased Pipe Slopes
- Parallel Pipes
- Inline Storage
- Equalization Basin
- Increased Pumping Capacities

A target of 30-percent I&I Reduction has been identified. This was reflected in the modeling efforts by reducing the hydrologic contributing area by 30-percent. Simulations were performed to analyze the effects, in terms of runoff volume. The existing and reduced runoff volumes were compared to develop a gallons per day (gpd) of I&I removed.

Gravity lines for replacement were sized for 2/3 pipe full conditions unless documented otherwise. In general, the proposed pipe replacements consisted of increasing pipe diameters. Projects described herein include only the pipe size increases to meet capacity requirements and do not consider increasing pipe diameters downstream to accommodate the proposed larger pipe sizes upstream. Design efforts should include consideration of increasing pipe sizes downstream of improvement if needed.

Proposed increases to pump stations were based on a fixed pumping capacity for planning level purposes. A detailed analysis would be required to estimate head conditions and pump selections, dependent on the final development proposed. All CIPs summarized in the following sections were developed at a planning

level based upon available information at the time of the report. Design efforts should include confirming all assumptions and data included herein.

8.2 Opinion of Probable Costs

Estimates, forecast, projections, and schedules prepared by Burns & McDonnell relating to costs, quantities, demand, or pricing (including, but not limited to, property costs, construction, operations, and maintenance costs, and/or energy or commodity demand and pricing), are opinions based on Burns & McDonnell's experience, qualifications, and judgement. Burns & McDonnell has no control over weather, cost and availability of labor, material and equipment, labor productivity, energy or commodity pricing, demand or usage, population demographics, market conditions, changes in technology, and other economic or political factors affecting such estimates or projections. In addition, Burns & McDonnell has no control over the uncertainty and potential disruptions to the labor and work force and supply chain caused by a regional, national or global outbreak and spread of an infectious disease, such as COVID-19. It should be acknowledged that actual results may vary significantly from the representations and opinions herein, and nothing herein shall be construed as a guarantee or warranty of conclusions, results, or opinions. Burns & McDonnell makes no guarantee or warranty (actual or implied) that costs, schedules, quantities, and related items will not vary from the opinions contained in the estimates, schedules, results, or other statements or opinions prepared by Burns & McDonnell.

8.2.1 Basis of Costs

For unit costs on new sewer construction, regional costs were used based on BMcD experience. Applicable site adjustment factors were applied to adjust the base construction costs for site-specific conditions.

Opinions of probable costs have been developed for each of the planning level CIP projects that are described in the following sections. The total opinion of probable cost for each CIP included a 20% contingency and a 15% estimate for design engineering. An estimate for surveying, geotechnical, etc were not included in the CIP costs. It should be noted that costs are in 2022 dollars. Costs were not inflated for CIP projects based on the recommended phasing year. Detailed costs are available in Appendix E.

8.3 CIP Projects

Twenty-four (24) projects were considered for the CIP. Projects were organized based on location within the overall sewer system and appropriate phasing for existing conditions, Phase 1, and Phase 2 development. The improvements within the Phase 1 and Phase 2 development conditions are development driven and should be considered when development begins to extend beyond the existing system. Figure 8-1 and Figure 8-2 illustrates the proposed CIP projects for Phase 1 and Phase 2, respectively, with the

following sections describing each project in detail. Tier 1 projects are required to meet LOS requirements. Tier 2 projects should be considered to improve hydraulics near proposed CIPs.

Additional field investigations will be required in the area south of 50 HWY between Thompson Blvd and Industrial Rd to determine the appropriate CIP.

Figure 8-1: Existing Conditions and Phase 1 Capital Improvement Plan

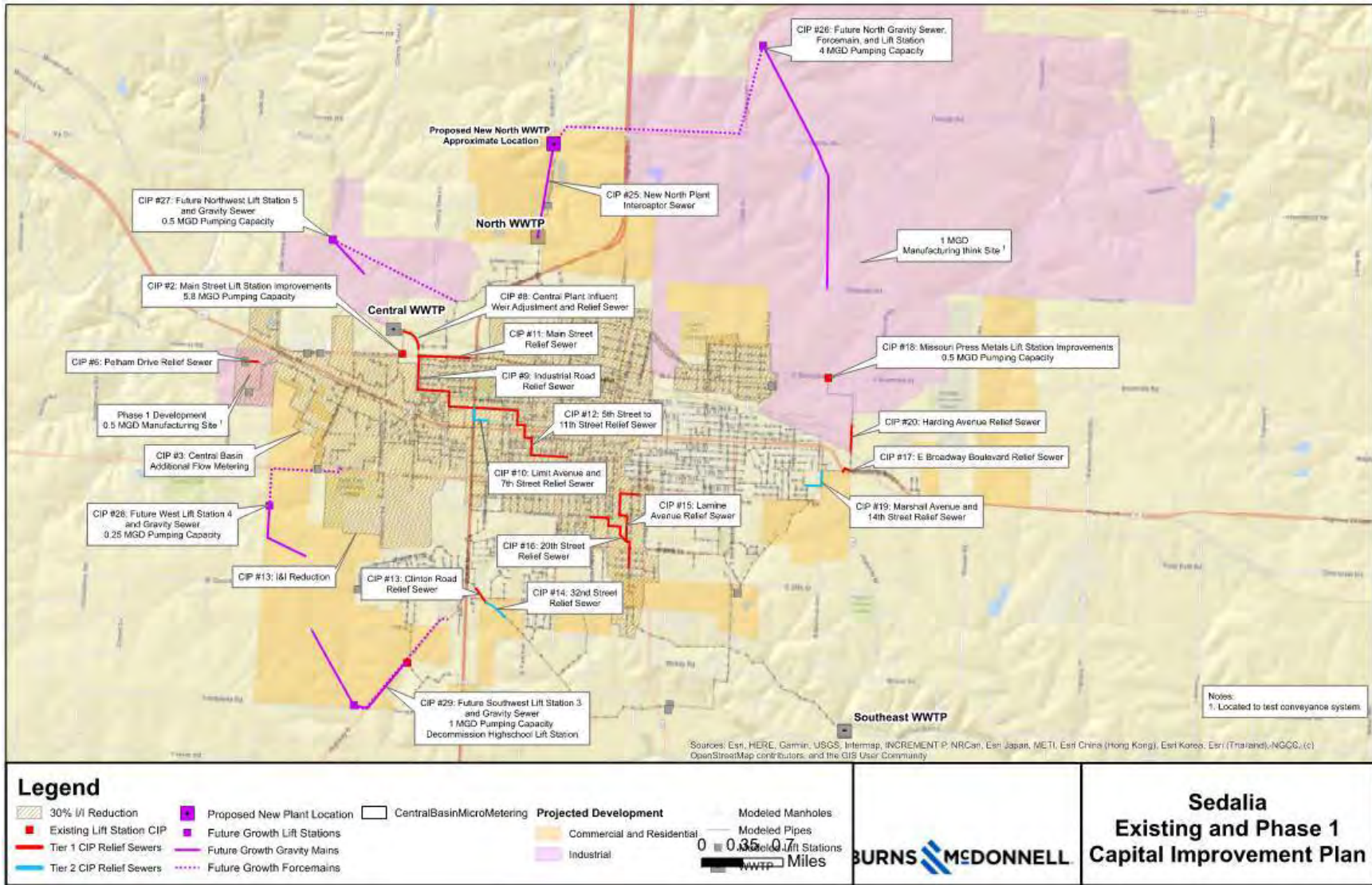
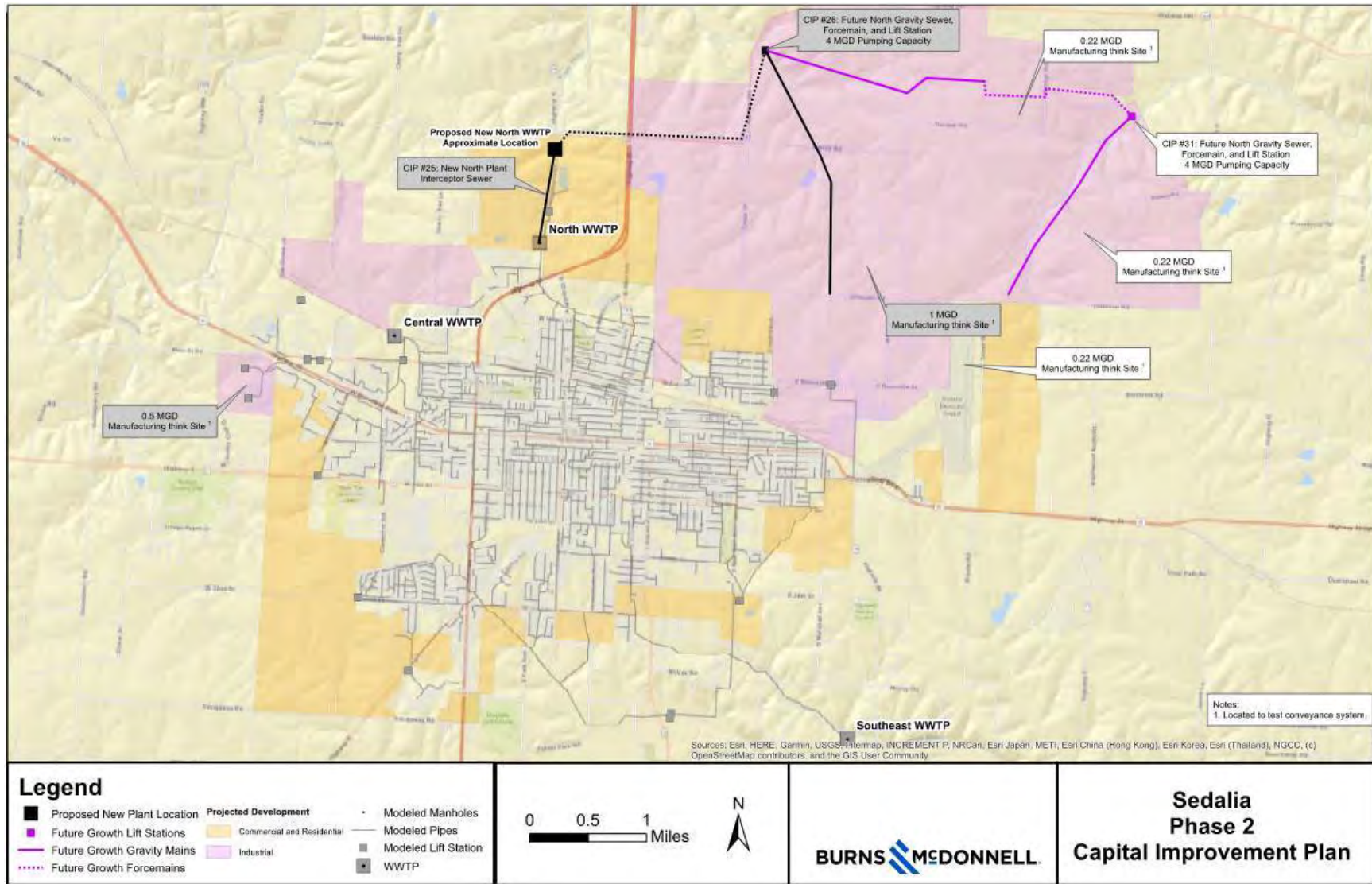


Figure 8-2: Phase 2 Capital Improvement Plan



8.3.1 CIP #1 - I&I Reduction

8.3.1.1 Description

It is recommended that a program of I&I reduction be implemented for six (6) meter basins. CIP #1 is divided into CIP #1A through #1G, see Table 8-1 below.

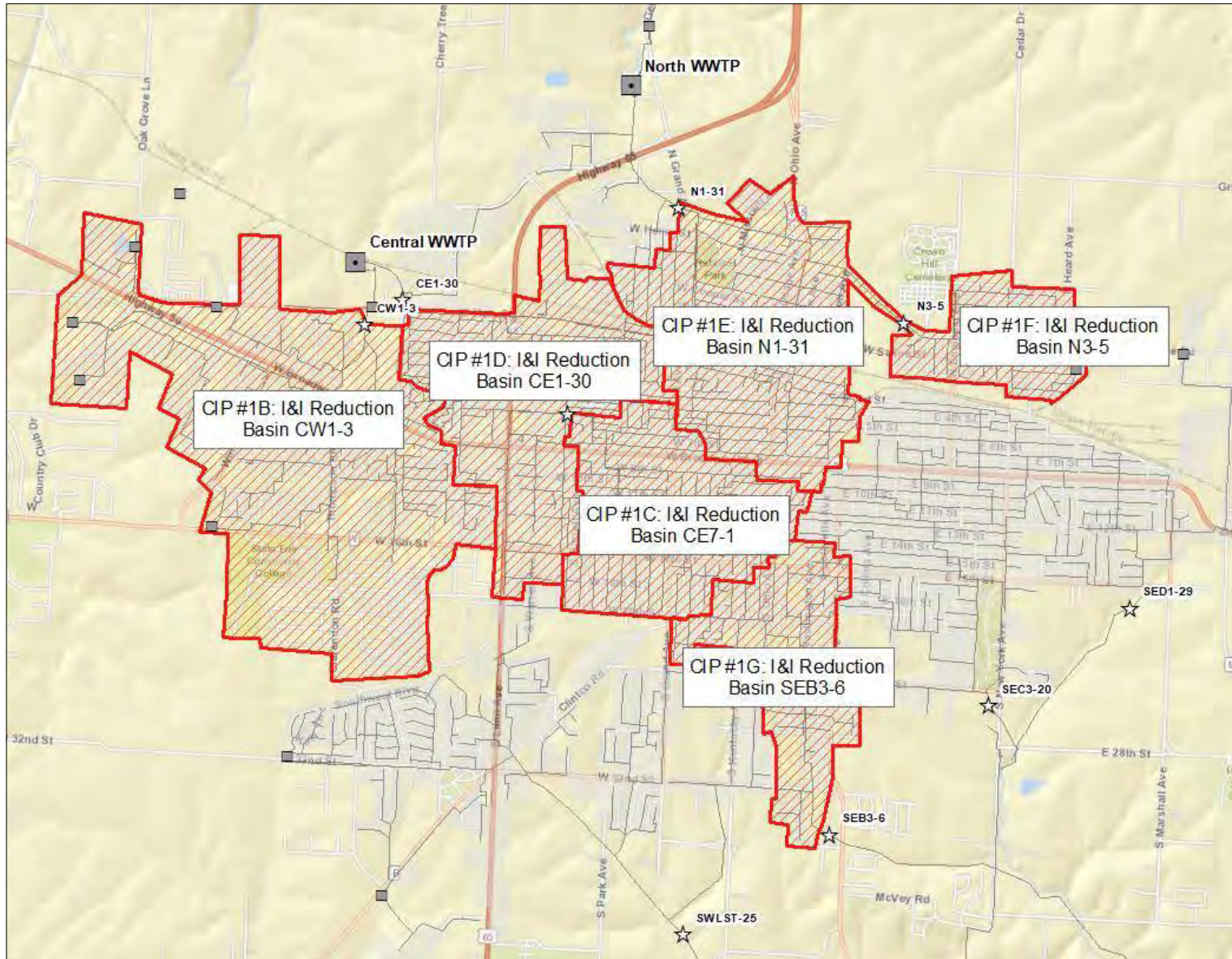
Table 8-1: CIP #1 Summary

CIP #	Description
1A	I&I Assessment and Reduction Program Development
1B	I&I Assessment & Reduction – Basin CW1-3 (Pilot Basin)
1C	I&I Assessment – Basin CE7-1
1D	I&I Assessment – Basin CE1-30
1E	I&I Assessment – Basin N1-31
1F	I&I Assessment – Basin N3-5
1G	I&I Assessment – Basin SEB3-6

As discussed in previous sections, the flow analysis indicated these areas to be one of the higher ranked I&I meter catchments. This was also reflected in the existing conditions analysis, which showed a reduced level of service for conduits and manholes in the area.

The proposed I&I reduction was referenced as the I&I Reduction Project for phasing purposes. The hatched area in Figure 8-3 below shows the modeled area with a target of 30-percent I&I reduction.

Figure 8-3: CIP #1 - I&I Reduction Projects



The model analysis was based on a 30-percent I&I reduction. Once the repairs are complete, post-construction flow monitoring should be conducted to evaluate the level of reduction achieved. Additional investigation and repairs may be needed to achieve the 30-percent I&I reduction as analyzed. CIP project development and phasing documented herein are based on this assumption. Additional costs will be required for post construction flow monitoring.

It is recommended that an I&I reduction project is completed in one flow meter basin, CW1-3, followed by post construction flow monitoring to evaluate if the prescribed I&I reduction goal has been met.

8.3.1.2 Phasing and Opinion of Probable Costs

These projects would be the first to be implemented in order to evaluate if the prescribed 30-percent I&I reduction was achieved.

Table 8-2 below summarizes the opinion of probable costs for an I&I Reduction Program, including I&I Assessment and I&I Reduction for pilot basin CW1-3. CIP #1A includes cost associated with I&I Assessment & Reduction Program Development. The I&I Assessment costs include CCTV of 30% of the small diameter sewers and manhole inspections. I&I Assessment costs do not include costs associated with large diameter CCTV or smoke testing. I&I Reduction costs include targeted CIPP lining and service connection repairs, targeted manhole repairs and an assumed number of point repairs. Estimation of costs associated with removal of possible illicit stormwater connections are not included. The probable cost estimate includes an assumed 15% engineering cost. Only I&I Assessment costs are included for basins CE7-1, CE1-30, N1-31, N3-5 and SEB3-6. Upon completion of the I&I Assessment of these basins, more refined I&I Reduction costs can be established.

Table 8-2: CIP #1 Opinion of Probable Costs

CIP #	Flow Meter Basin	I&I Volume Removed (Million Gallons)	I&I Assessment Costs	I&I Reduction Costs	Total Costs
1A	-	-	-	-	\$250,000
1B	CW1-3	1.7	\$196,000	\$4,025,000	\$4,221,000
1C	CE7-1	1.8	\$161,000	-	\$161,000
1D	CE1-30	3.0	\$196,000	-	\$196,000
1E	N1-31	1.2	\$184,000	-	\$184,000
1F	N3-5	0.6	\$69,000	-	\$69,000
1G	SEB3-6	0.6	\$92,000	-	\$92,000

An I&I reduction plan and program promotes system renewal and reduction in stormwater inflow and infiltration, which decreases the volume of flow treated during wet weather events. The City currently allocates \$150,000 per year toward I&I Reduction efforts. Based on discussions with the City, it is

recommended to continue the I&I reduction program beyond the SSES evaluation completed to date. An ongoing I&I reduction program is recommended.

All following CIP projects are based upon implementation of the prescribed I&I Reduction.

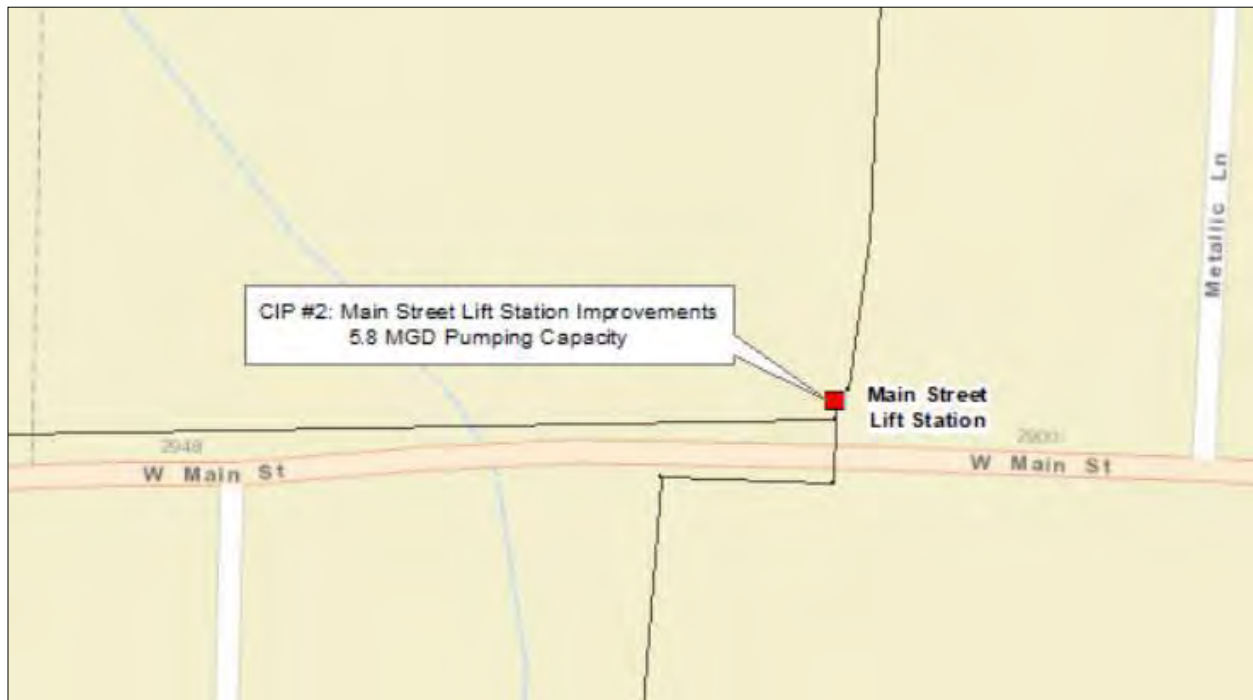
8.3.2 CIP #2 – Main Street Lift Station Improvements

This project is dependent on successful completion of the prescribed I&I Reduction program. This project description and cost are intended to provide an estimate of conveyance requirements.

8.3.2.1 Description

The Main Street Lift Station Improvements project helps address the existing flow constriction located upstream of the Main Street lift station. The model predicted that the Main Street lift station pumping capacity would need to increase from 5.4 MGD to 5.8 MGD. The pump station capacity increase would improve the hydraulics upstream, however CIP #2 alone will not meet necessary level of service requirements upstream based on current model predictions. Additional field investigations will be required in the area south of 50 HWY between Thompson Blvd and Industrial Rd to refine flow distributions and refine capital improvement needs. Figure 8-4 below illustrates the preliminary location of CIP #2.

Figure 8-4: CIP #2 - Main Street Lift Station Improvements



8.3.2.2 Phasing and Opinion of Probable Costs

CIP #2 improvements would help address the reduced level of service upstream of Main Street Lift Station. This project should follow completion of an I&I Reduction program. The total opinion of probable costs, shown below in Table 8-3, for CIP #2 was estimated to be \$964,000.

Table 8-3: CIP #2 Opinion of Probable Costs

Description	Quantity
Pump Station (5.8 MGD)	1
Opinion of Probable Costs	\$964,000

8.3.3 CIP #3 – Central Basin Additional Flow Metering

Additional field investigations will be required in the area south of 50 HWY between Thompson Blvd and Industrial Rd to determine the appropriate CIP.

8.3.3.1 Description

Based on the flow monitoring completed to date, the hydraulic model is predicting hydraulic restrictions along Thompson Boulevard. This model prediction does not align with the City knowledge and basement backup history. Therefore, additional flow monitoring to better refine the model in this basin is proposed.

Figure 8-5: CIP #3 – Central Basin Additional Flow Metering



8.3.3.2 Phasing and Opinion of Probable Costs

CIP #3 includes additional flow monitoring and model updates to better refine CIPs in the Central basin. The total opinion of probable costs, shown below in Table 8-4, for CIP #3 was estimated to be \$46,000.

Table 8-4: CIP #3 Opinion of Probable Costs

Description	Quantity
Additional Flow Monitoring	1
Opinion of Probable Costs	\$46,000

8.3.4 CIP #6 – Pelham Drive Relief Sewer

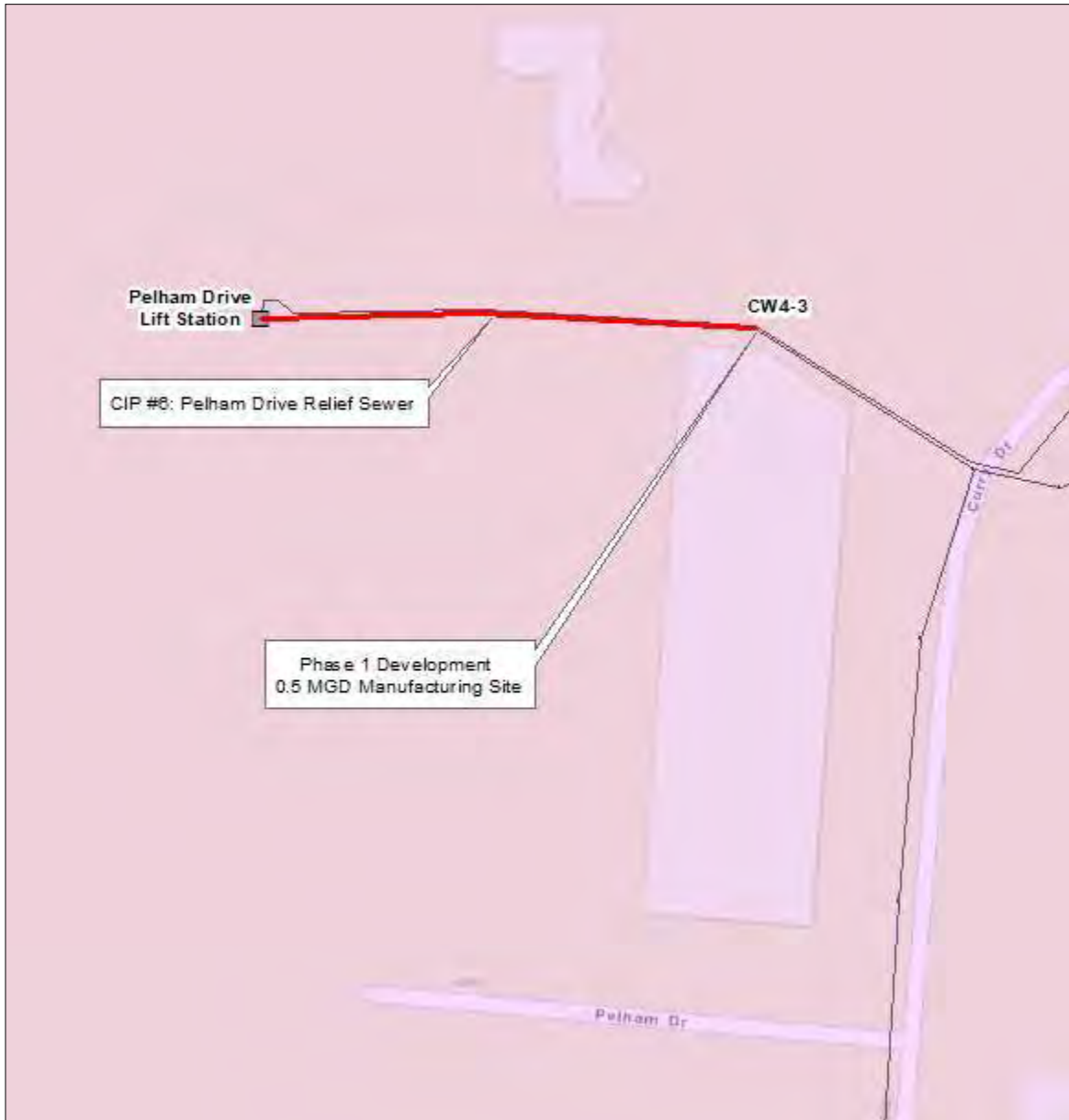
This project is dependent on successful completion of the prescribed I&I Reduction program. This project description and cost are intended to provide an estimate of conveyance requirements.

8.3.4.1 Description

The Pelham Drive Relief Sewer addresses the existing flow constriction located upstream of Pelham lift station. Phase 1 development includes the addition of a large industrial water user (estimated 0.5 MGD) in this area. The Pelham Drive Relief Sewer is necessary to address the existing flow constrictions and to accommodate this Phase 1 development. These flow constrictions are identified in two reaches upstream of the Pelham lift station in the model. This project would increase the pipe diameters to 12-inch between manhole CW4-3 and the Pelham lift station.

This relief sewer project would allow for this area to meet the required level of service and eliminate the current flow constriction. Figure 8-6 below illustrates the preliminary location of CIP #6.

Figure 8-6: CIP #6 - Pelham Drive Relief Sewer



8.3.4.2 Phasing and Opinion of Probable Costs

This project should follow completion of an I&I Reduction program. The total opinion of probable costs for the project was estimated to be \$564,000.

Table 8-5: CIP #6 Opinion of Probable Costs

Description	Quantity
12-inch sewer main	600 LF
Total Opinion of Probable Costs	\$564,000

8.3.5 CIP #8 – Central Plant Relief Sewer & Weir Adjustment

This project is dependent on successful completion of the prescribed I&I Reduction program. This project description and cost are intended to provide an estimate of conveyance requirements.

8.3.5.1 Description

The Central Plant Relief Sewer & Weir Adjustment addresses the existing flow constriction located upstream of the Central Wastewater Treatment Plant (WWTP). These flow constrictions are identified between manhole CE1-20 and the Central WWTP in the model. This project would increase the pipe diameters to 36-inch between manhole CE1-20 and the Central WWTP. This project would also include an adjustment to the influent weir elevation at the Central WWTP; from approximately an elevation 797' to 796'. Proposed weir adjustment has been evaluated for planning level purposes. A detailed analysis would be required to determine feasibility. Figure 8-7 below illustrates the preliminary location of CIP #8.

Figure 8-7: CIP #8 - Central Plant Relief Sewer & Weir Adjustment



8.3.5.2 Phasing and Opinion of Probable Costs

This project should follow completion of an I&I Reduction program. The total opinion of probable costs for the project was estimated to be \$2,082,000.

Table 8-6: CIP #8 Opinion of Probable Costs

Description	Quantity
36-inch sewer main	1,200 LF
Central Plant Influent Weir Adjustment	1 EA
Total Opinion of Probable Costs	\$2,082,000

8.3.6 CIP #9 – Industrial Road Relief Sewer

This project is dependent on successful completion of the prescribed I&I Reduction program. This project description and cost are intended to provide an estimate of conveyance requirements.

8.3.6.1 Description

The Industrial Road Relief Sewer addresses the existing flow constrictions located from the intersection of W 5th Street and S Barrett Avenue downstream to the intersection of W Main Street and Industrial Road. These flow constrictions are identified between manhole CE7-19 and the CE2-1 in the model. This project would increase the pipe diameters to 36-inch between manhole CE7-19 and CE4-18 and 30-inch between manhole CE4-18 and CE2-1. Figure 8-8 below illustrates the preliminary location of CIP #9.

Figure 8-8: CIP #9 - Industrial Road Relief Sewer



8.3.6.2 Phasing and Opinion of Probable Costs

This project should follow completion of an I&I Reduction program. The total opinion of probable costs for the project was estimated to be \$9,718,000.

Table 8-7: CIP #9 Opinion of Probable Costs

Description	Quantity
30-inch sewer main	1,400 LF
36-inch sewer main	4,700 LF
Total Opinion of Probable Costs	\$9,718,000

8.3.7 CIP #10 – Limit Avenue and 7th Street Relief Sewer

This project is dependent on successful completion of the prescribed I&I Reduction program. This project description and cost are intended to provide an estimate of conveyance requirements.

8.3.7.1 Description

The Limit Avenue and 7th Street Relief Sewer improves hydraulic performance between manhole CE5-44 and CE4-18. CE4-18 is a manhole along proposed CIP #9. This project is not necessary to meet required level of service but is aimed at improving system hydraulics and reducing the potential for basement backups. This project would increase the pipe diameters to 15-inch between manhole CE5-44 and CE5-2 and increase the pipe diameters to 18-inch between CE5-2 and CE4-18. Figure 8-9 below illustrates the preliminary location of CIP #10.

Figure 8-9: CIP #10 - Limit Avenue and 7th Street Relief Sewer



8.3.7.2 Phasing and Opinion of Probable Costs

This project should follow completion of an I&I Reduction program. The total opinion of probable costs for the project was estimated to be \$1,415,000.

Table 8-8: CIP #10 Opinion of Probable Costs

Description	Quantity
15-inch sewer main	600 LF
18-inch sewer main	700 LF
Opinion of Probable Costs	\$1,415,000

8.3.8 CIP #11 – Main Street Relief Sewer

This project is dependent on successful completion of the prescribed I&I Reduction program. This project description and cost are intended to provide an estimate of conveyance requirements.

8.3.8.1 Description

The Main Street Relief Sewer addresses the existing flow constrictions located from the intersection of W Main Street and I-65 downstream to the intersection of W Main Street and Industrial Road. These flow constrictions are identified between manhole CE2-60 and the CE2-1 in the model. This project would increase the pipe diameters to 18-inch between manhole CE2-60 and the CE2-1. Figure 8-10 below illustrates the preliminary location of CIP #11.

Figure 8-10: CIP #11 - Main Street Relief Sewer



8.3.8.2 Phasing and Opinion of Probable Costs

This project should follow completion of an I&I Reduction program. The total opinion of probable costs for the project was estimated to be \$2,369,000.

Table 8-9: CIP #11 Opinion of Probable Costs

Description	Quantity
18-inch sewer main	2,300 LF
Opinion of Probable Costs	\$2,369,000

8.3.9 CIP #12 – 5th Street to 11th Street Relief Sewer

This project is dependent on successful completion of the prescribed I&I Reduction program. This project description and cost are intended to provide an estimate of conveyance requirements.

8.3.9.1 Description

The 5th Street to 11th Street Relief Sewer addresses the existing flow constrictions near the intersection of S Harrison Avenue and 11th Street downstream to the intersection of S Barrett Avenue and W 5th Street. These flow constrictions are identified in the model between manholes CE10-5 and CE7-19. This project would increase the following pipe diameters:

- Increase pipe diameters to 21-inches between manholes CE10-5 and CE10-2
- Increase pipe diameters to 24-inches for pipe segments between manholes CE10-2 and CE9-2
- Increase pipe diameters to 30-inches for pipe segments between manholes CE9-2 and CE7-21/CE7-45B. It is important to note that there are already parallel sewer mains between these manholes. The proposed 30-inch pipe diameter is based on the assumption that both parallel sewers will be abandoned.
- Increase pipe diameters to 36-inches for pipe segments between manholes CE7-21/CE7-45B and CE7-19. It is important to note that there are already parallel sewer mains between these manholes. The proposed 36-inch pipe diameter is based on the assumption that both parallel sewers will be abandoned.

This relief sewer project would allow for this area to meet the required level of service and eliminate the current flow constriction. Figure 8-11 below illustrates the preliminary location of CIP #12.

Figure 8-11: CIP #12 - 5th Street to 11th Street Relief Sewer



8.3.9.2 Phasing and Opinion of Probable Costs

This project should follow completion of an I&I Reduction program. The total opinion of probable costs for the project was estimated to be \$6,992,000.

Table 8-10: CIP #12 Opinion of Probable Costs

Description	Quantity
21-inch sewer main	400 LF
24-inch sewer main	3,000 LF
30-inch sewer main	1,400 LF
36-inch sewer main	300 LF
Opinion of Probable Costs	\$6,992,000

8.3.10 CIP #13 – Clinton Road Relief Sewer

This project is not dependent on completion of an I&I Reduction program. This project description and cost are intended to provide an estimate of conveyance requirements.

8.3.10.1 Description

The Clinton Road Relief Sewer addresses the existing flow constrictions located between Clinton Road and W 32nd Street. These flow constrictions are identified between manhole SEA5-23 and SWLST-1 in the model. This project would increase the pipe diameters to 18-inch between manhole SEA5-23 and SWLST-1. This relief sewer project would allow for this area to meet the required level of service and eliminate the current flow constriction. Figure 8-12 below illustrates the preliminary location of CIP #13.

Figure 8-12: CIP #13 - Clinton Road Relief Sewer



8.3.10.2 Phasing and Opinion of Probable Costs

The total opinion of probable costs for the project was estimated to be \$909,000.

Table 8-11: CIP #13 Opinion of Probable Costs

Description	Quantity
18-inch sewer main	800 LF
Opinion of Probable Costs	\$909,000

8.3.11 CIP #14 – 32nd Street Relief Sewer

This project is not dependent on completion of an I&I Reduction program. This project description and cost are intended to provide an estimate of conveyance requirements.

8.3.11.1 Description

The flow constrictions identified in the model are located south of 32nd Street. This project would increase pipe diameters to 21-inches between manholes SWLST-1 and SWLST-3 and increase pipe diameters to 24-inches for pipe segments between manholes SWLST-3 and SWLST-4.

The 32nd Street Relief Sewer improves hydraulic performance south of CIP #13. This project is not necessary to meet required level of service but is aimed at reducing the potential for basement backups. Figure 8-13 below illustrates the preliminary location of CIP #14.

Figure 8-13: CIP #14 - 32nd Street Relief Sewer



8.3.11.2 Phasing and Opinion of Probable Costs

The total opinion of probable costs for the project was estimated to be \$1,346,000.

Table 8-12: CIP #14 Opinion of Probable Costs

Description	Quantity
21-inch sewer main	750 LF
24-inch sewer main	400 LF
Opinion of Probable Costs	\$1,346,000

8.3.12 CIP #15 – Lamine Avenue Relief Sewer

This project is dependent on successful completion of the prescribed I&I Reduction program. This project description and cost are intended to provide an estimate of conveyance requirements.

8.3.12.1 Description

The Lamine Avenue Relief Sewer addresses the existing flow constrictions located from the intersection of E 16th Street and Lafayette Avenue downstream to the intersection of E 24th Street and S Washington Avenue. This project would increase the following pipe diameters:

- Increase pipe diameters to 15-inches between manholes SEB2-31 and SEB2-10
- Increase pipe diameters to 15-inches between manholes SEB1-13 and SEB2-7
- Increase pipe diameters to 18-inches for pipe segments between manholes SEB2-7 and SEB2-10
- Increase pipe diameters to 21-inches for pipe segments between manholes SEB2-10 and SEB2-11

This relief sewer project would allow for this area to meet the required level of service and eliminate the current flow constriction. Figure 8-14 below illustrates the preliminary location of CIP #15.

Figure 8-14: CIP #15 - Lamine Avenue Relief Sewer



8.3.12.2 Phasing and Opinion of Probable Costs

This project should follow completion of an I&I Reduction program. The total opinion of probable costs for the project was estimated to be \$4,796,000.

Table 8-13: CIP #15 Opinion of Probable Costs

Description	Quantity
15-inch sewer main	4,100 LF
18-inch sewer main	450 LF
21-inch sewer main	60 LF
Opinion of Probable Costs	\$4,796,000

8.3.13 CIP #16 – 20th Street Relief Sewer

This project is dependent on successful completion of the prescribed I&I Reduction program. This project description and cost are intended to provide an estimate of conveyance requirements.

8.3.13.1 Description

The 20th Street Relief Sewer addresses the existing flow constrictions located from the intersection of E 20th Street and S Moniteau Avenue downstream to the intersection of E 24th Street and S Washington Avenue. These flow constrictions are identified between manhole SEB4-25 and SEB2-7 in the model. This project would increase the pipe diameters to 15-inch between manhole SEB4-25 and SEB4-6 and increase the pipe diameters to 18-inch between SEB4-6 and SEB2-7. This relief sewer project would allow for this area to meet the required level of service and eliminate the current flow constriction. Figure 8-15 below illustrates the preliminary location of CIP #16.

Figure 8-15: CIP #16 - 20th Street Relief Sewer



8.3.13.2 Phasing and Opinion of Probable Costs

This project should follow completion of an I&I Reduction program. The total opinion of probable costs for the project was estimated to be \$2,703,000.

Table 8-14: CIP #16 Opinion of Probable Costs

Description	Quantity
15-inch sewer main	1,200 LF
18-inch sewer main	1,300 LF
Opinion of Probable Costs	\$2,703,000

8.3.14 CIP #17 – E Broadway Boulevard Relief Sewer

This project is not dependent on successful completion of an I&I Reduction program. This project description and cost are intended to provide an estimate of conveyance requirements.

8.3.14.1 Description

The E Broadway Boulevard Relief Sewer addresses the existing flow constrictions located from the intersection of E Broadway Boulevard and S Harding Avenue. These flow constrictions are identified between manhole SED7-8 and SED7-3 in the model. This project would increase the pipe diameters to 15-inch between manhole SED7-8 and SED7-3. This relief sewer project would allow for this area to meet the

required level of service and eliminate the current flow constriction. Figure 8-16 below illustrates the preliminary location of CIP #17.

Figure 8-16: CIP #17 - E Broadway Boulevard Relief Sewer



8.3.14.2 Phasing and Opinion of Probable Costs

The total opinion of probable costs for the project was estimated to be \$598,000.

Table 8-15: CIP #17 Opinion of Probable Costs

Description	Quantity
15-inch sewer main	580 LF
Opinion of Probable Costs	\$598,000

8.3.15 CIP #18 – Missouri Press Metals Lift Station Improvements

This project is not dependent on successful completion of an I&I Reduction program. This project description and cost are intended to provide an estimate of conveyance requirements.

8.3.15.1 Description

The Missouri Press Metals Lift Station Improvements project addresses the existing flow constriction located upstream of Missouri Press Metals lift station. This project would increase the Missouri Press Metals lift station pumping capacity from 0.16 MGD to 0.5 MGD. The pump station capacity increase

would allow for this area to meet the required level of service and eliminate the current flow constriction. Figure 8-17 below illustrates the preliminary location of CIP #18.

Figure 8-17: CIP #18 - Missouri Press Metals Lift Station Improvements



8.3.15.2 Phasing and Opinion of Probable Costs

The total opinion of probable costs for the project was estimated to be \$1,265,000.

Table 8-16: CIP #18 Opinion of Probable Costs

Description	Quantity
Pump Station (0.5 MGD)	1
Opinion of Probable Costs	\$1,265,000

8.3.16 CIP #19 – Marshall Avenue and E 14th Street Relief Sewer

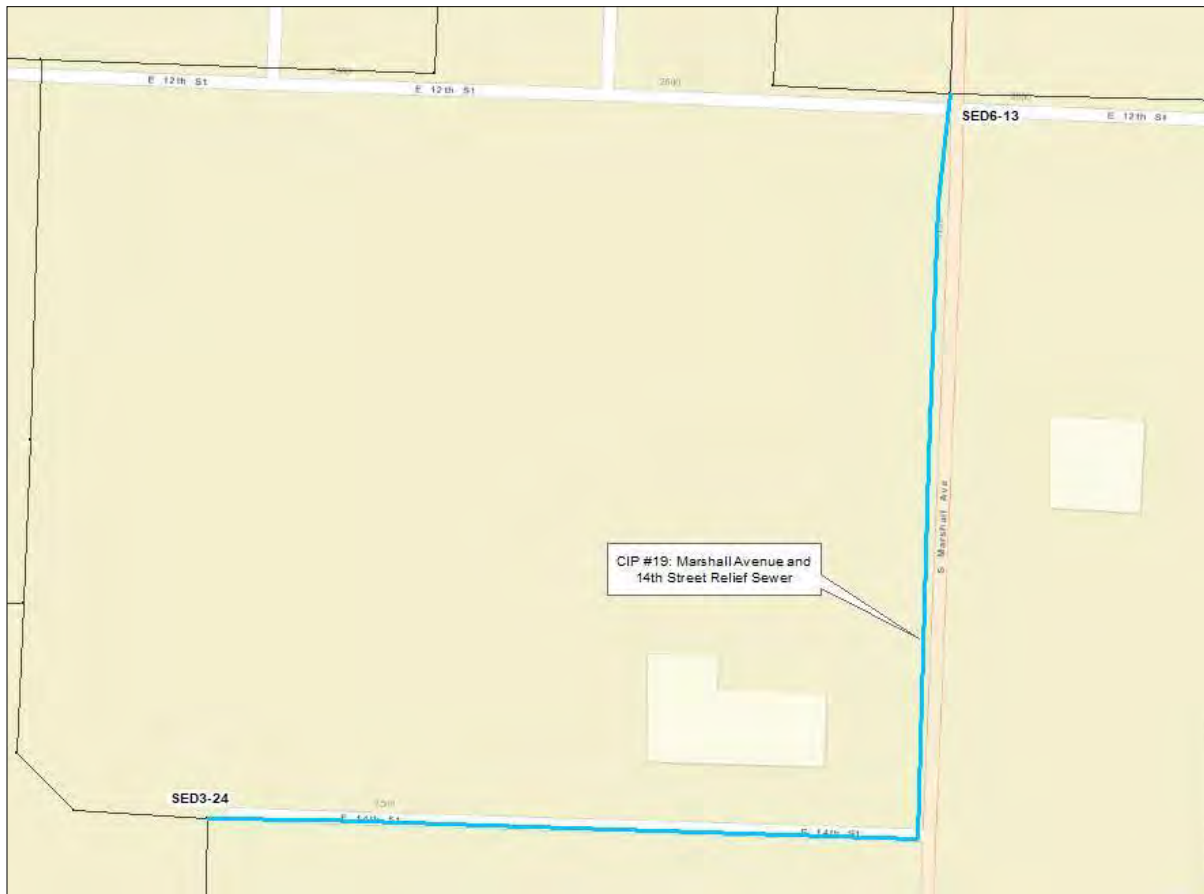
This project is not dependent on successful completion of an I&I Reduction program. This project description and cost are intended to provide an estimate of conveyance requirements.

8.3.16.1 Description

The Marshall Avenue and E 14th Street Relief Sewer addresses the existing flow constrictions located from the intersection of Marshall Avenue and E 14th Street. These flow constrictions are identified between manhole SED6-13 and SED3-24 in the model. This project would increase the pipe diameters to 18-inch between manhole SED6-13 and SED3-24.

The Marshall Avenue and E 14th Street Relief Sewer improves hydraulic performance downstream of CIP #17. This project is not necessary to meet required level of service but is aimed at reducing the potential for basement backups. Figure 8-18 below illustrates the preliminary location of CIP #19.

Figure 8-18: CIP #19 - Marshall Avenue and E 14th Street Relief Sewer



8.3.16.2 Phasing and Opinion of Probable Costs

The total opinion of probable costs for the project was estimated to be \$1,702,000.

Table 8-17: CIP #19 Opinion of Probable Costs

Description	Quantity
18-inch sewer main	1,500 LF
Opinion of Probable Costs	\$1,702,000

8.3.17 CIP #20 – Harding Avenue Relief Sewer

This project is not dependent on successful completion of an I&I Reduction program. This project description and cost are intended to provide an estimate of conveyance requirements.

8.3.17.1 Description

The Harding Avenue Relief Sewer addresses the existing flow constrictions located along Harding Avenue south of E Harvey Street. These flow constrictions are identified between manhole SED7-21 and SED7-18 in the model. This project would increase the pipe diameters to 15-inch between manhole SED7-21 and

SED7-18. This relief sewer project would allow for this area to meet the required level of service and eliminate the current flow constriction. Figure 8-19 below illustrates the preliminary location of CIP #20.

Figure 8-19: CIP #20 - Harding Avenue Relief Sewer



8.3.17.2 Phasing and Opinion of Probable Costs

The total opinion of probable costs for the project was estimated to be \$1,231,000.

Table 8-18: CIP #20 Opinion of Probable Costs

Description	Quantity
15-inch sewer main	1,200 LF
Opinion of Probable Costs	\$1,231,000

8.3.18 CIP #25 – New North Plant Interceptor Sewer

This project is dependent on construction of the new North WWTP. This project description and cost are intended to provide an estimate of conveyance requirements to the new plant.

8.3.18.1 Description

When construction of the new North WWTP is complete, a 36-inch interceptor sewer will be required to convey flow from the existing North WWTP to the new proposed North WWTP. Figure 8-20 below illustrates the preliminary location of CIP #25.

Figure 8-20: CIP #25 - New North Plant Interceptor Sewer



8.3.18.2 Phasing and Opinion of Probable Costs

This project should follow completion of construction of the new North WWTP. The total opinion of probable costs for the project was estimated to be \$6,210,000.

Table 8-19: CIP #25 Opinion of Probable Costs

Description	Quantity
36-inch sewer main	5,000 LF
Opinion of Probable Costs	\$6,210,000

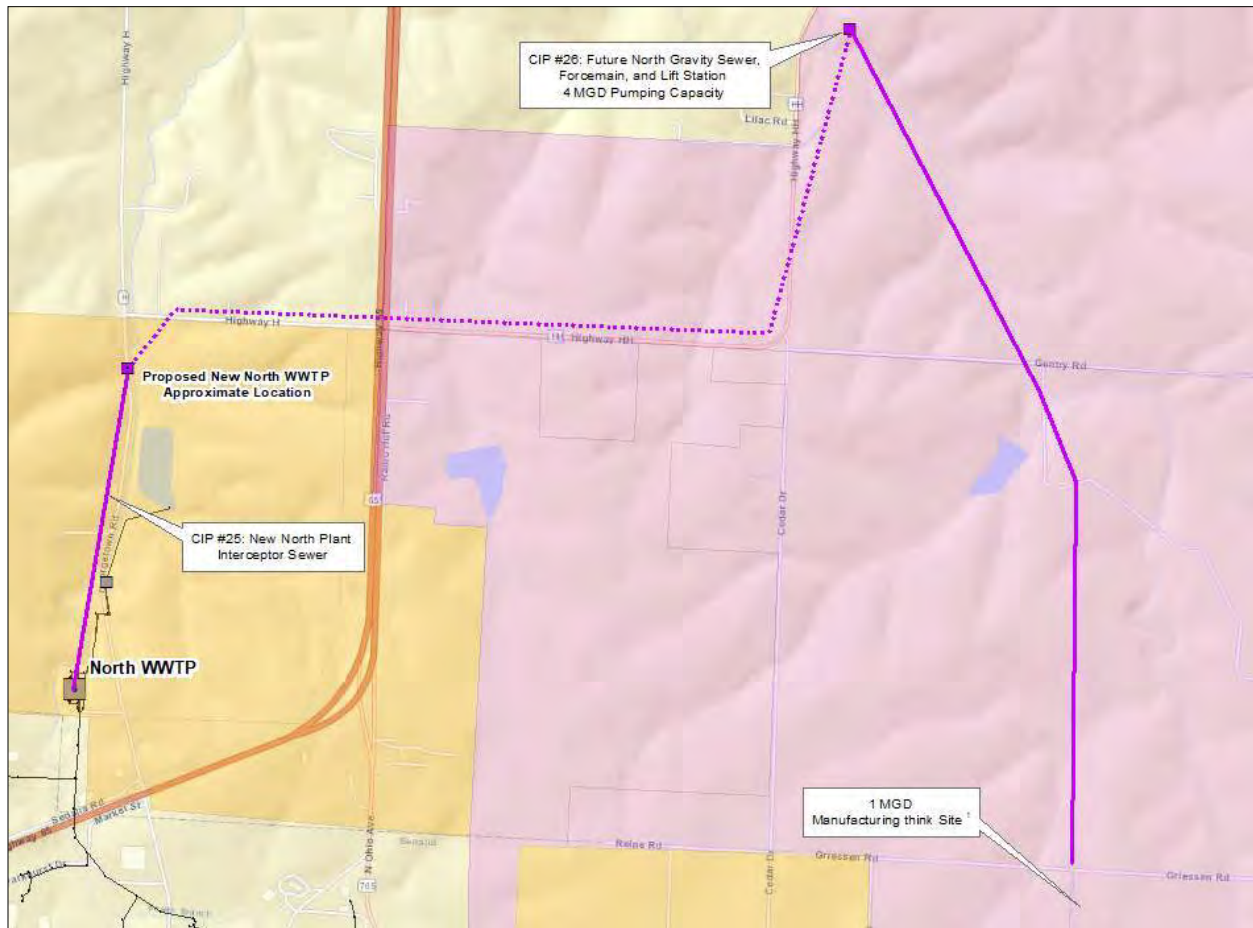
8.3.19 CIP #26 – Future North Lift Station 1 and Gravity Sewer

This project is dependent on Phase 1 development northeast of the City, which is approximated to occur north of the City between Randall Road and Highway 65, see Figure 8-1. When development begins extending in this area a pump station will be required. This project description and cost are intended to provide an estimate of conveyance requirements, actual development requirements should be considered prior to construction.

8.3.19.1 Description

When development begins north of the City between Randall Road and Highway 65 a pump station will be required to service the area. Model results predict that a 4 MGD pump station is required for Phase 1 and Phase 2 development northeast of the City. Modeling results indicated an 18-inch gravity main would be required and a 12-inch force main would be needed to connect to the new North WWTP. Improvements in this area are development driven and should be considered once development starts extending in this area. Location of capital improvements are subject to change as development plans in the area are finalized. Figure 8-21 below illustrates the preliminary location of CIP #26.

Figure 8-21: CIP #26 - Future North Lift Station 1 and Gravity Sewer



8.3.19.2 Phasing and Opinion of Probable Costs

This project is development driven and should be considered once development is expected to connect upstream of the proposed improvements. The total opinion of probable costs for the project was estimated to be \$18,032,000.

Table 8-20: CIP #26 Opinion of Probable Costs

Description	Quantity
18-inch sewer main	12,000 LF
Pump Station (4 MGD)	1
12-inch forcemain	13,000 LF
Opinion of Probable Costs	\$18,032,000

8.3.20 CIP #27 – Future Northwest Lift Station 5 and Gravity Sewer

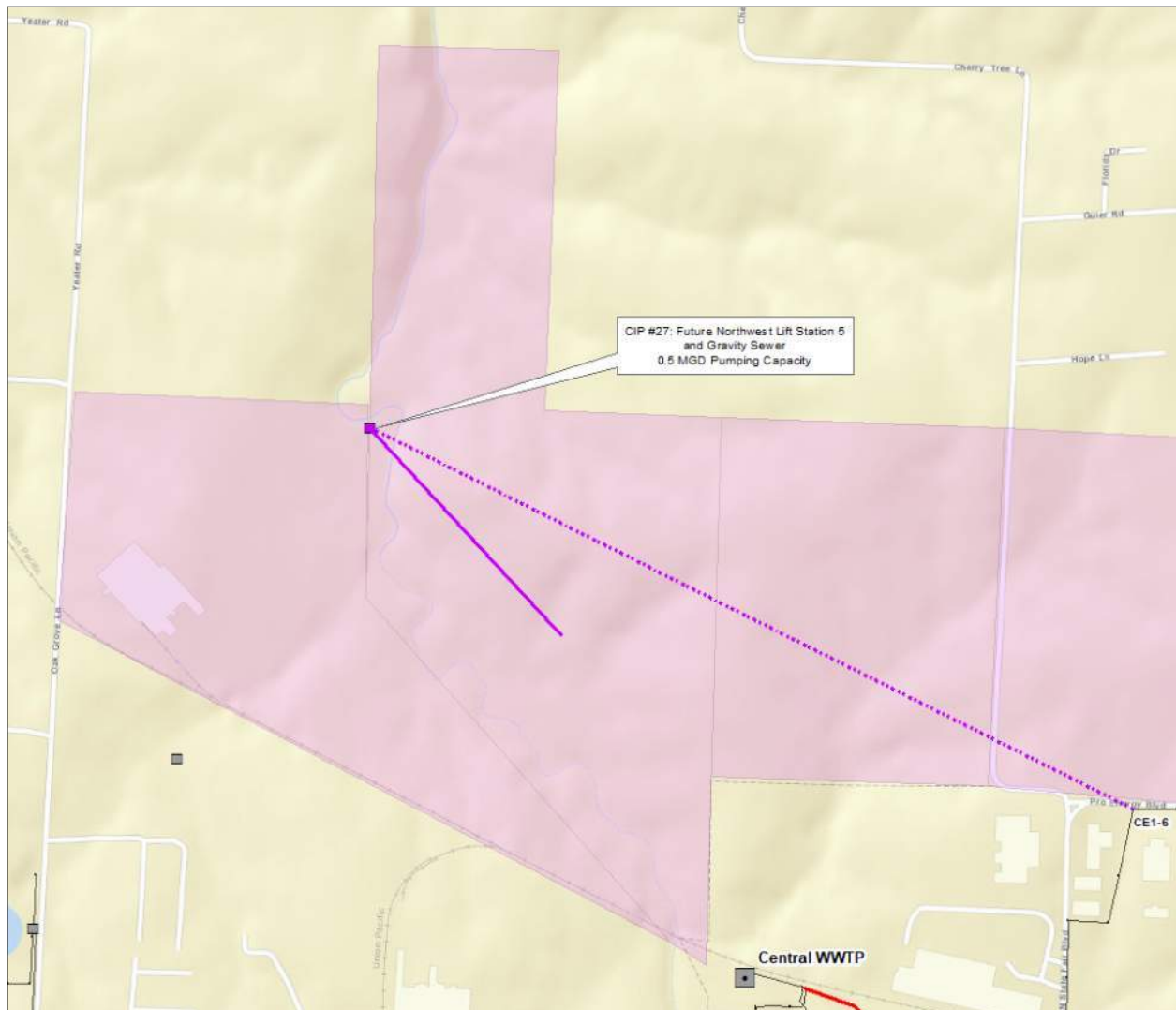
This project is dependent on Phase 1 development north of the existing Central WWTP, see Figure 8-1. When development begins extending in this area a pump station will be required. This project description

and cost are intended to provide an estimate of conveyance requirements, actual development requirements should be considered prior to construction.

8.3.20.1 Description

When development begins north of the of the existing Central WWTP, a pump station will be required to service the area. Model results predict that a 0.5 MGD pump station is required for development of this area. Modeling results indicated an 8-inch gravity main would be required and a 6-inch force main would be needed to connect to the existing collection system. Improvements in this area are development driven and should be considered once development starts extending in this area. Location of capital improvements are subject to change as development plans in the area are finalized. Figure 8-22 below illustrates the preliminary location of CIP #27.

Figure 8-22: CIP #27 - Future Northwest Lift Station 5 and Gravity Sewer



8.3.20.2 Phasing and Opinion of Probable Costs

This project is development driven and should be considered once development is expected to connect upstream of the proposed improvements. The total opinion of probable costs for the project was estimated to be \$1,587,000.

Table 8-21: CIP #27 Opinion of Probable Costs

Description	Quantity
8-inch sewer main	2,600 LF
Pump Station (0.5 MGD)	1
6-inch forcemain	6,300 LF
Opinion of Probable Costs	\$1,587,000

8.3.21 CIP #28 – Future West Lift Station 4 and Gravity Sewer

This project is dependent on Phase 1 development west of town between Highway Y and W 32nd Street, see Figure 8-1. When development begins extending in this area a pump station will be required. This project description and cost are intended to provide an estimate of conveyance requirements, actual development requirements should be considered prior to construction.

8.3.21.1 Description

When development begins west of town between Highway Y and W 32nd Street, a pump station will be required to service the area. Model results predict that a 0.25 MGD pump station is required for development of this area. Modeling results indicated an 8-inch gravity main would be required and a 6-inch force main would be needed to connect to the existing collection system. Improvements in this area are development driven and should be considered once development starts extending in this area. Location of capital improvements are subject to change as development plans in the area are finalized. Figure 8-23 below illustrates the preliminary location of CIP #28.

Figure 8-23: CIP #28 - Future West Lift Station 4 and Gravity Sewer



8.3.21.2 Phasing and Opinion of Probable Costs

This project is development driven and should be considered once development is expected to connect upstream of the proposed improvements. The total opinion of probable costs for the project was estimated to be \$1,553,000.

Table 8-22: CIP #28 Opinion of Probable Costs

Description	Quantity
8-inch sewer main	3,500 LF
Pump Station (0.25 MGD)	1
6-inch forcemain	4,900 LF
Opinion of Probable Costs	\$1,553,000

8.3.22 CIP #29 – Future Southwest Lift Station 3 and Gravity Sewer

This project is dependent on Phase 1 development southwest of town between W 32nd Street and Sacajawea Road, see Figure 8-1. When development begins extending in this area a pump station will be required. This project description and cost are intended to provide an estimate of conveyance requirements, actual development requirements should be considered prior to construction.

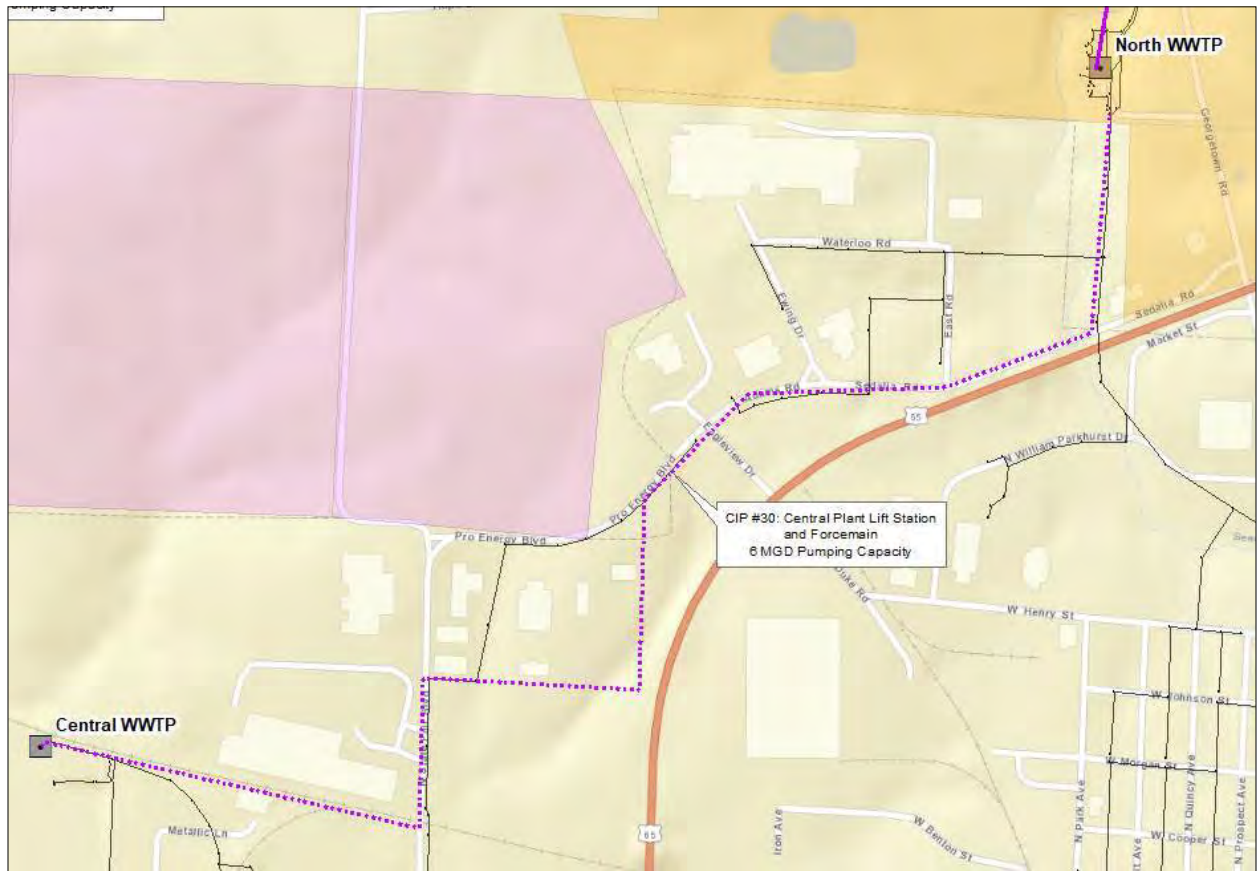
8.3.22.1 Description

When development begins southwest of town between W 32nd Street and Sacajawea Road, a pump station will be required to service the area. It is recommended that the Highschool Drive Lift Station be decommissioned at this time. A new 10-inch gravity main would be required to convey Highschool Drive Lift Station flows to the new Southwest Lift Station 3. Model results predict that a 1 MGD pump station is required for development of this area. Modeling results indicated a 10-inch gravity main would be required and a 6-inch force main would be needed to connect to the existing collection system. Improvements in this area are development driven and should be considered once development starts extending in this area. Location of capital improvements are subject to change as development plans in the area are finalized. Figure 8-24 below illustrates the preliminary location of CIP #29.

8.3.23.1 Description

When Central WWTP effluent limitations or development require that the Central WWTP be decommissioned, a pump station will be required to convey current Central WWTP flows to the new North WWTP. Model results predict that a 6 MGD pump station is required. Modeling results indicated a 12-inch force main would be needed to connect to the new North WWTP. Figure 8-25 below illustrates the preliminary location of CIP #30.

Figure 8-25: CIP #30 - Future Central Plant Lift Station and Gravity Sewer



8.3.23.2 Phasing and Opinion of Probable Costs

This project should be considered when the Central WWTP will be decommissioned. The total opinion of probable costs for the project was estimated to be \$3,496,000.

Table 8-24: CIP #30 Opinion of Probable Costs

Description	Quantity
Pump Station (6 MGD)	1
12-inch forcemain	9,500 LF
Opinion of Probable Costs	\$3,496,000

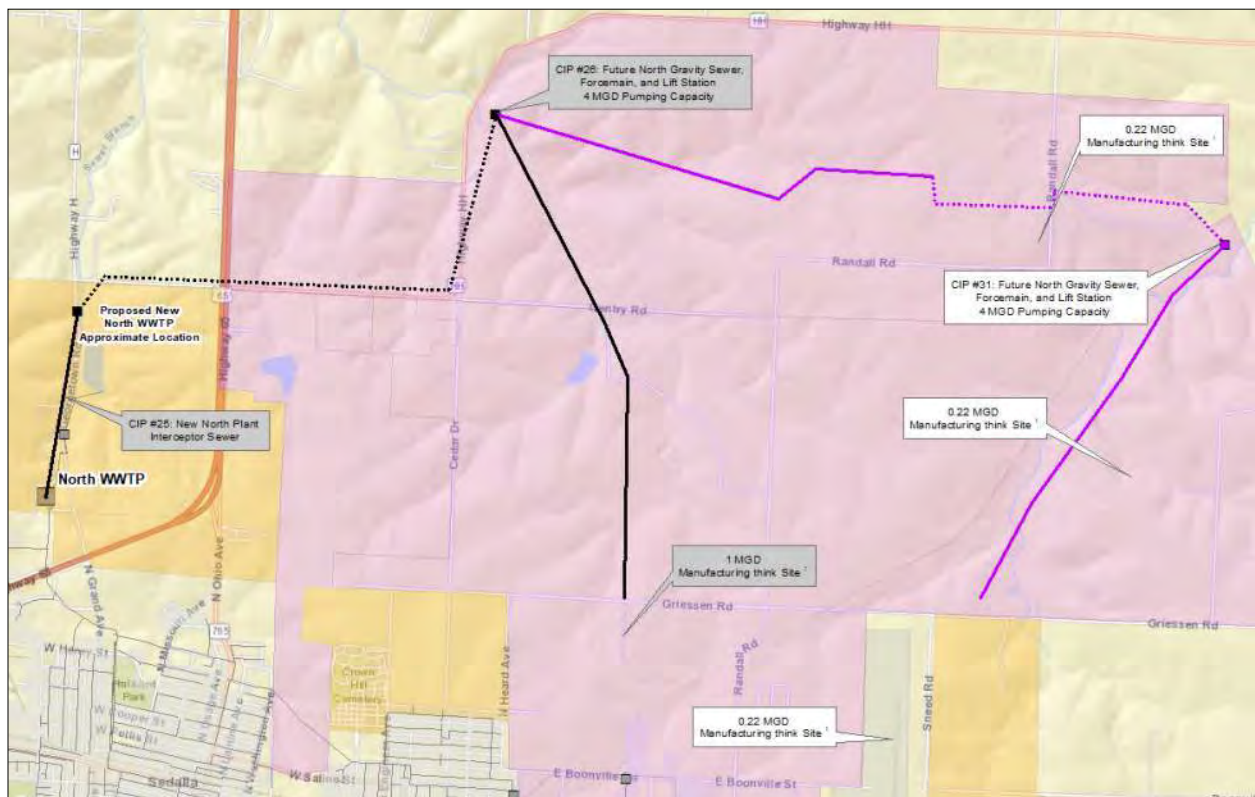
8.3.24 CIP #31 – Future North Lift Station 2 and Gravity Sewer

This project is dependent on Phase 2 development northeast of the city, which is approximated to occur northeast of the City between Randall Road and Salem Road, see Figure 8-2. When development begins extending in this area a pump station will be required. This project description and cost are intended to provide an estimate of conveyance requirements, actual development requirements should be considered prior to construction.

8.3.24.1 Description

When development begins north of the City between Randall Road and Salem Road a pump station will be required to service the area. Model results predict that a 4 MGD pump station is required for Phase 2 development. Figure 8-26 below illustrates the preliminary location of CIP #31. Modeling results indicated an 18-inch gravity main and 21-inch gravity main would be required and a 12-inch force main would be needed to connect to the Future North Lift Station (CIP #26). Improvements in this area are development driven and should be considered once development starts extending in this area. Location of capital improvements are subject to change as development plans in the area are finalized.

Figure 8-26: CIP #31 - Future North Lift Station 2 and Gravity Sewer



8.3.24.2 Phasing and Opinion of Probable Costs

This project is development driven and should be considered once development is expected to connect upstream of the proposed improvements. The total opinion of probable costs for the project was estimated to be \$12,029,000.

Table 8-25: CIP #31 Opinion of Probable Costs

Description	Quantity
18-inch sewer main	5,300 LF
21-inch sewer main	4,500 LF
Pump Station (4 MGD)	1
12-inch forcemain	7,900 LF
Opinion of Probable Costs	\$12,029,000

9.0 CONCLUSIONS AND RECOMMENDATIONS

In conclusion, a hydraulic model was developed to encompass all sanitary sewer 4-inches and larger in diameter. To reflect the current infrastructure condition and flows, hydraulic model calibration and verification was completed. The hydraulic model calibration and verification was found to reproduce satisfactory sewer flow and volume results for a variety of rainfall events. Therefore, the model was utilized for the evaluation of the existing system.

Overall modeling results of the existing system predicted locations at a reduced level of service for the 2-year, 6-hour event. Modeling indicated the main causes of reduced level of service were the high I&I contributing to Meter Catchment CW1-3, CE7-1, CE1-30, N1-31, N3-5, and SEB3-6.

CIP projects were then developed based on the hydraulic model. Project phasing and opinion of probable costs were completed for each of the 25 CIPs. Refer to Table 9-1 below for summary.

Table 9-1: CIP Project Summary

Phasing	CIP #	CIP Project	Opinion of Probable Construction Costs ²	Opinion of Probable Engineering Costs ³	Opinion of Probable Costs
Existing	1A	I&I Assessment & Reduction Program Development	-	\$250,000	\$250,000
	1B	I&I Assessment & Reduction - Basin CW1-3	\$3,670,000	\$551,000	\$4,221,000
	1C	I&I Assessment - Basin CE7-1	\$140,000	\$21,000	\$161,000
	1D	I&I Assessment - Basin CE1-30	\$170,000	\$26,000	\$196,000
	1E	I&I Assessment - Basin N1-31	\$160,000	\$24,000	\$184,000
	1F	I&I Assessment - Basin N3-5	\$60,000	\$9,000	\$69,000
	1G	I&I Assessment - Basin SEB3-6	\$80,000	\$12,000	\$92,000
	2	Main Street Lift Station Improvements	\$838,000	\$126,000	\$964,000
	3	Central Basin Additional Flow Metering	\$40,000	\$6,000	\$46,000
	6	Pelham Drive Relief Sewer	\$490,000	\$74,000	\$564,000
	8	Central Plant Relief Sewer & Weir Adjustment	\$1,810,000	\$272,000	\$2,082,000
	9	Industrial Road Relief Sewer	\$8,450,000	\$1,268,000	\$9,718,000
	10	Limit Ave and 7th Street Relief Sewer	\$1,230,000	\$185,000	\$1,415,000
	11	Main Street Relief Sewer	\$2,060,000	\$309,000	\$2,369,000
	12	5th Street to 11th Street Relief Sewer	\$6,080,000	\$912,000	\$6,992,000
	13	Clinton Road Relief Sewer	\$790,000	\$119,000	\$909,000
	14	32nd Street Relief Sewer	\$1,170,000	\$176,000	\$1,346,000
	15	Lamine Avenue Relief Sewer	\$4,170,000	\$626,000	\$4,796,000
	16	20th Street Relief Sewer	\$2,350,000	\$353,000	\$2,703,000
	17	E Broadway Boulevard Relief Sewer	\$520,000	\$78,000	\$598,000
	18	Missouri Press Metals Lift Station Improvements	\$1,100,000	\$165,000	\$1,265,000
	19	Marshall Avenue and E 14th Street Relief Sewer	\$1,480,000	\$222,000	\$1,702,000
	20	Harding Avenue Relief Sewer	\$1,070,000	\$161,000	\$1,231,000
	25	New North Plant Interceptor Sewer	\$5,400,000	\$810,000	\$6,210,000
	26	Future North Lift Station 1 and Gravity Sewer	\$15,680,000	\$2,352,000	\$18,032,000
	Phase 1	27	Future Northwest Lift Station 5 and Gravity Sewer	\$1,380,000	\$207,000
28		Future West Lift Station 4 and Gravity Sewer	\$1,350,000	\$203,000	\$1,553,000
29		Future Southwest Lift Station 3 and Gravity Sewer	\$2,010,000	\$302,000	\$2,312,000
Phase 2	30	Future North Lift Station 2 and Gravity Sewer	\$3,040,000	\$456,000	\$3,496,000
Future Phase	31	Future Central Plant Lift Station and Forcemain	\$10,460,000	\$1,569,000	\$12,029,000
1. Opinion of Probable Costs were developed based on 2022 unit costs.					
2. Opinion of Probable Construction Costs include a 20% contingency.					
3. Opinion of Probable Engineering Costs are assumed at 15% of Opinion of Probable Construction Costs.					

It is recommended that the City move forward with the target of 30-percent I&I reduction in meter catchments CW1-3, CE7-1, CE1-30, N1-31, N3-5, and SEB3-6.

An I&I reduction plan and program promotes system renewal and reduction in I&I, which improves system reliability and improves level of service. Based on discussions with the city, it is recommended to continue an I&I reduction program beyond the SSES evaluation completed to date. An ongoing I&I reduction program is recommended.

The remaining CIP projects should be pursued as I&I Reduction is completed and as development occurs. The model was used to evaluate the existing conveyance system under Phase 1 and Phase 2 development conditions. Actual development should be considered prior to the start of the development driven projects.

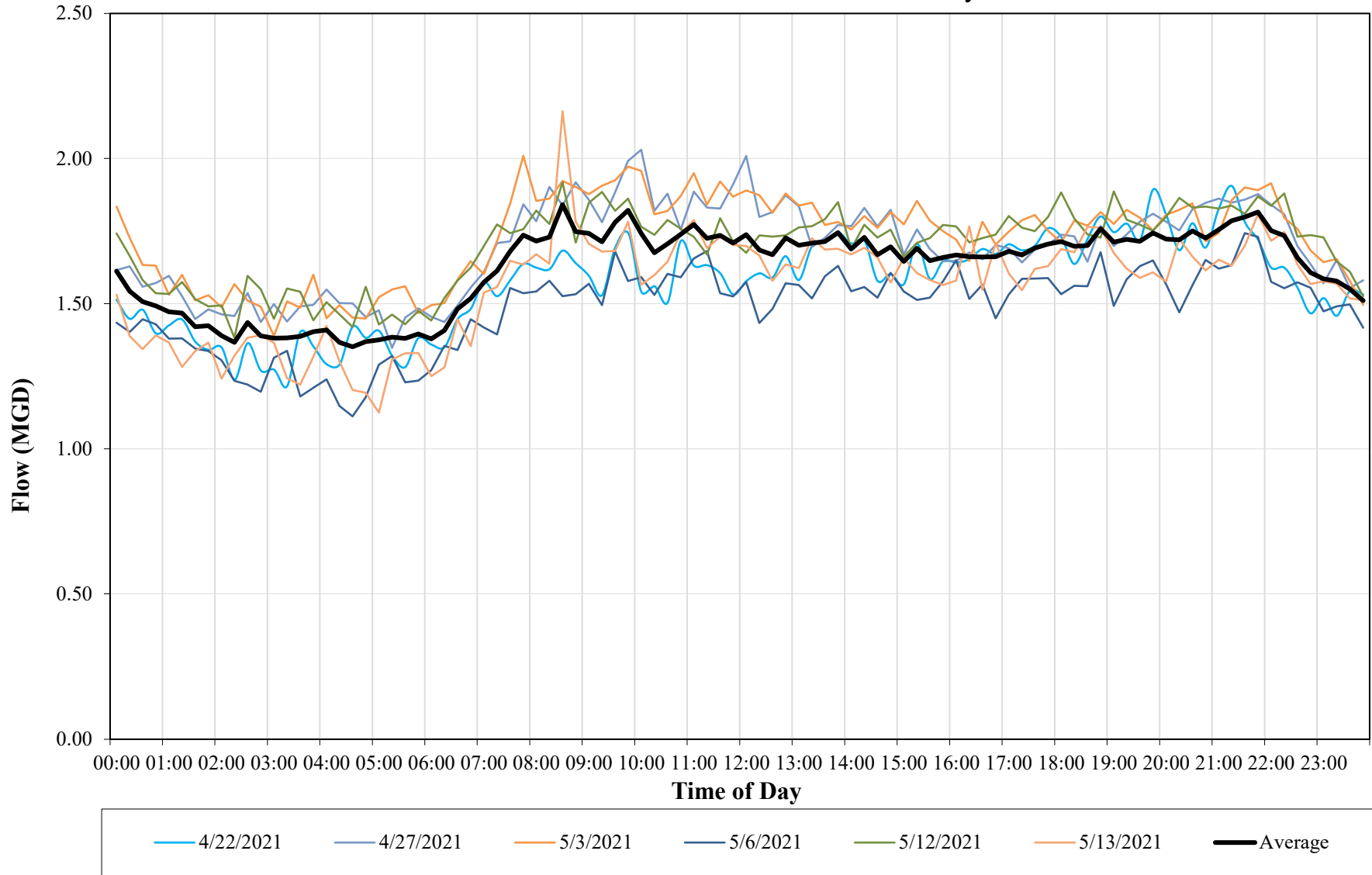
10.0 REFERENCES

United States Department of Commerce, *National Oceanic and Atmospheric Administration (NOAA) Atlas 14, Precipitation-Frequency Atlas of the United States, Volume 8 Version 2.0: Midwestern States*, 2013, http://www.nws.noaa.gov/oh/hdsc/PF_documents/Atlas14_Volume8.pdf.

APPENDIX A – DIURNAL PATTERNS

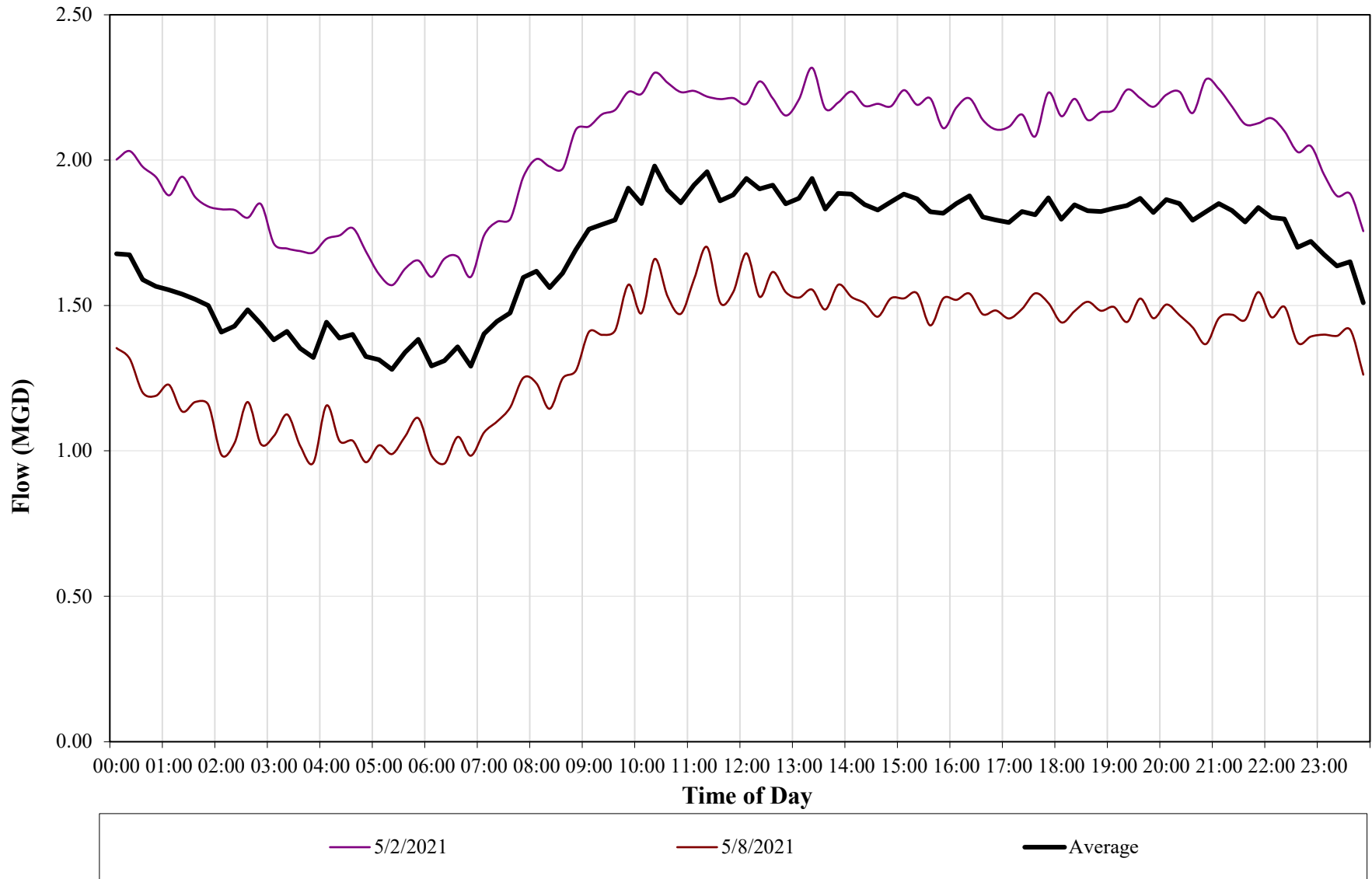
CE1-30 = FM05

ADDF DIURNAL CURVES Weekdays



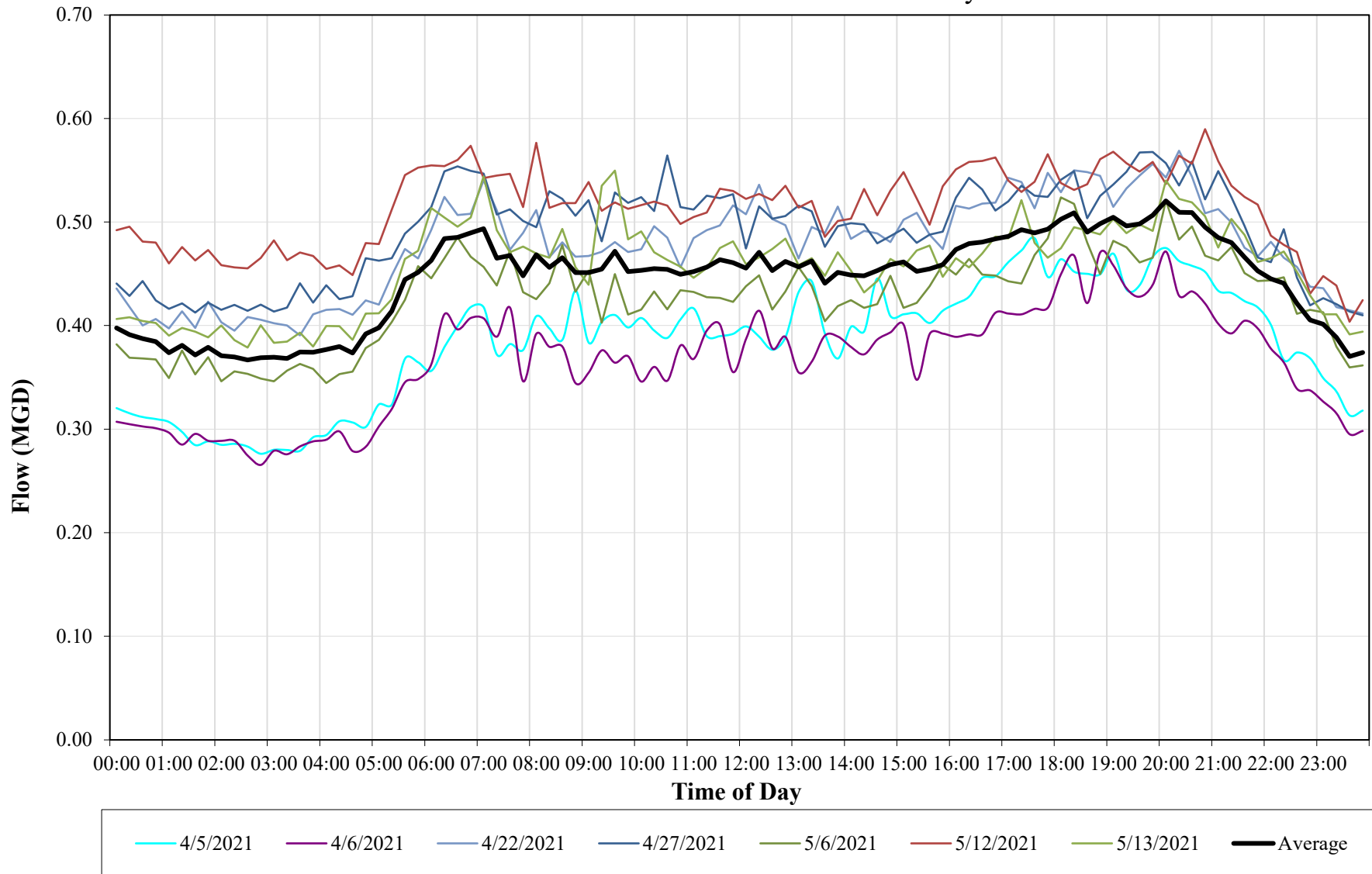
CE1-30 = FM05

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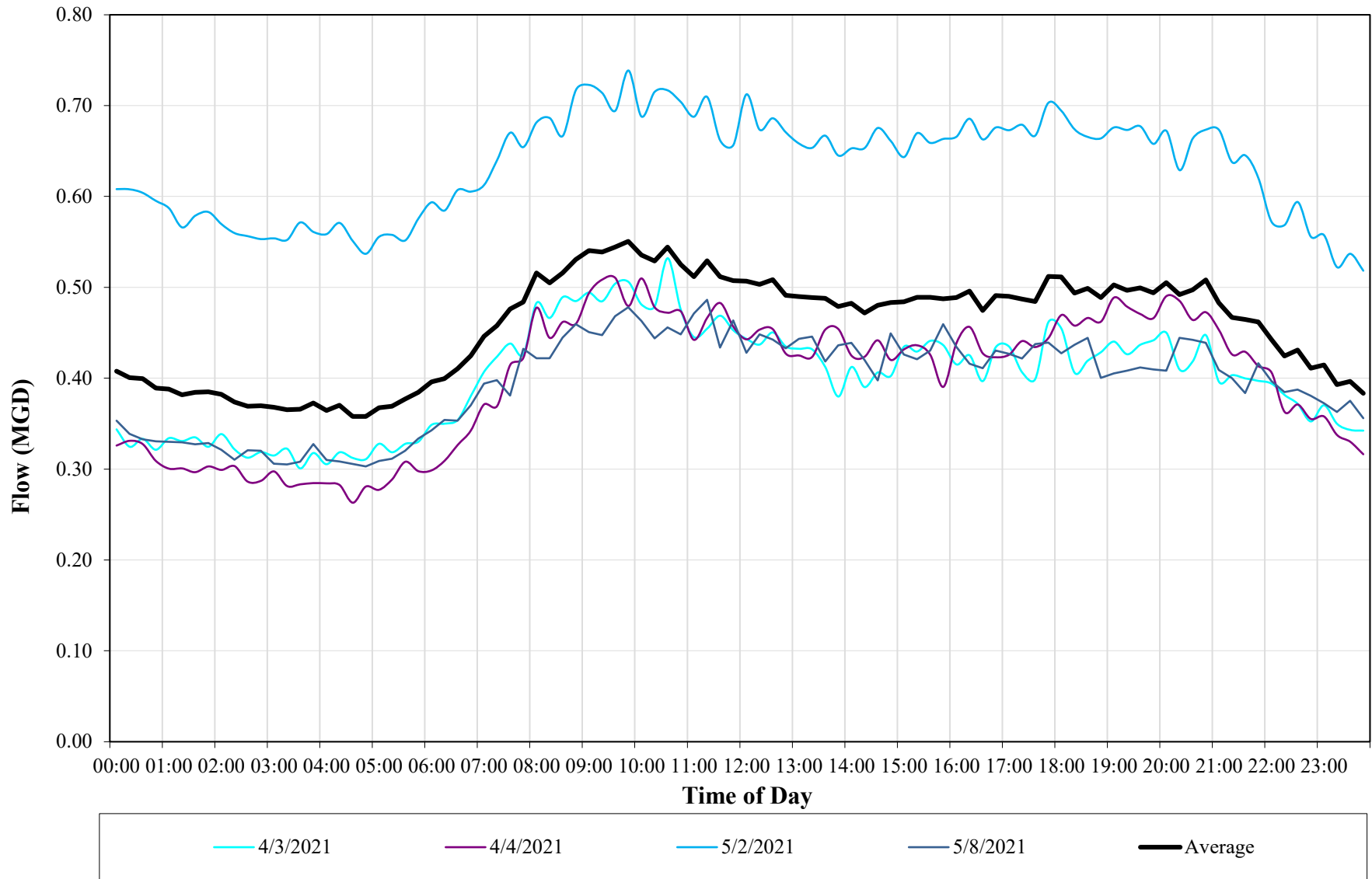
CE7-1 = FM03

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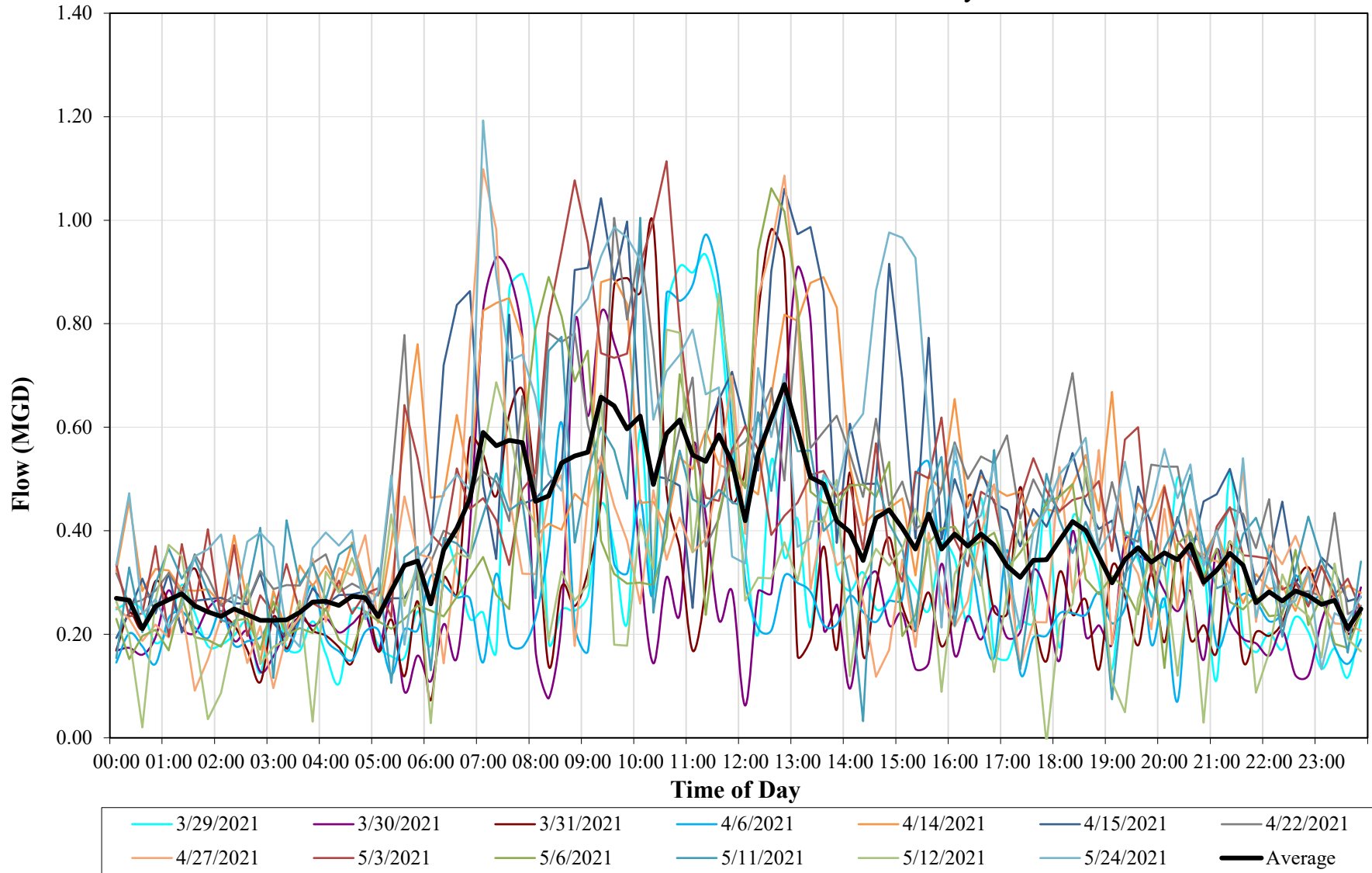
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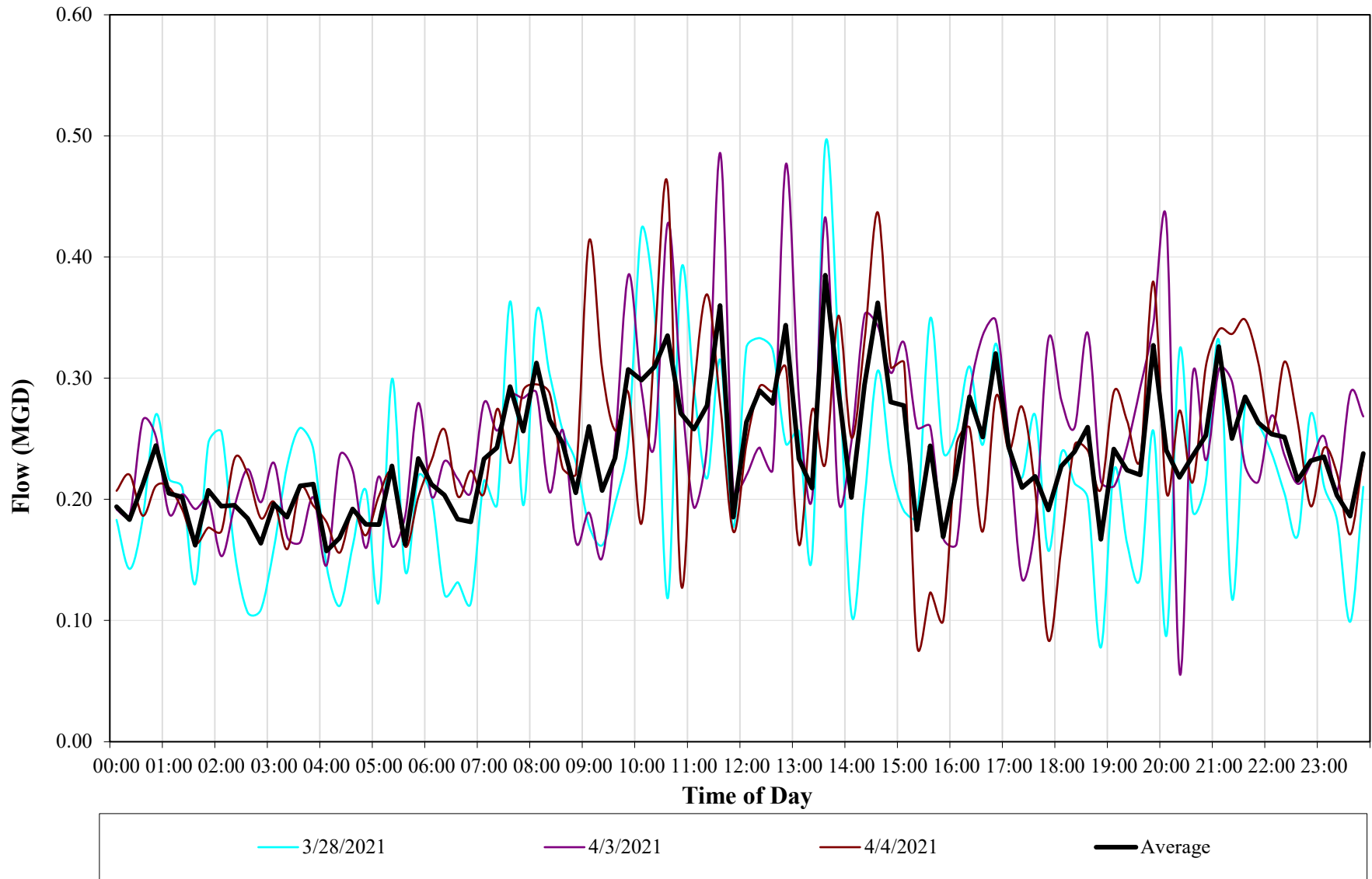
CW1-3 = FM01

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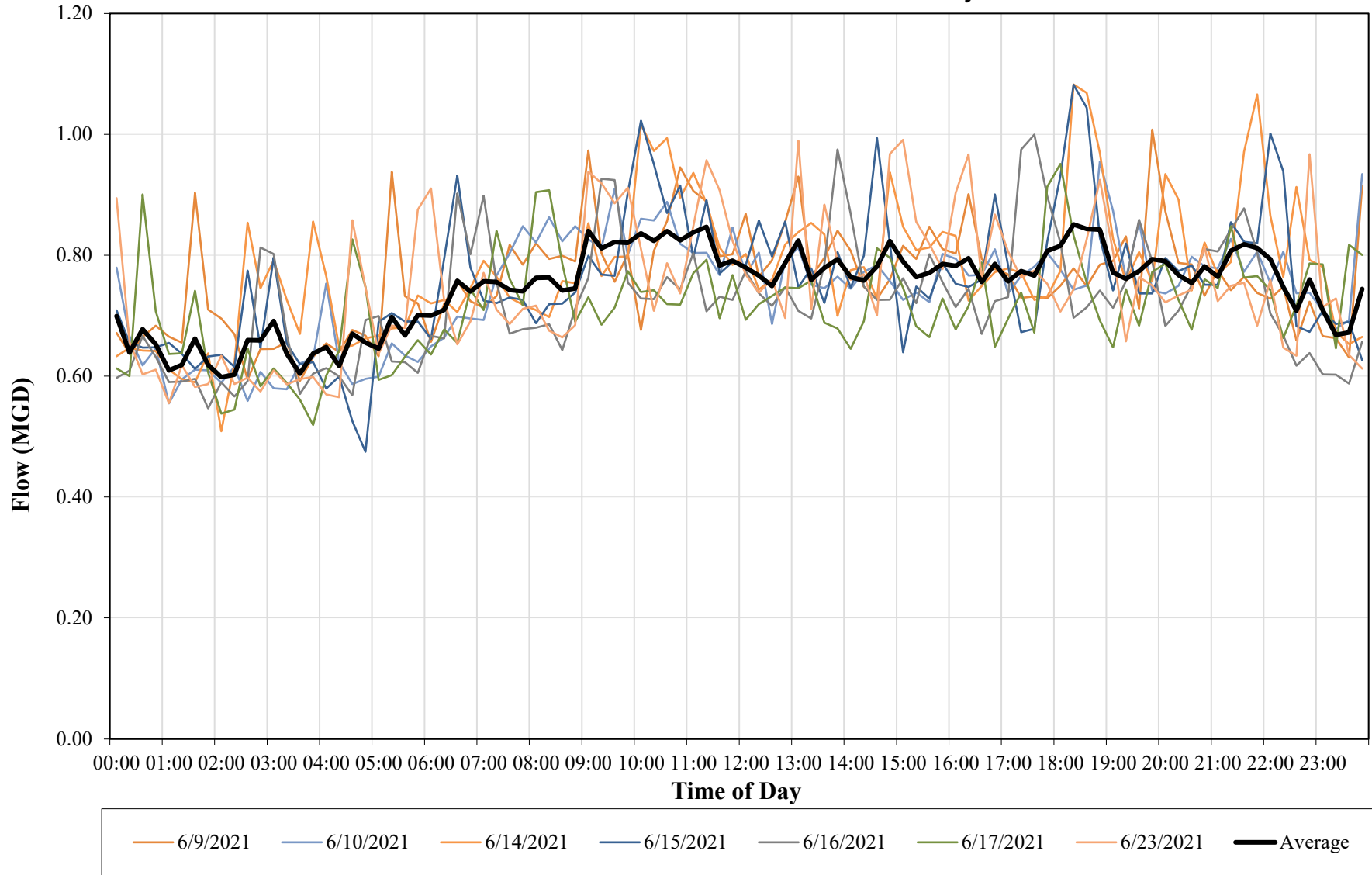
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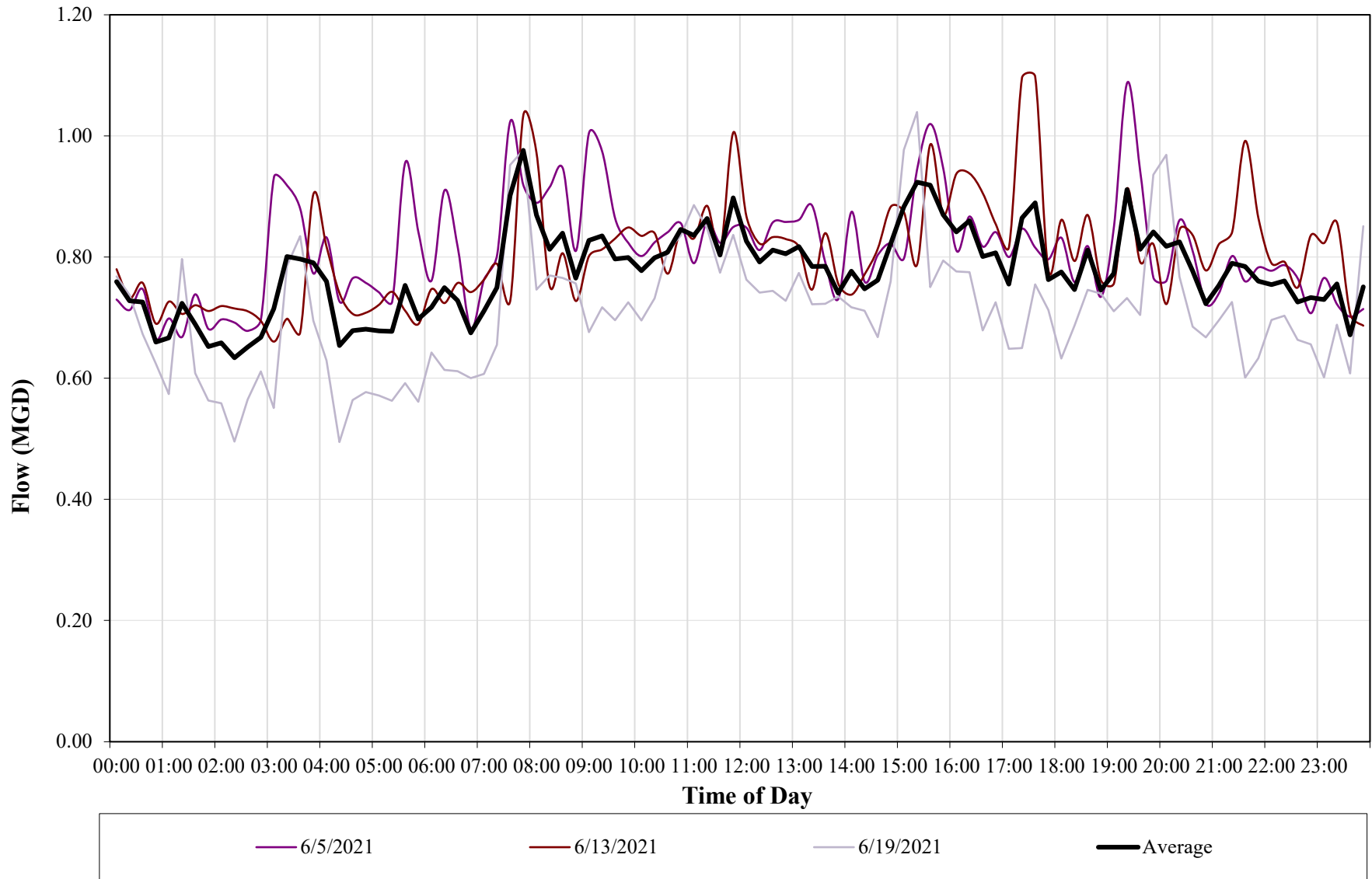
N1-31 = FM07

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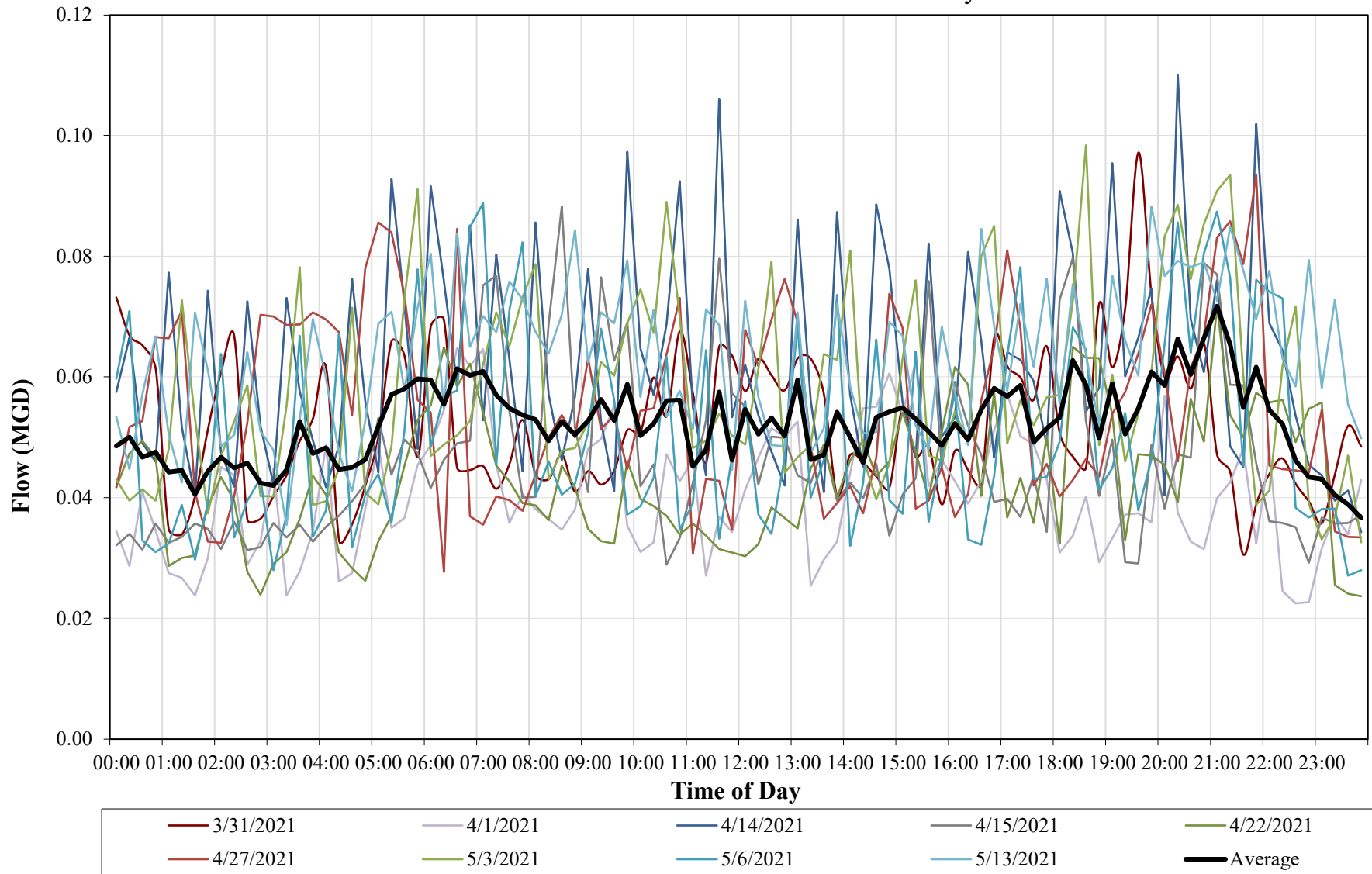
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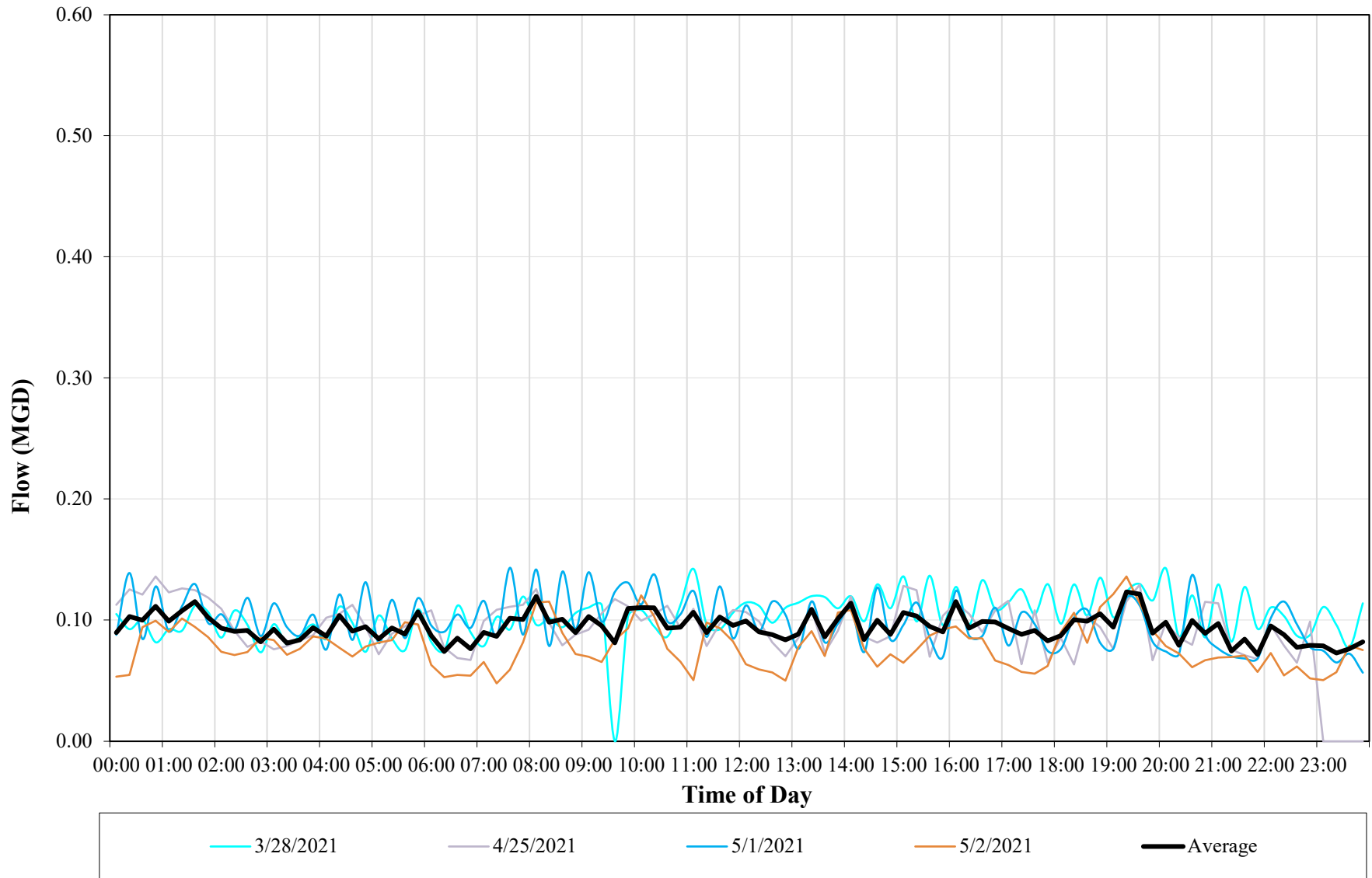
N3-5 = FM06

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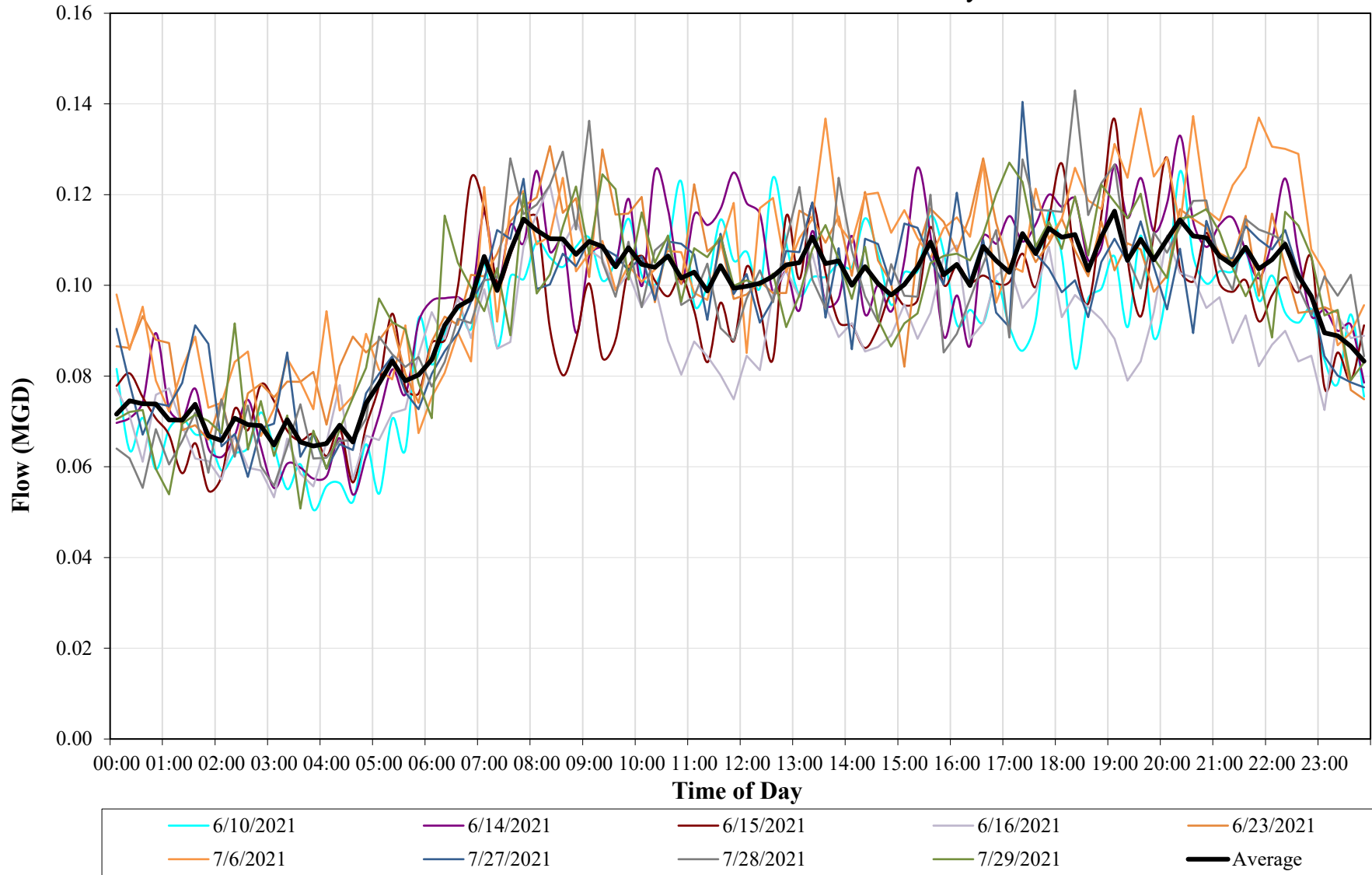
N3-5 = FM06

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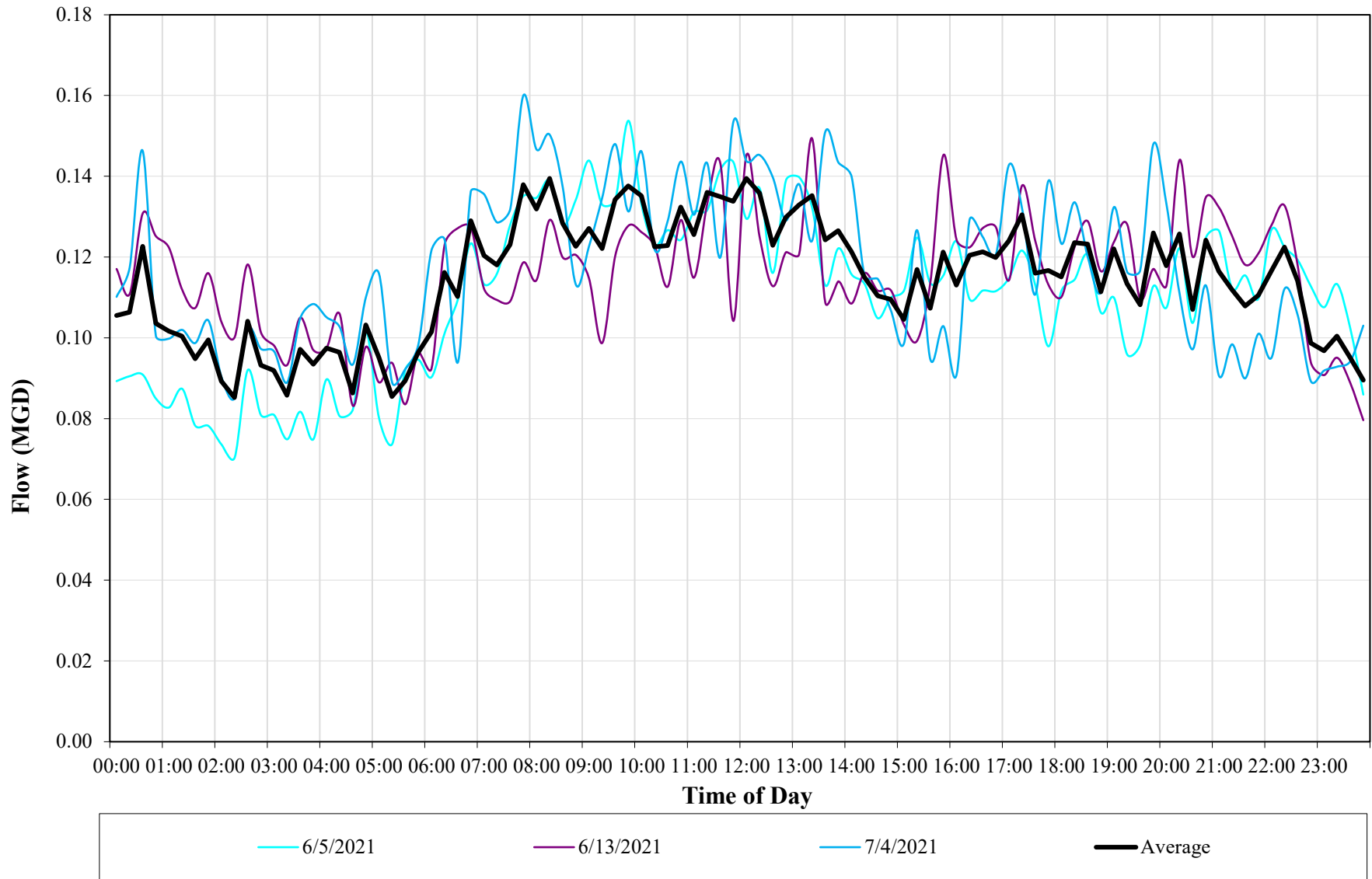
SEB3-6 = FM010

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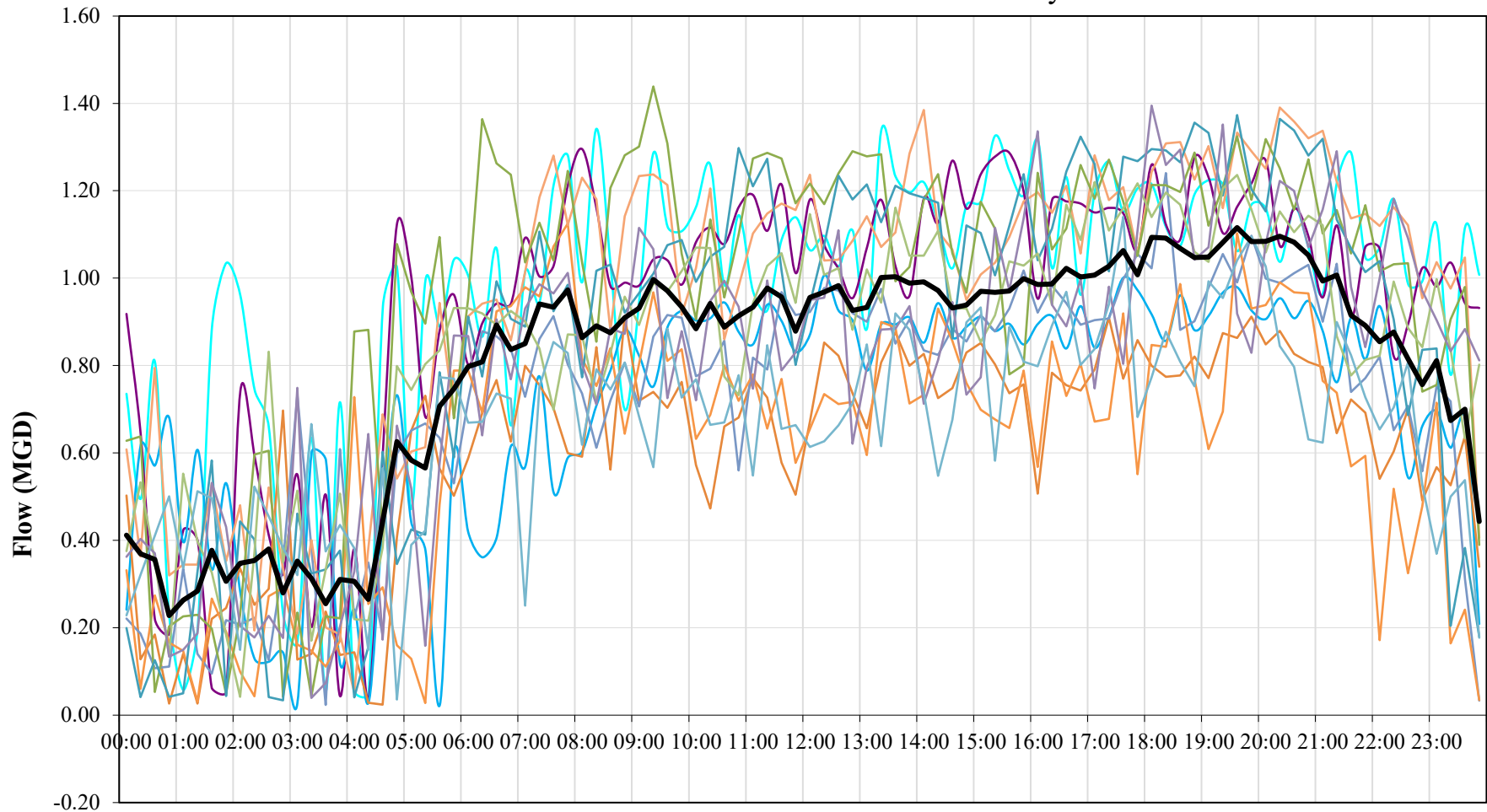
SEB3-6 = FM010

ADDF DIURNAL CURVES Weekend



SEC3-20 = FM08

ADDF DIURNAL CURVES Weekdays

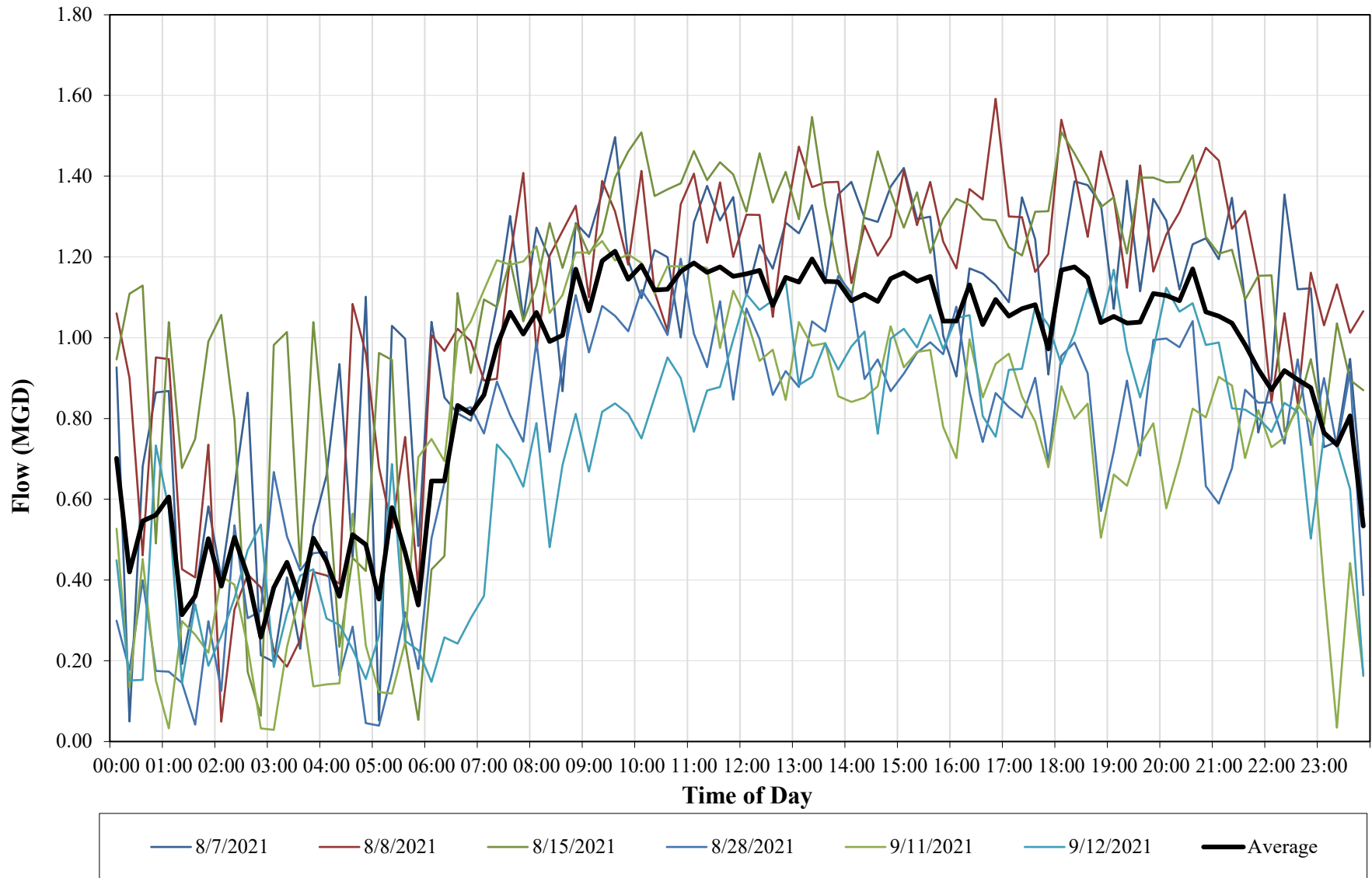


Time of Day						
8/4/2021	8/5/2021	9/6/2021	9/7/2021	9/8/2021	9/9/2021	8/16/2021
8/24/2021	8/25/2021	8/26/2021	9/2/2021	9/13/2021	Average	



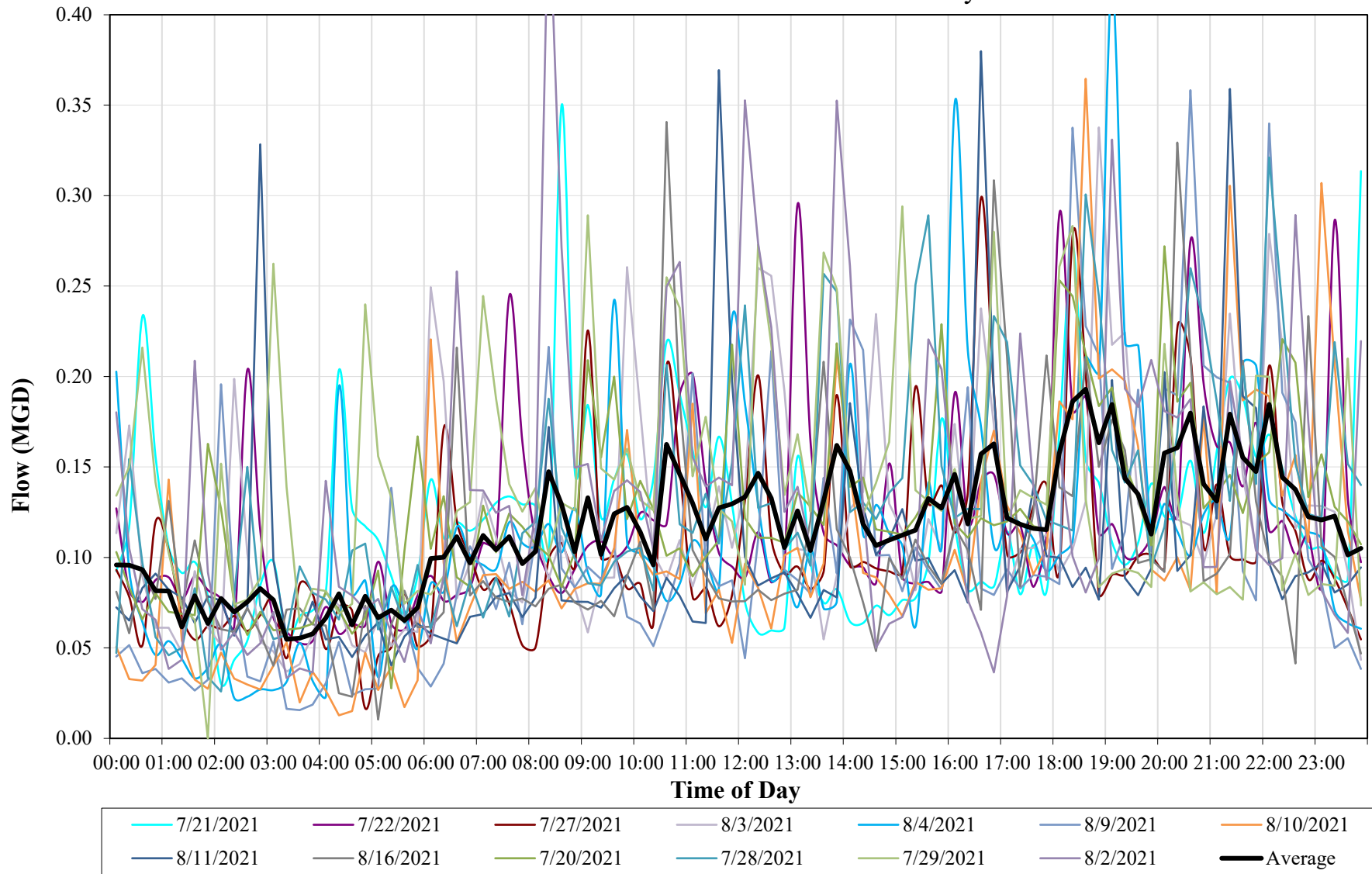
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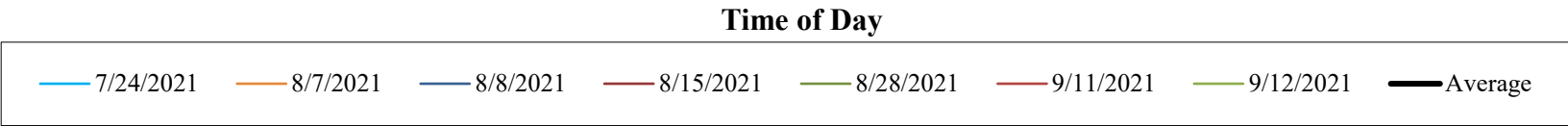
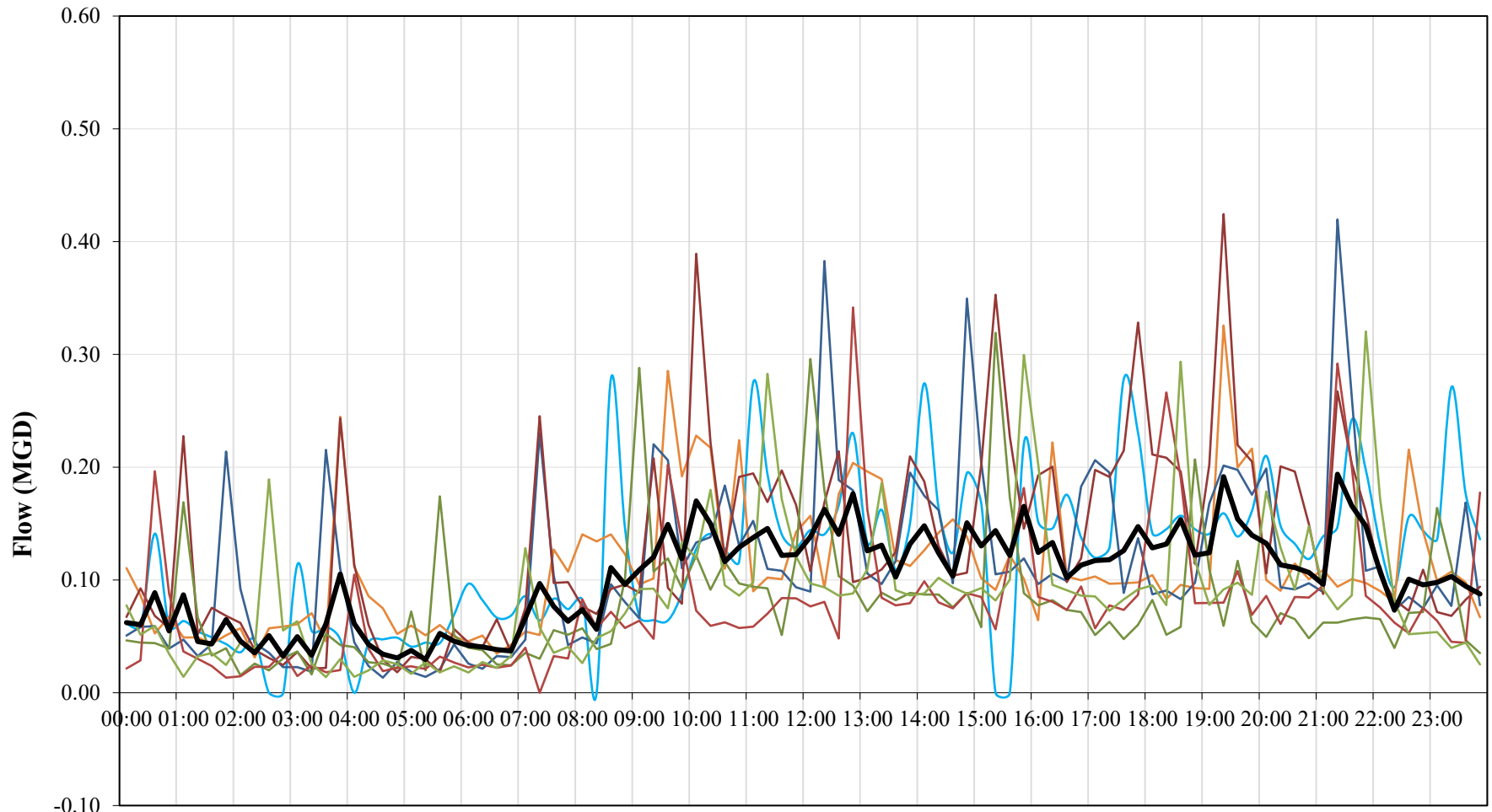
SED1-29 = FM09

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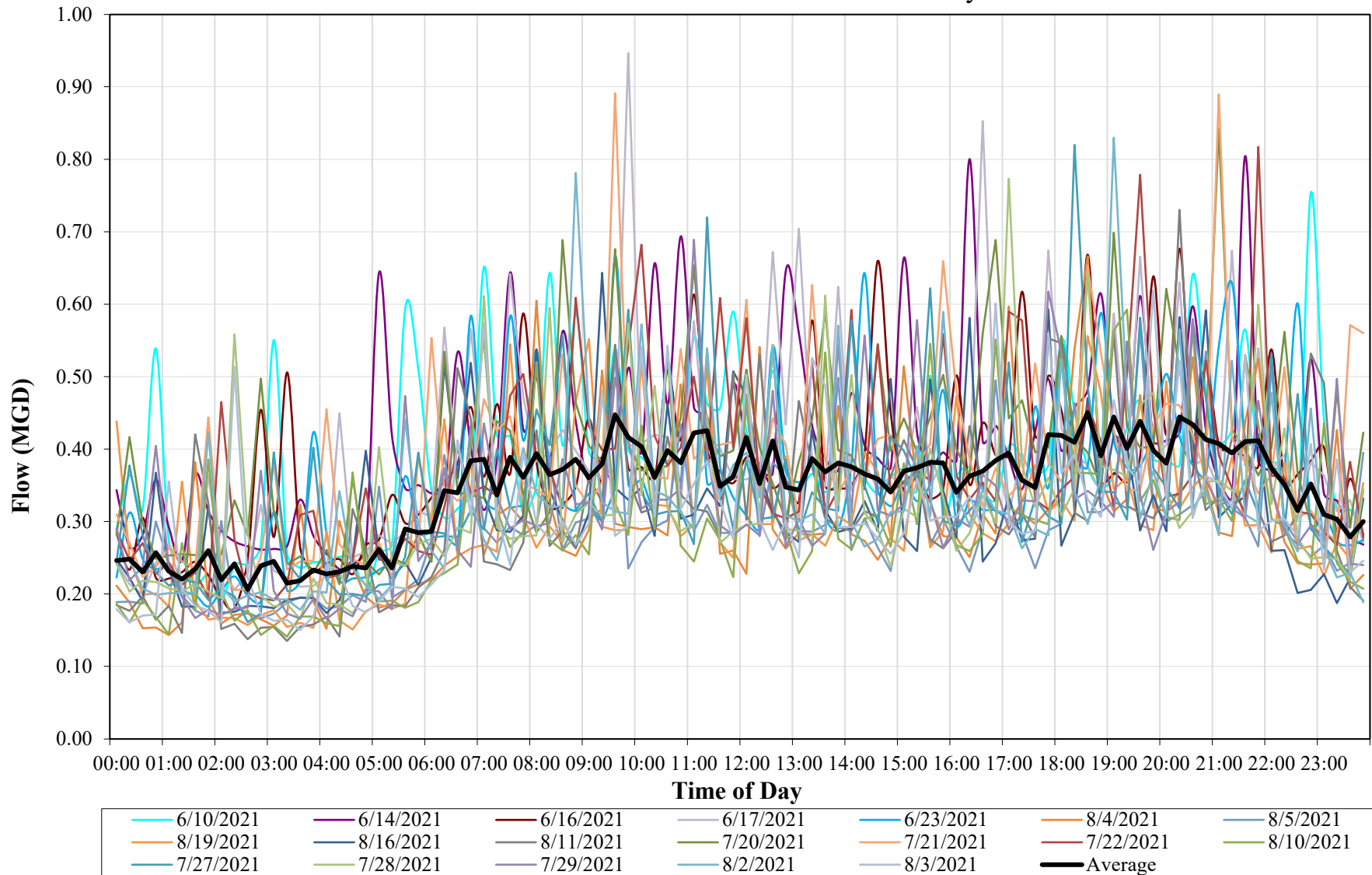
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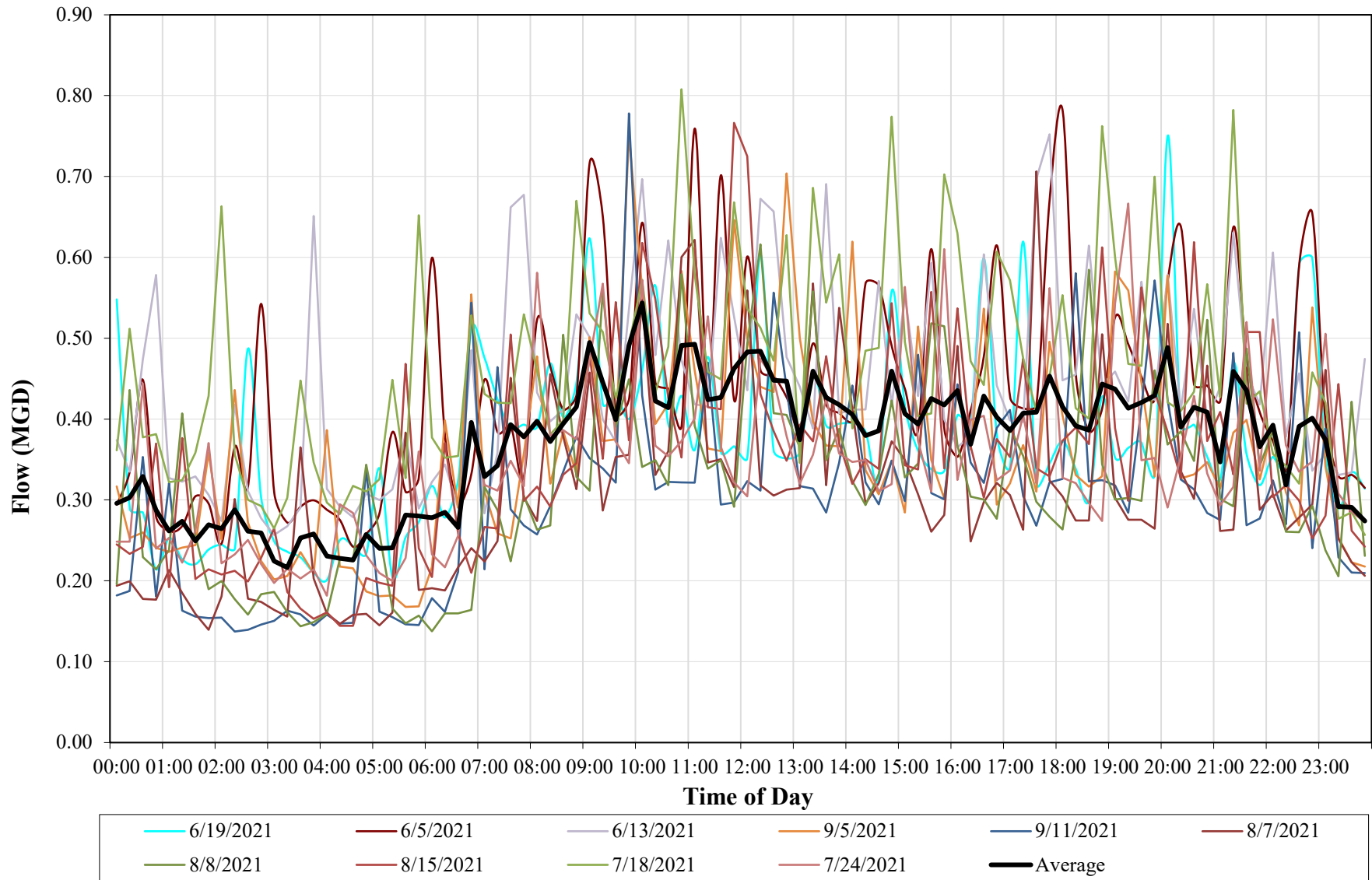
SWLST-25 = FM02

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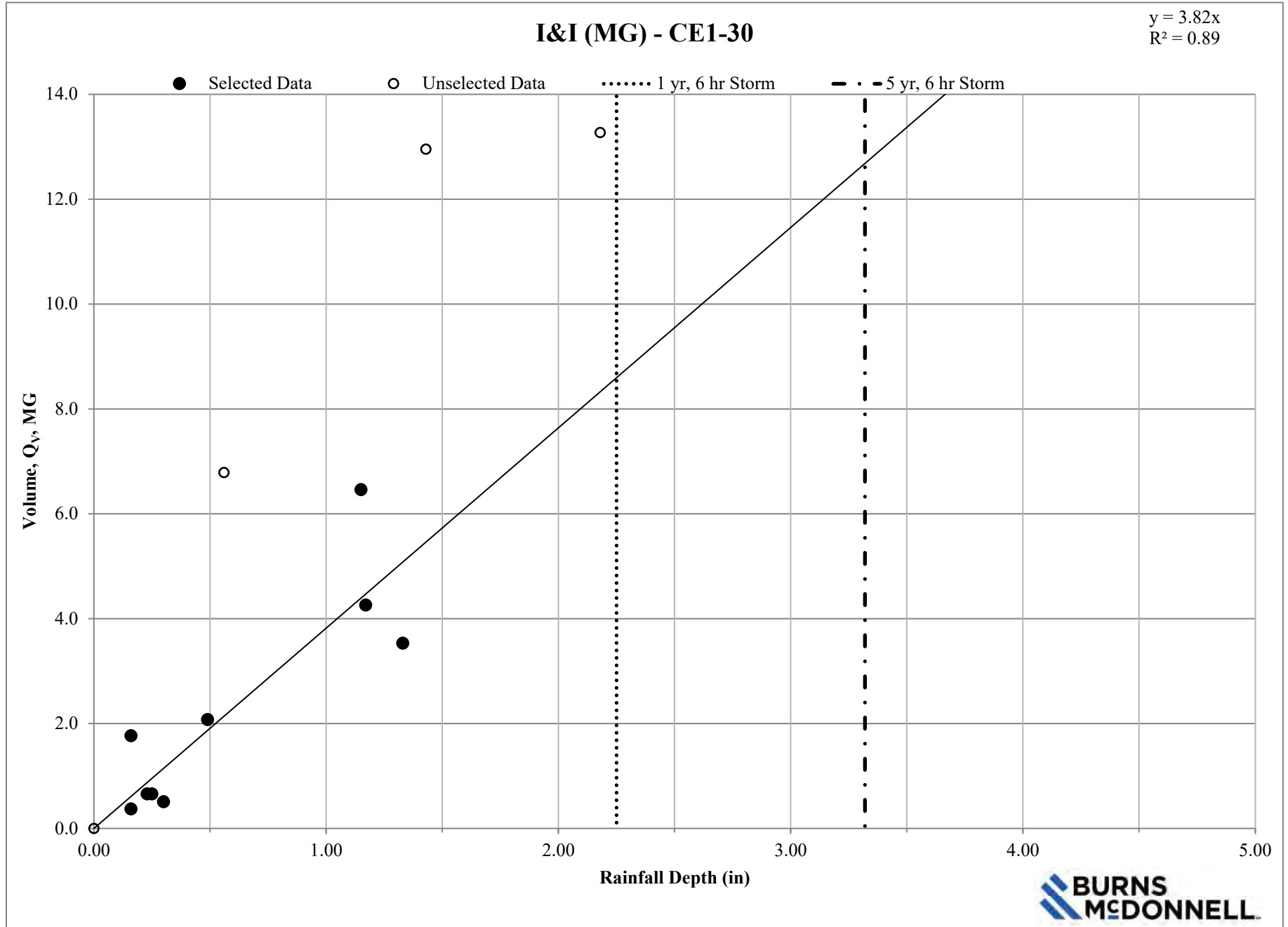


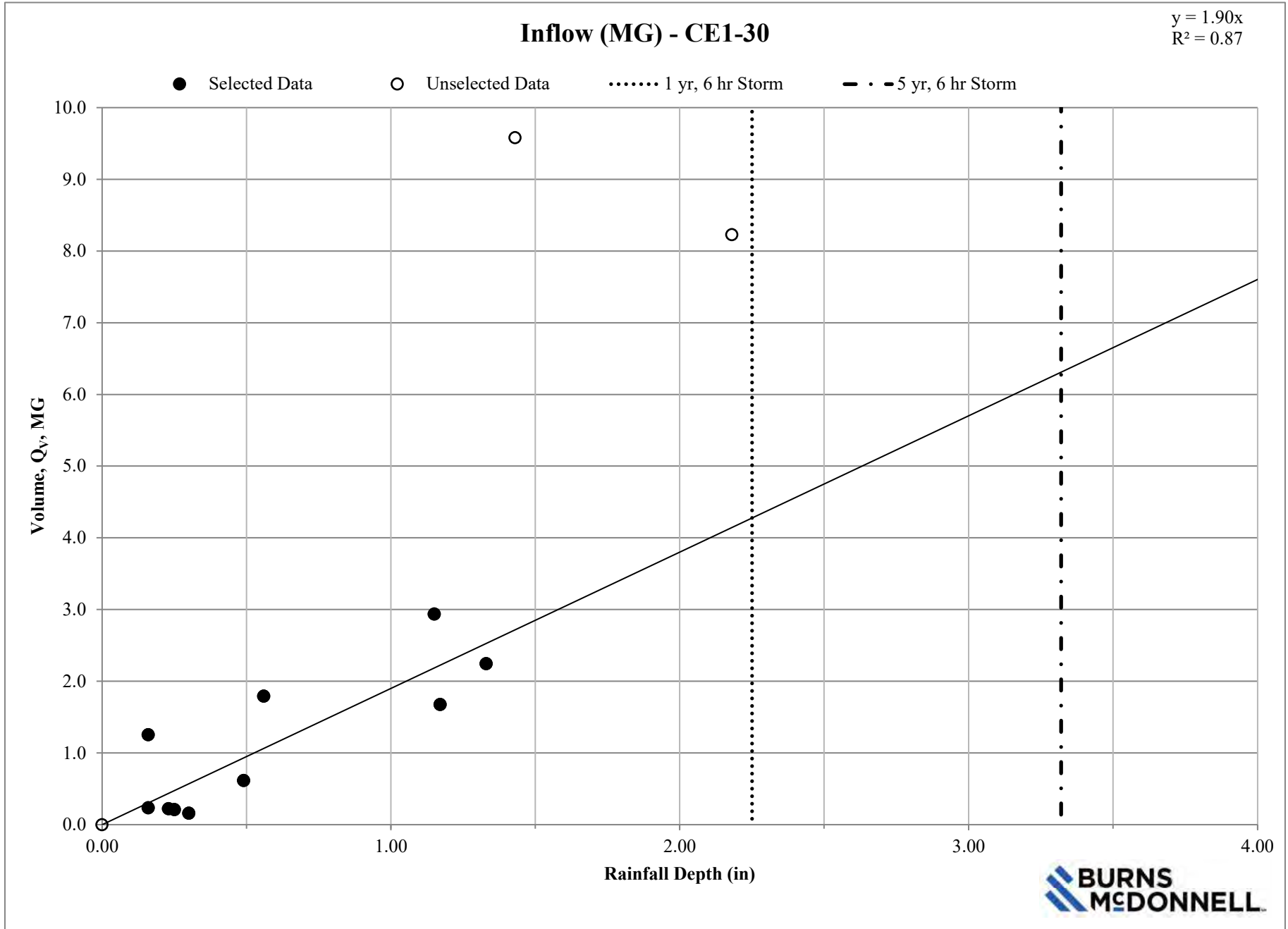
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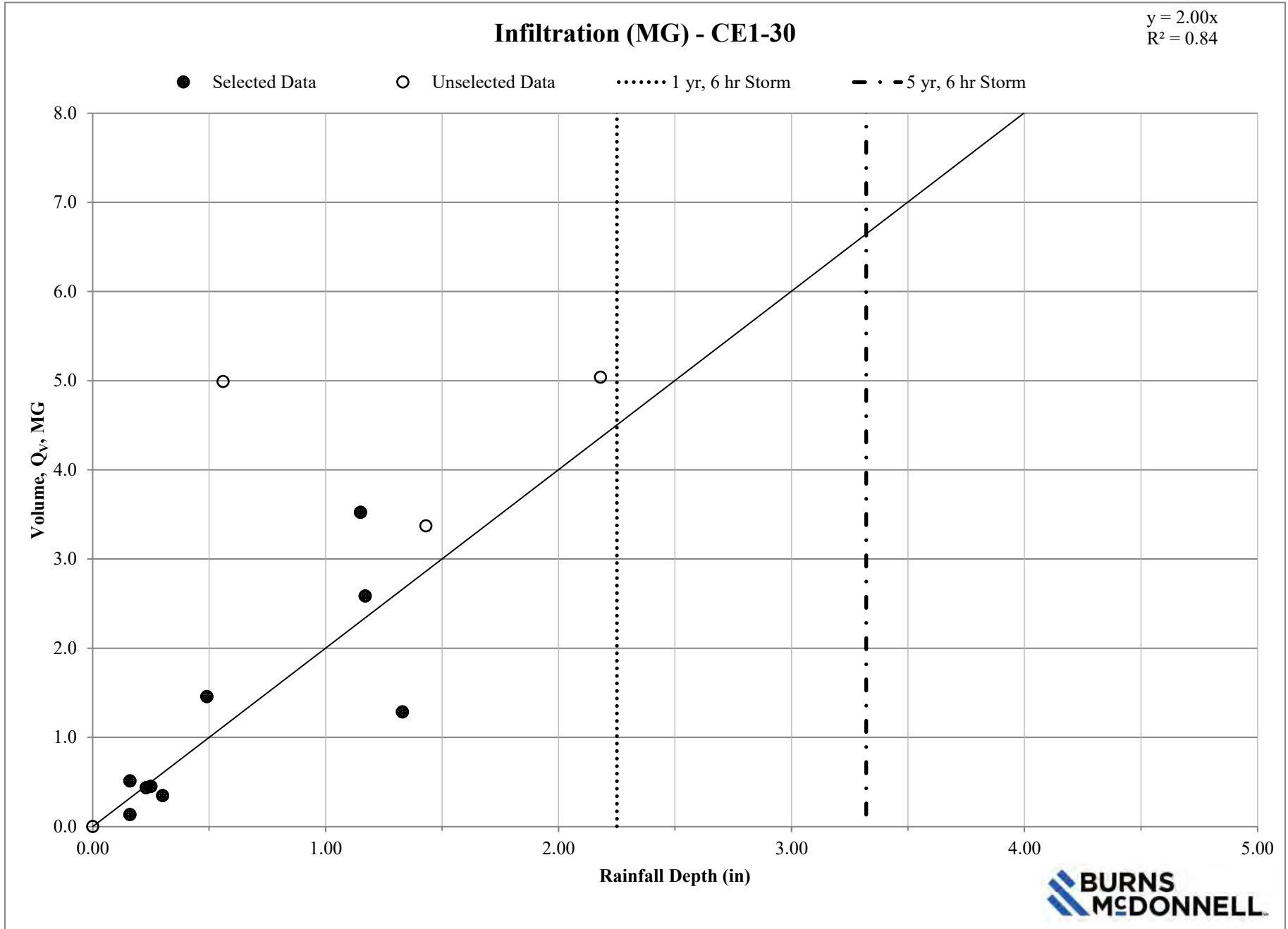
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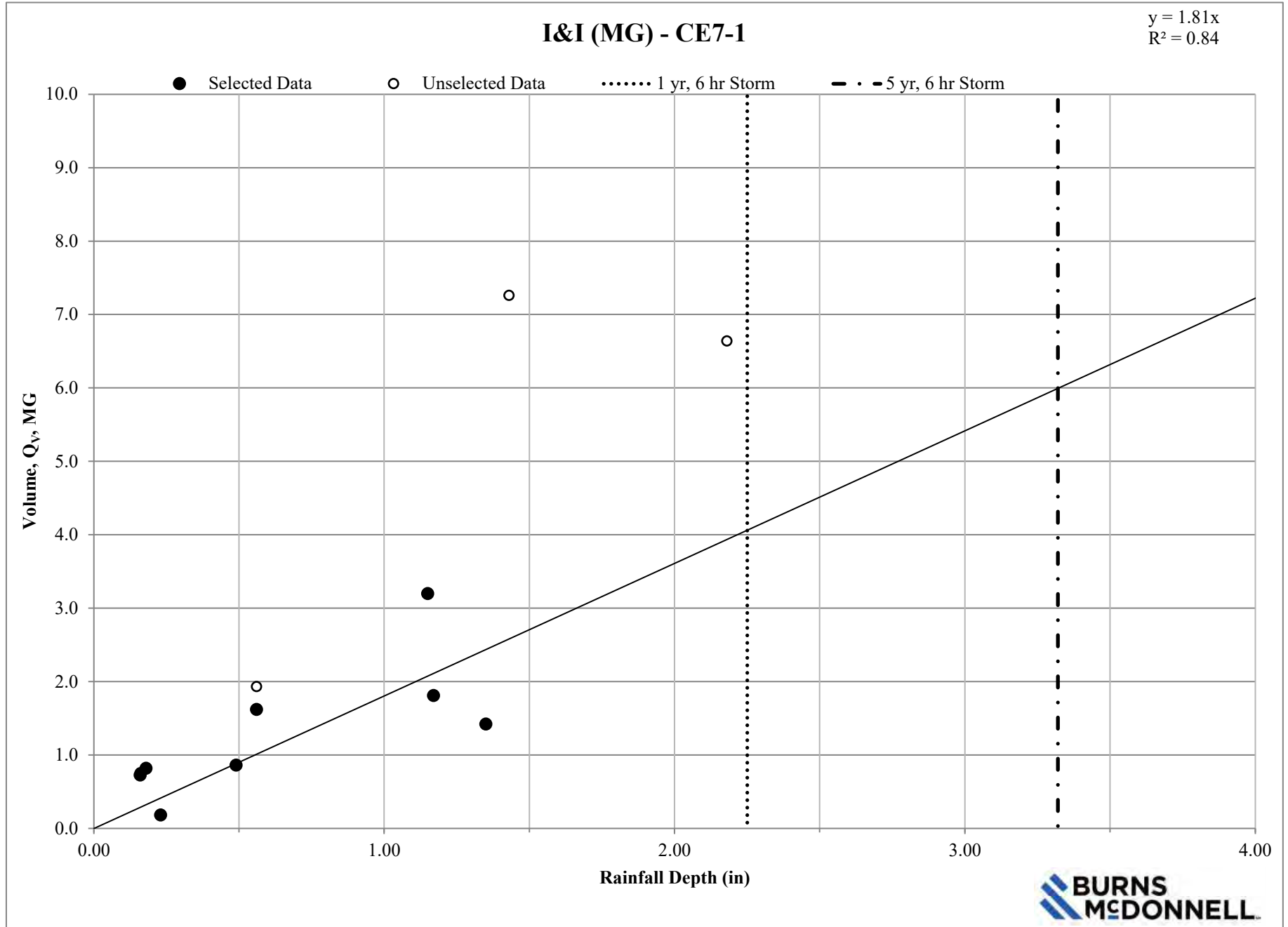


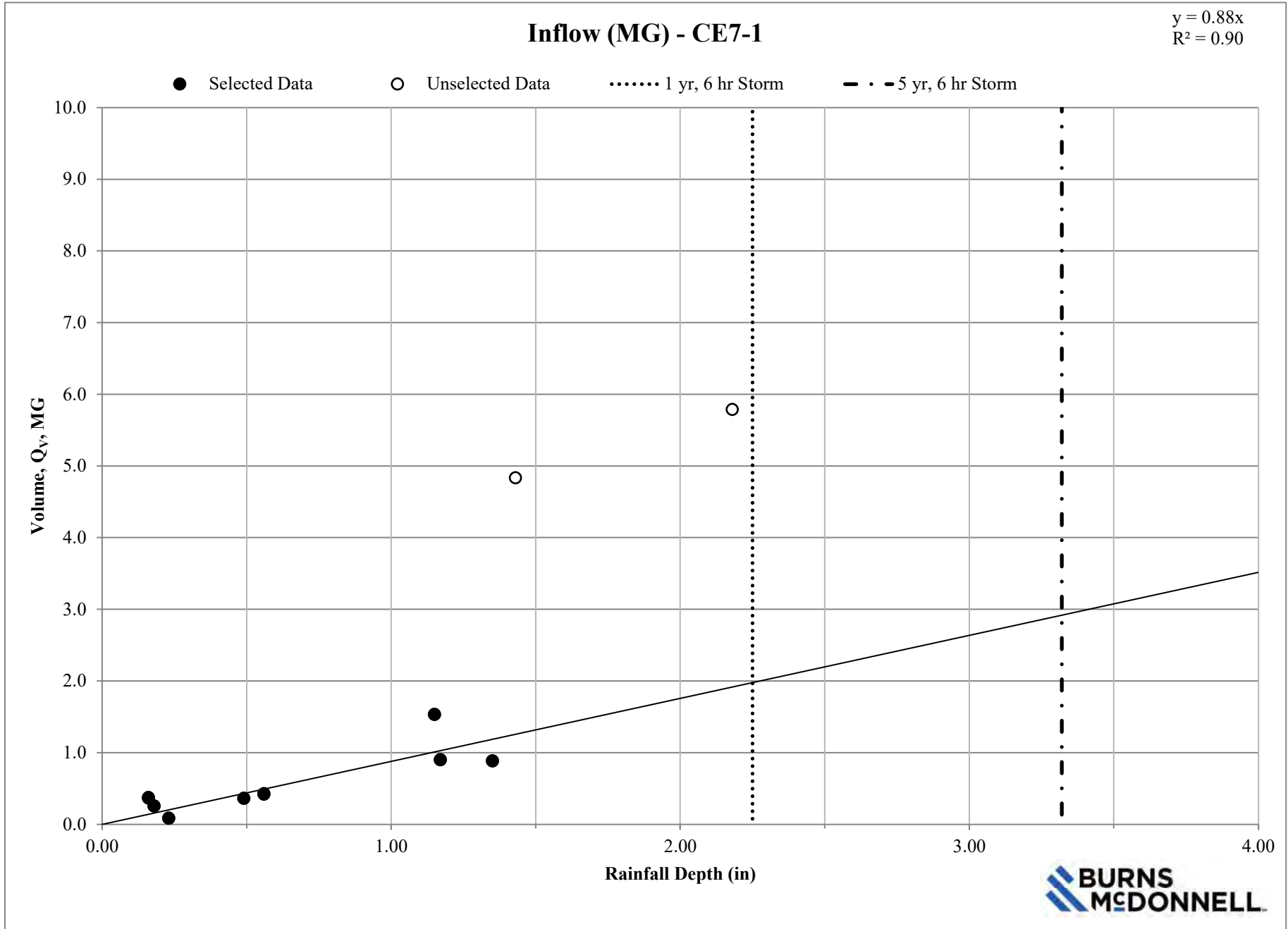
APPENDIX B – VOLUME VS. DEPTH GRAPHS

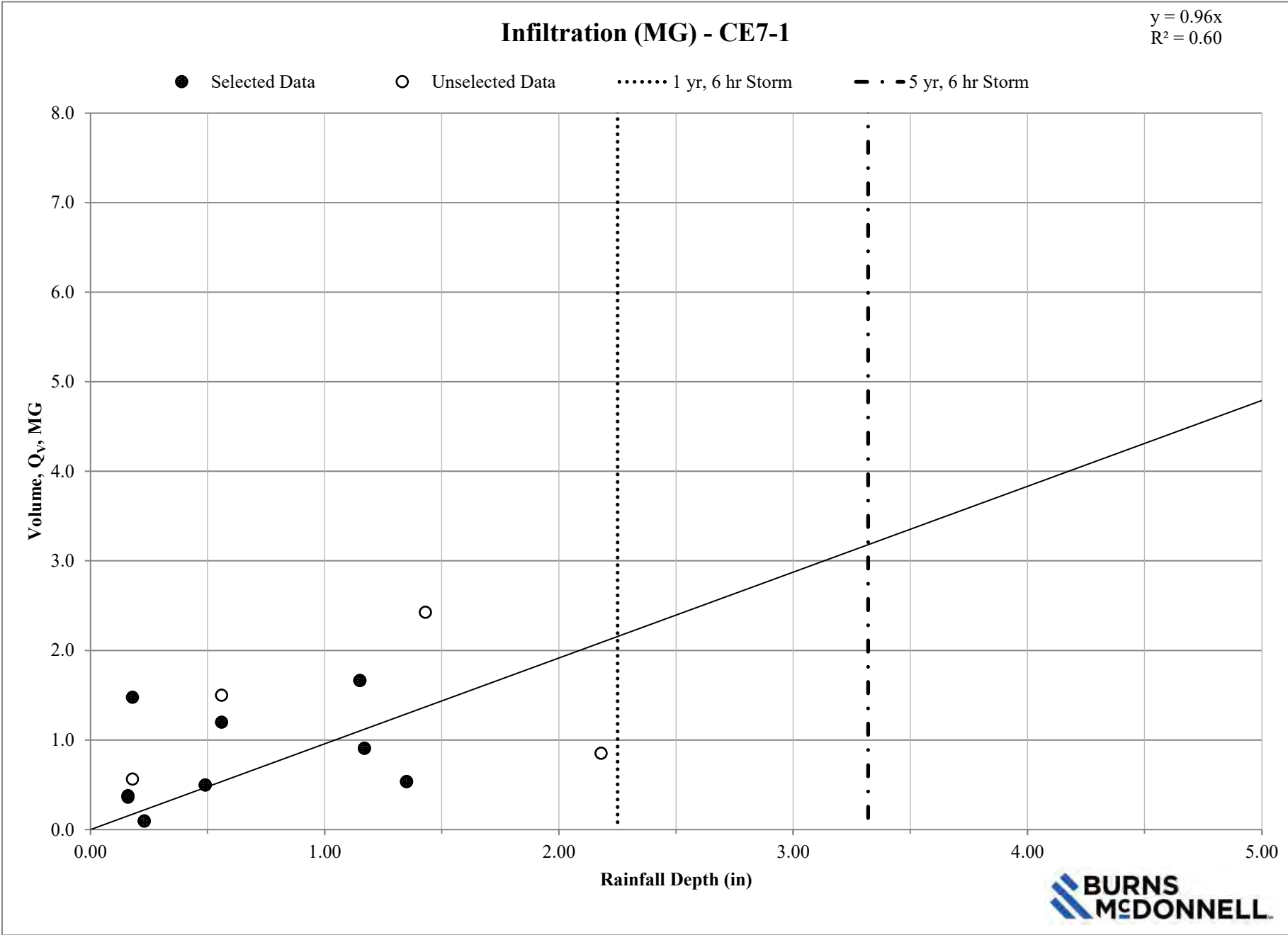


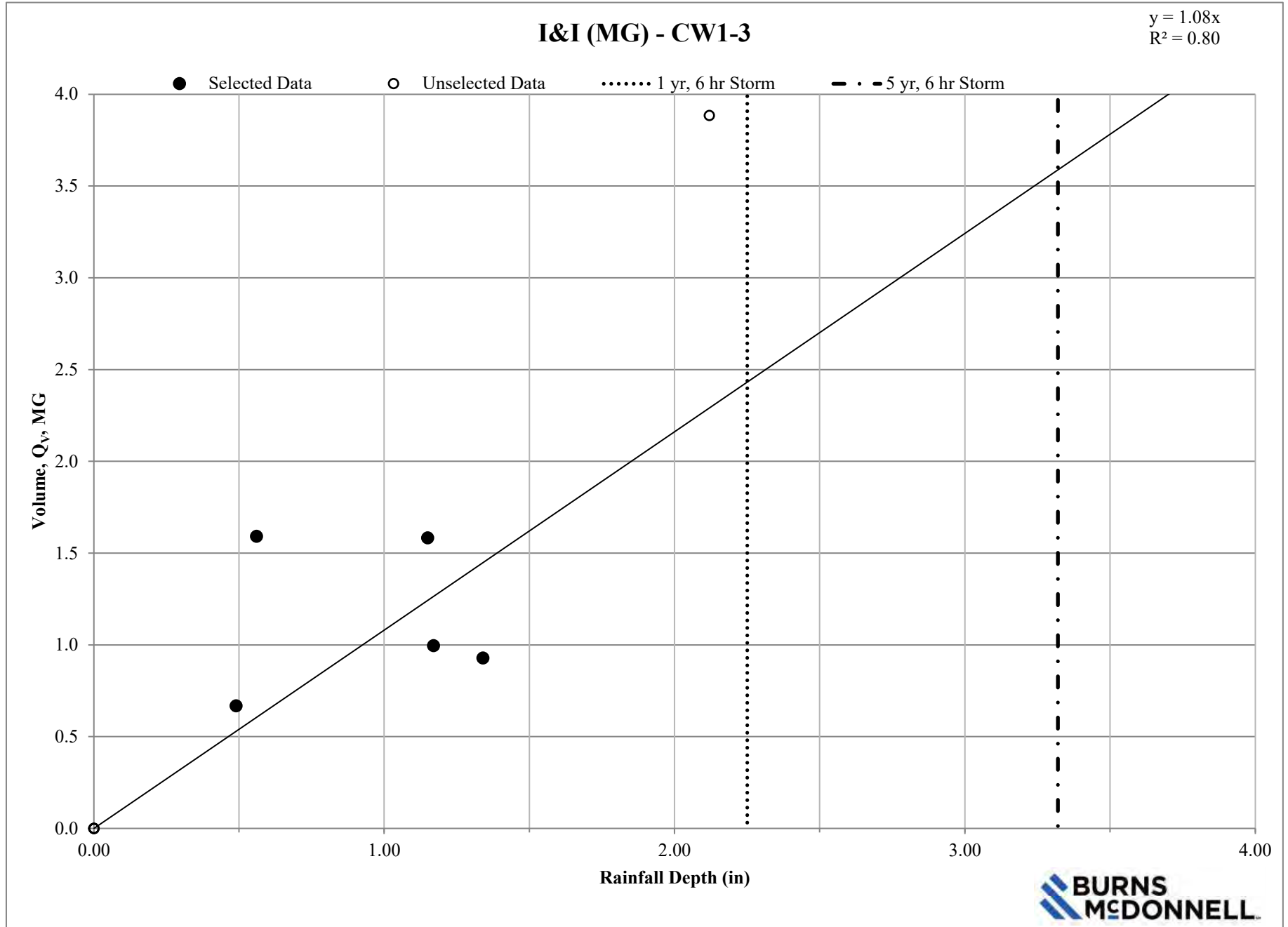


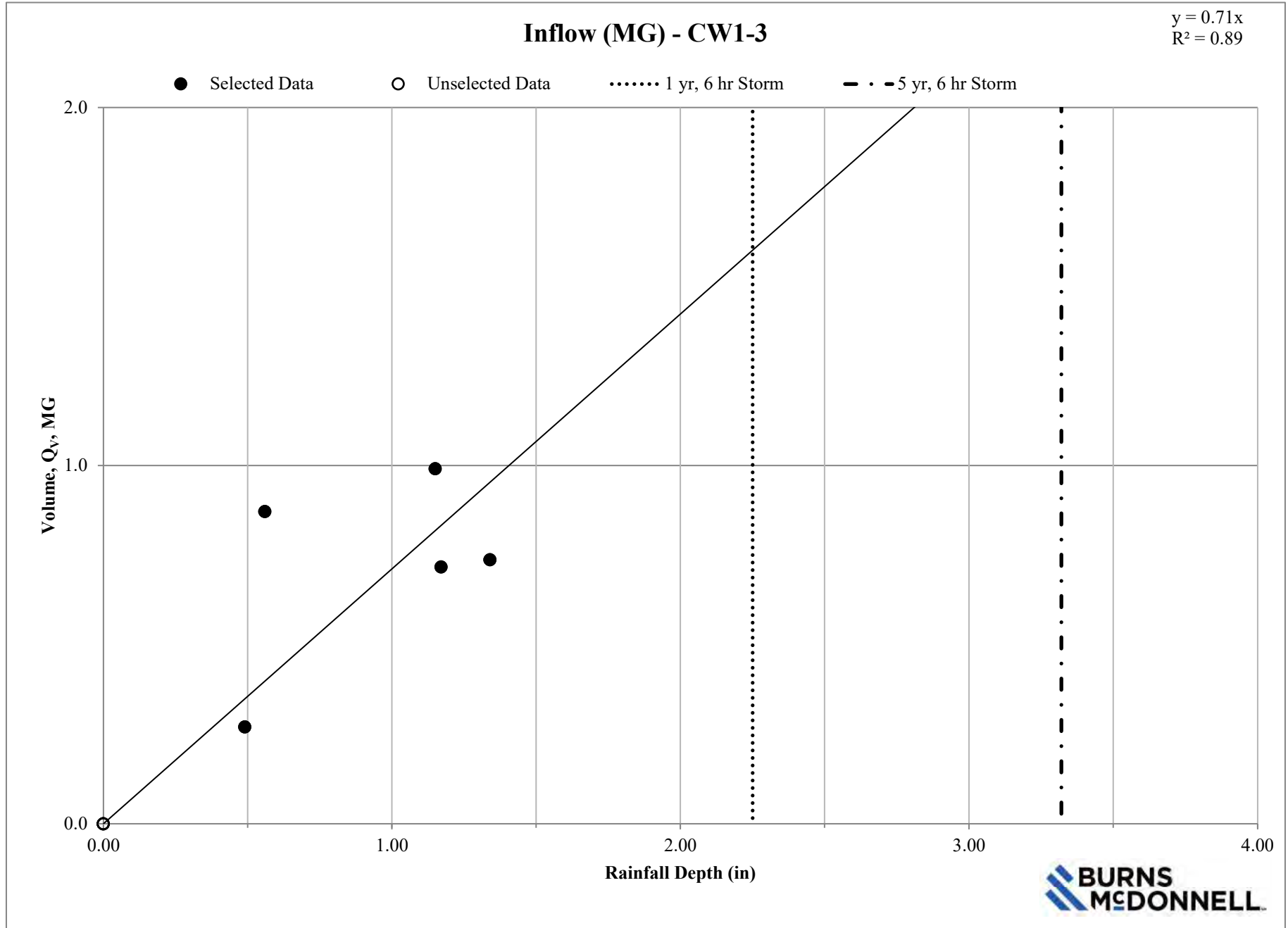


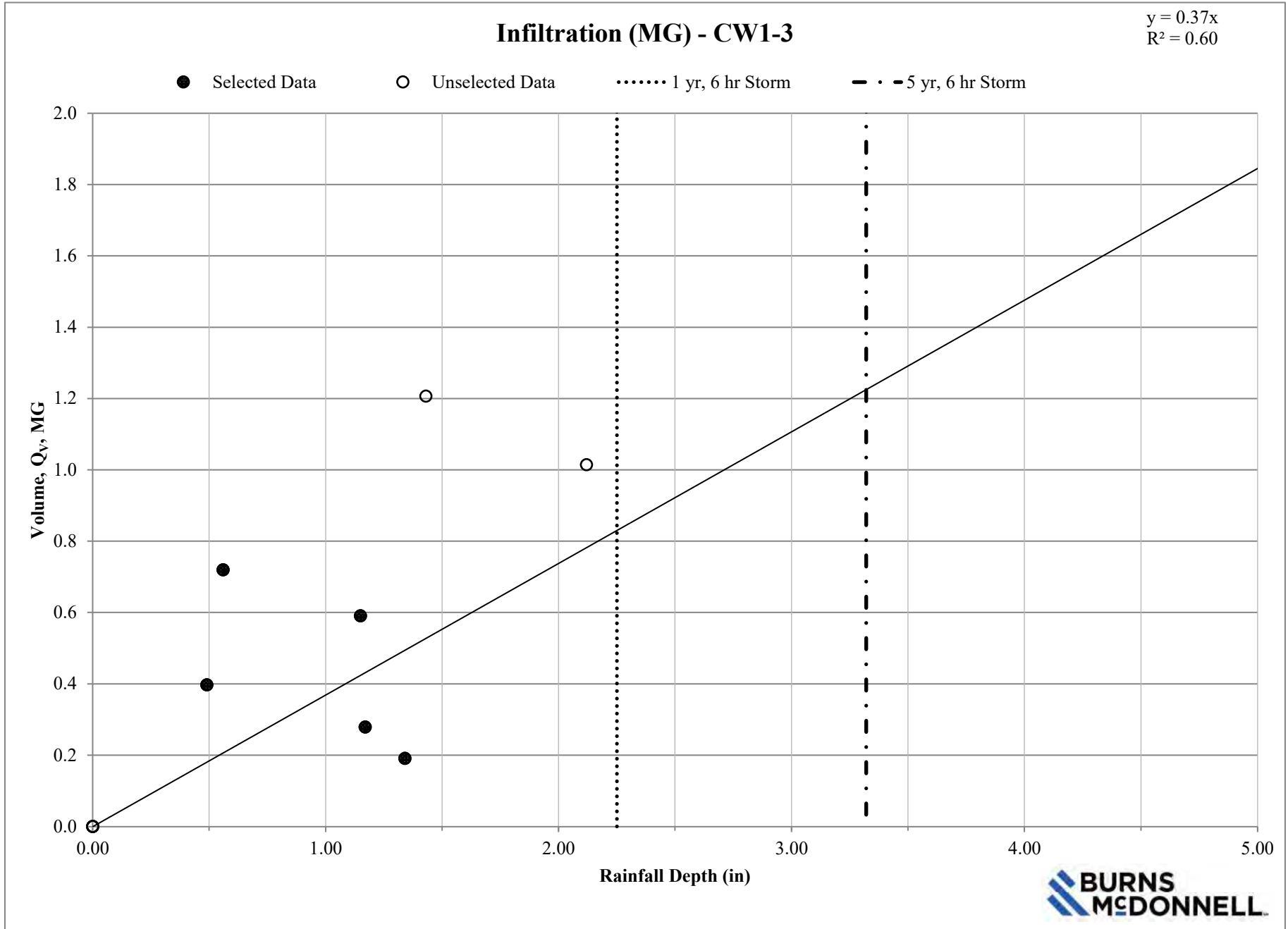


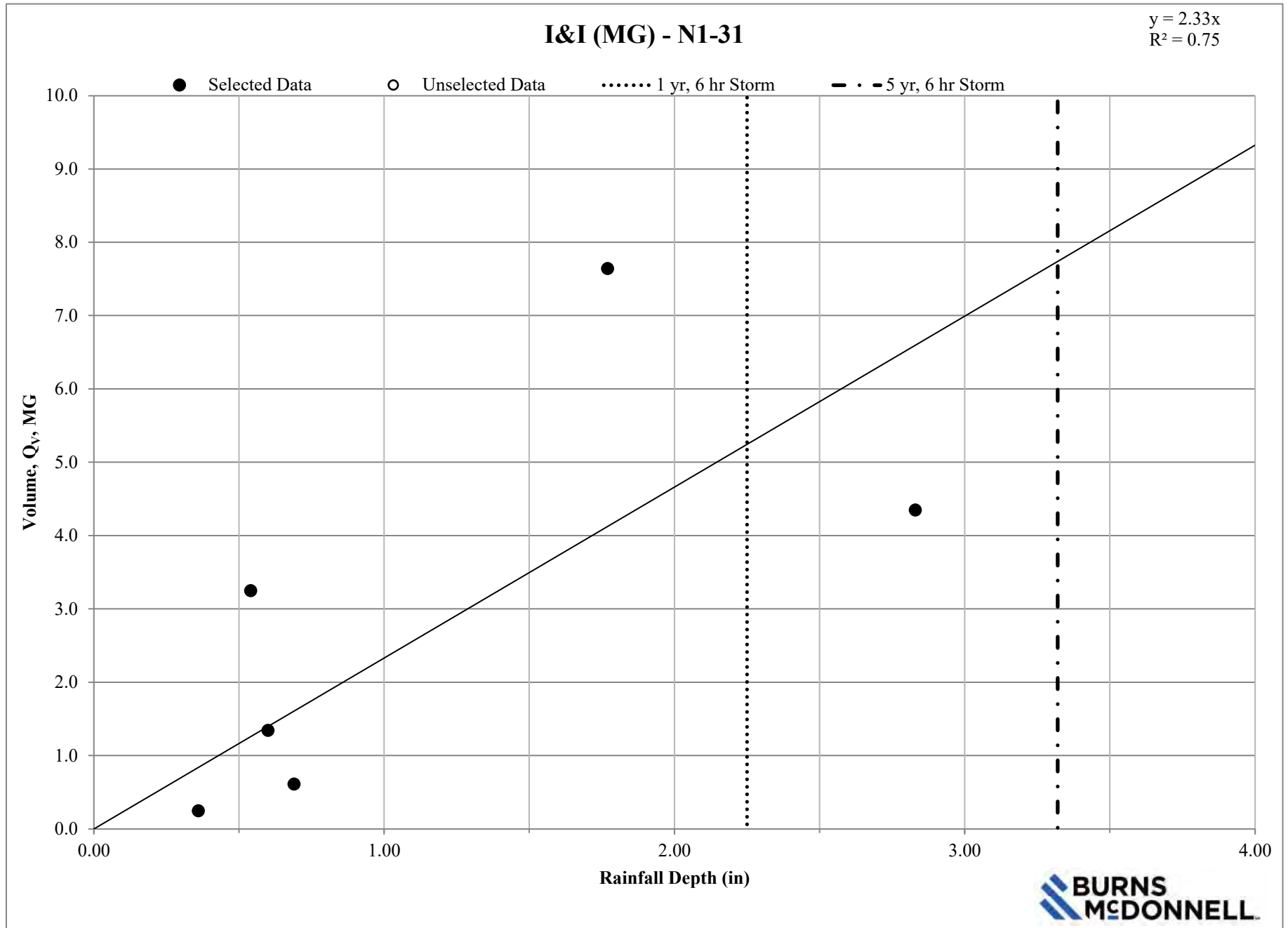


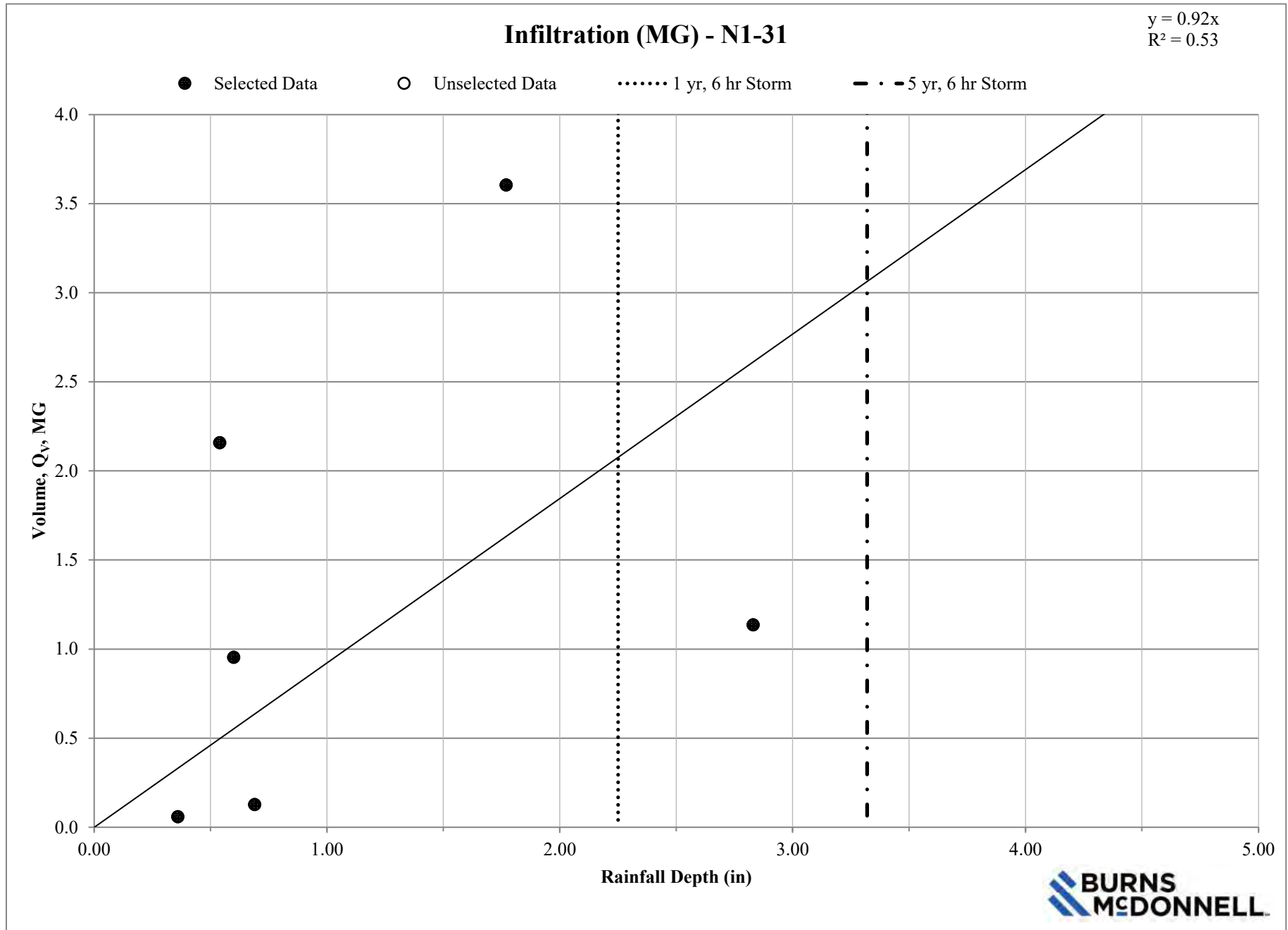


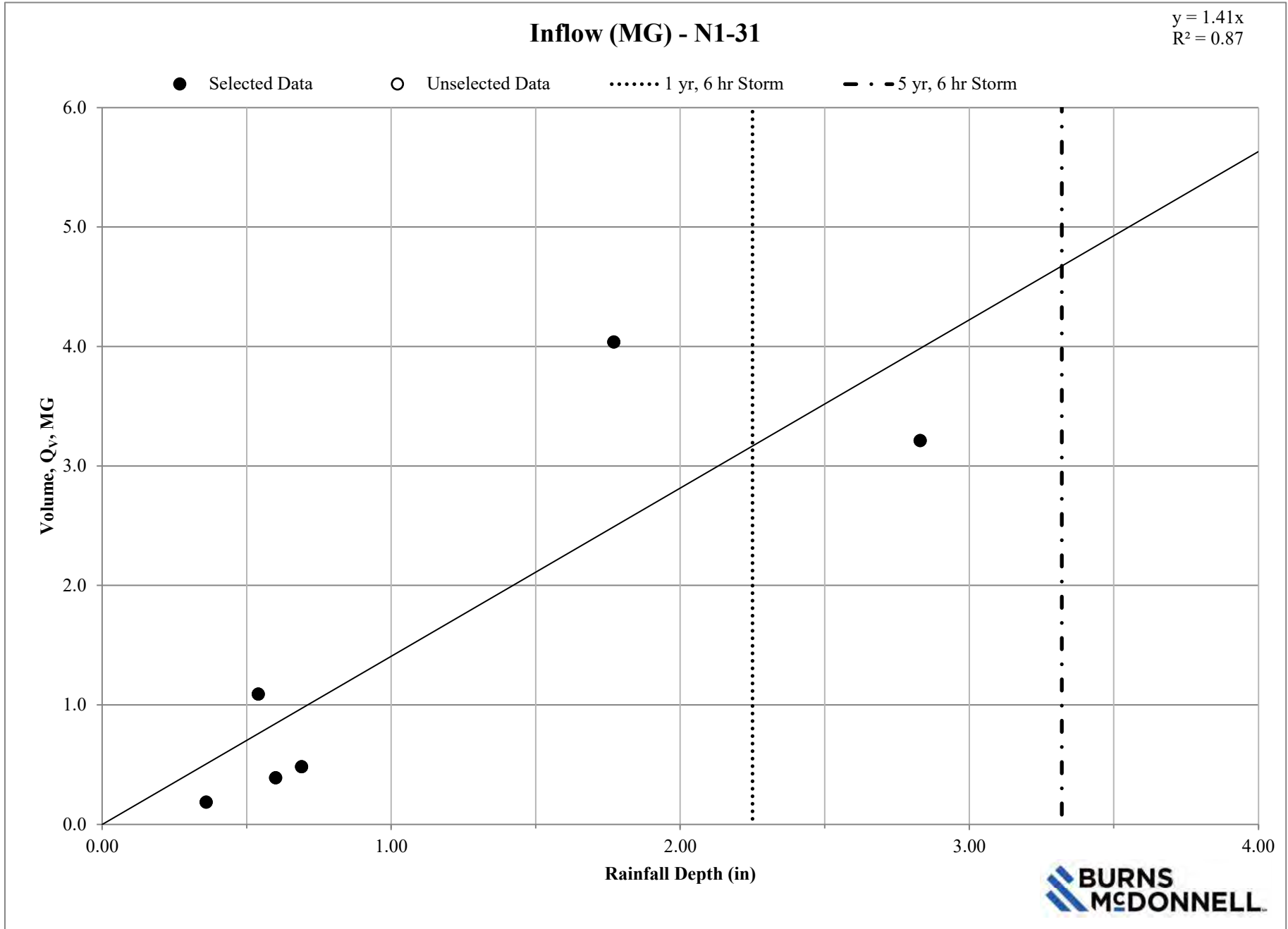


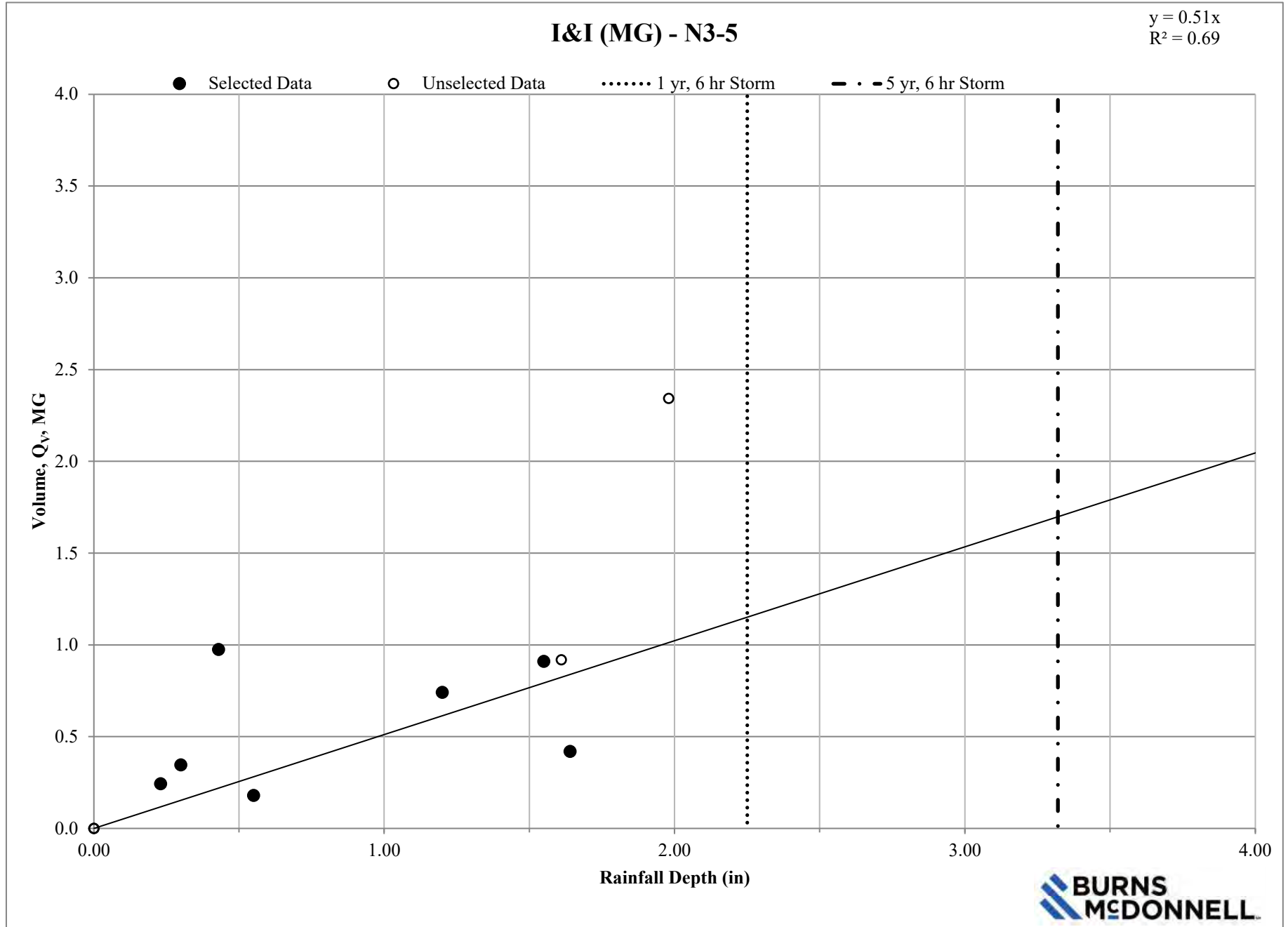


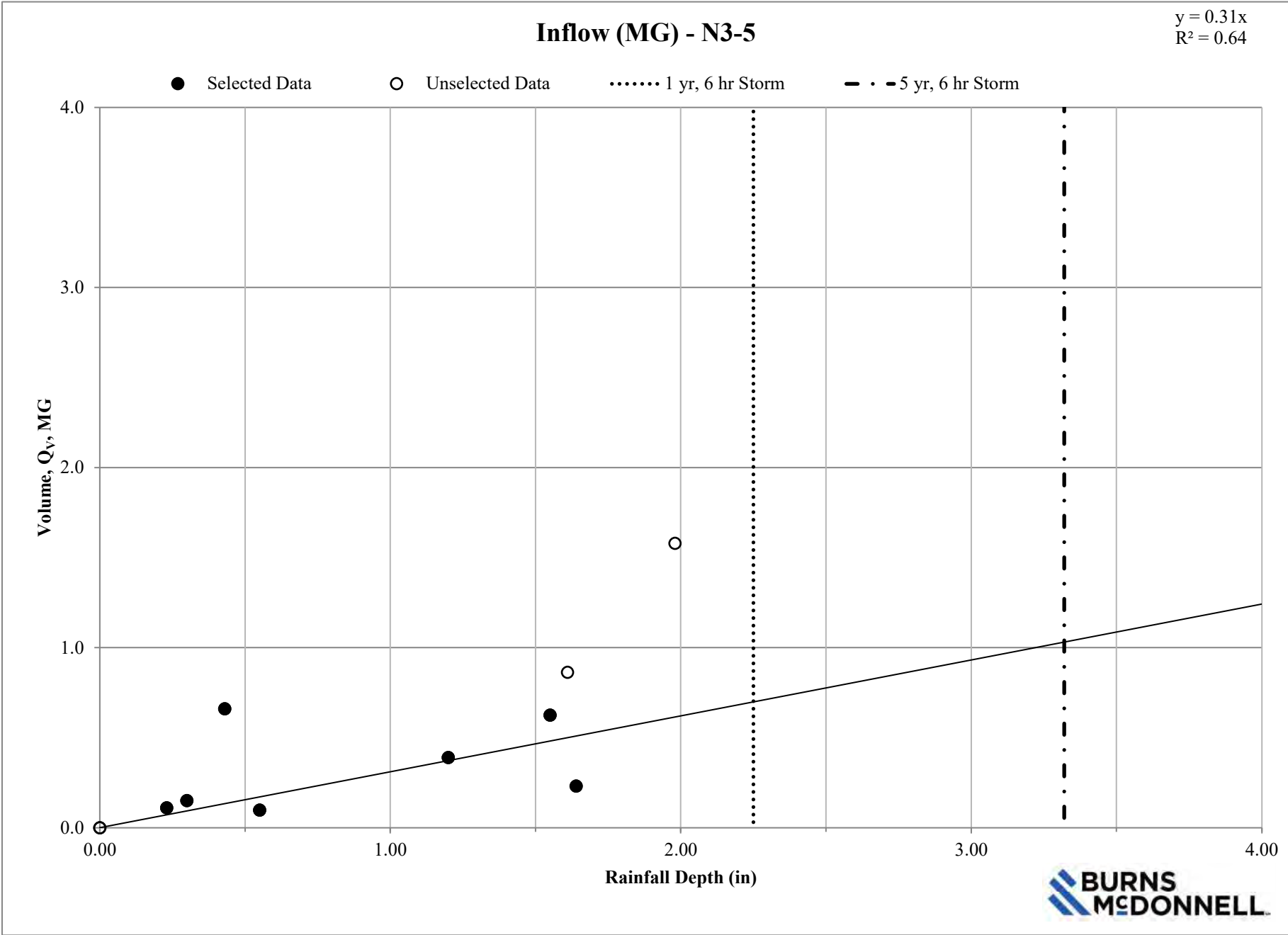


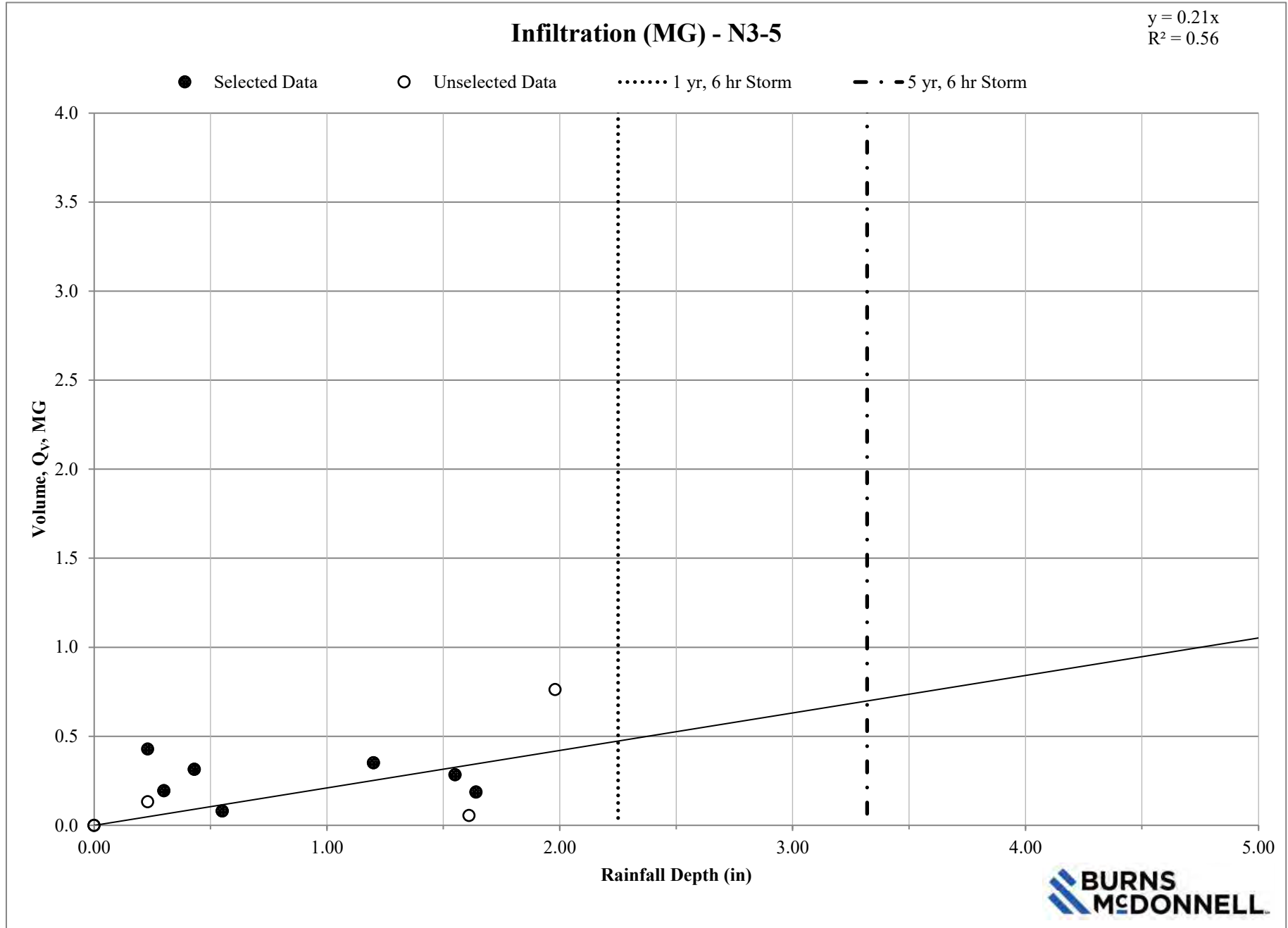


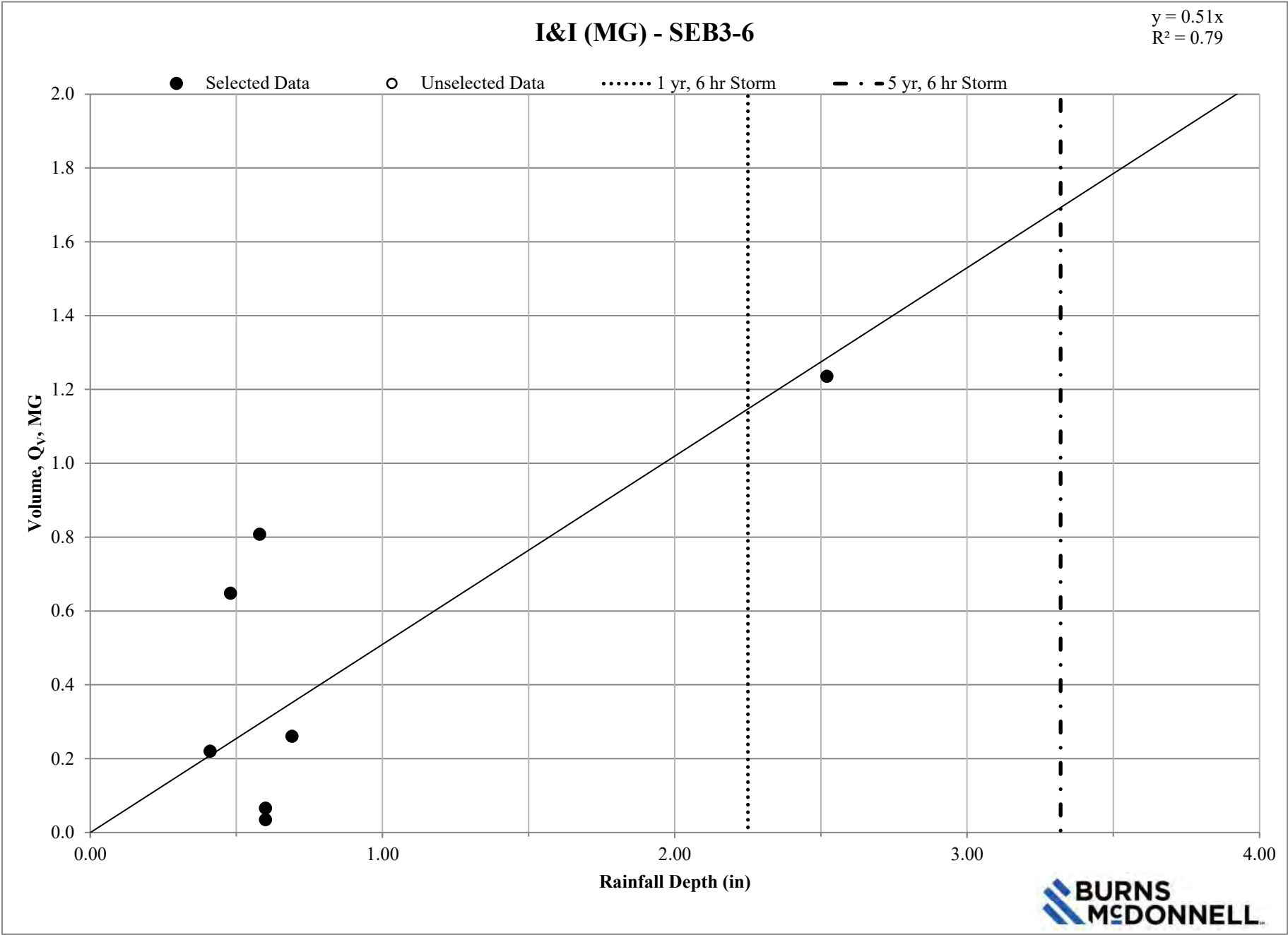


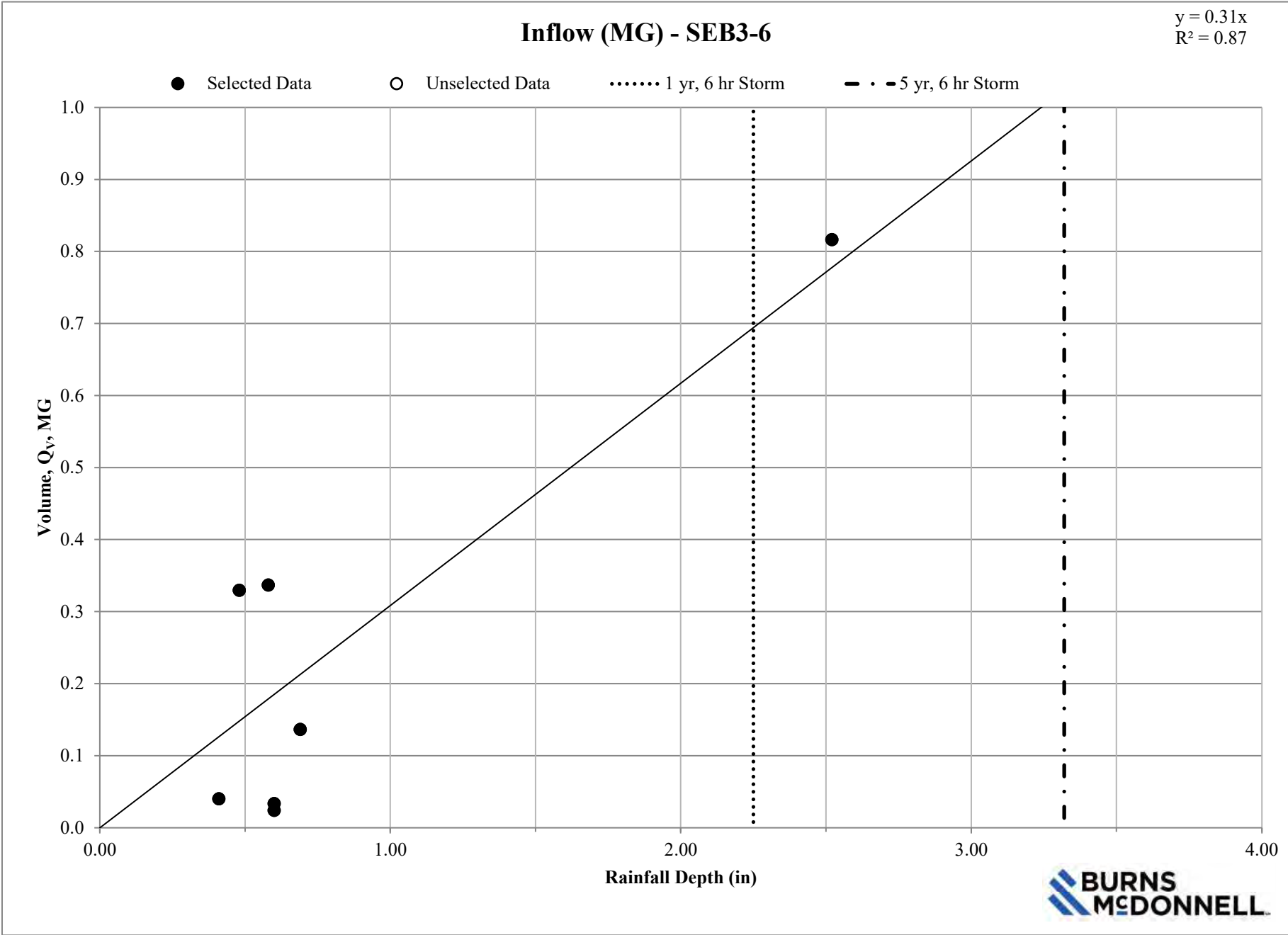


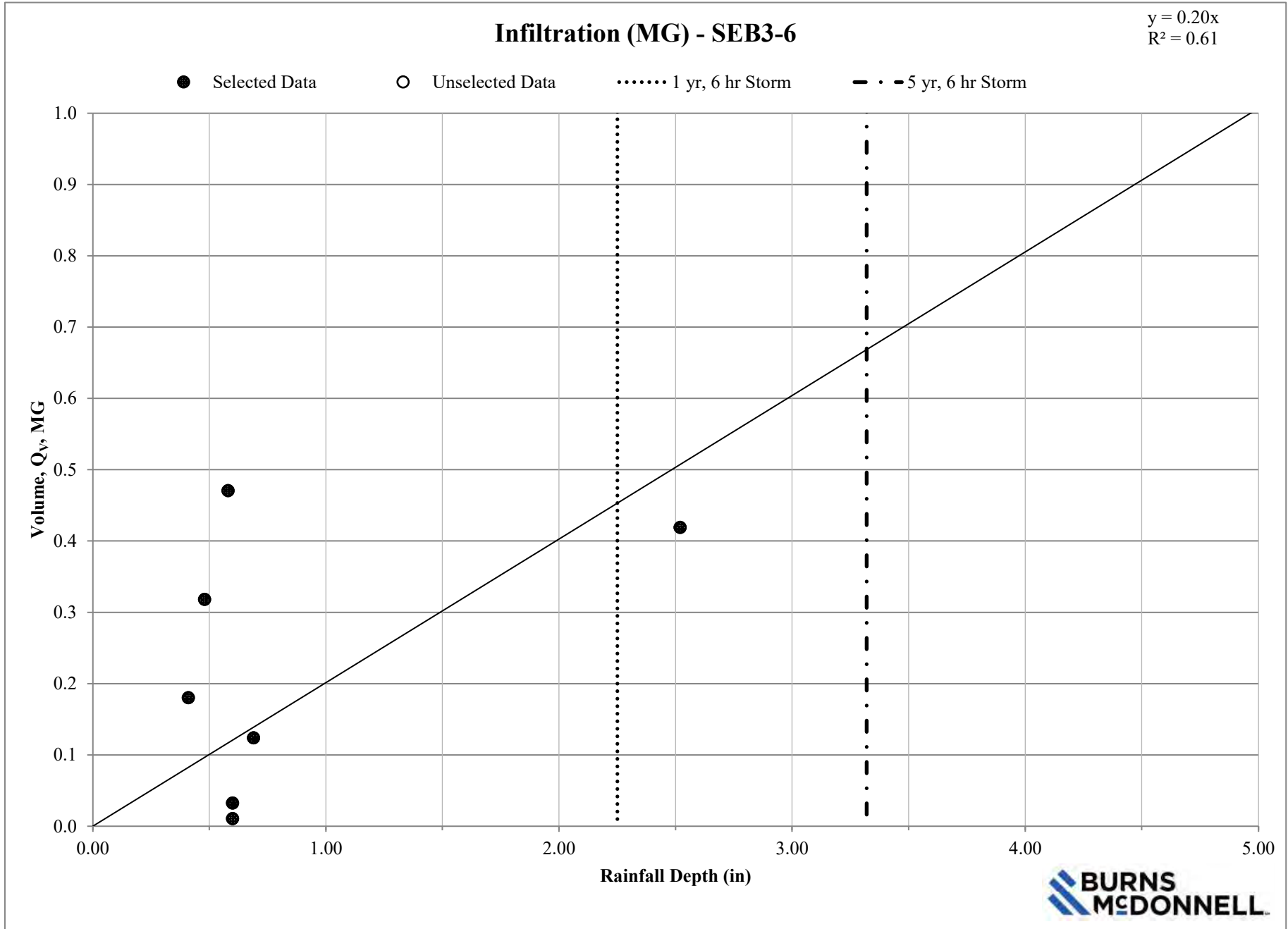


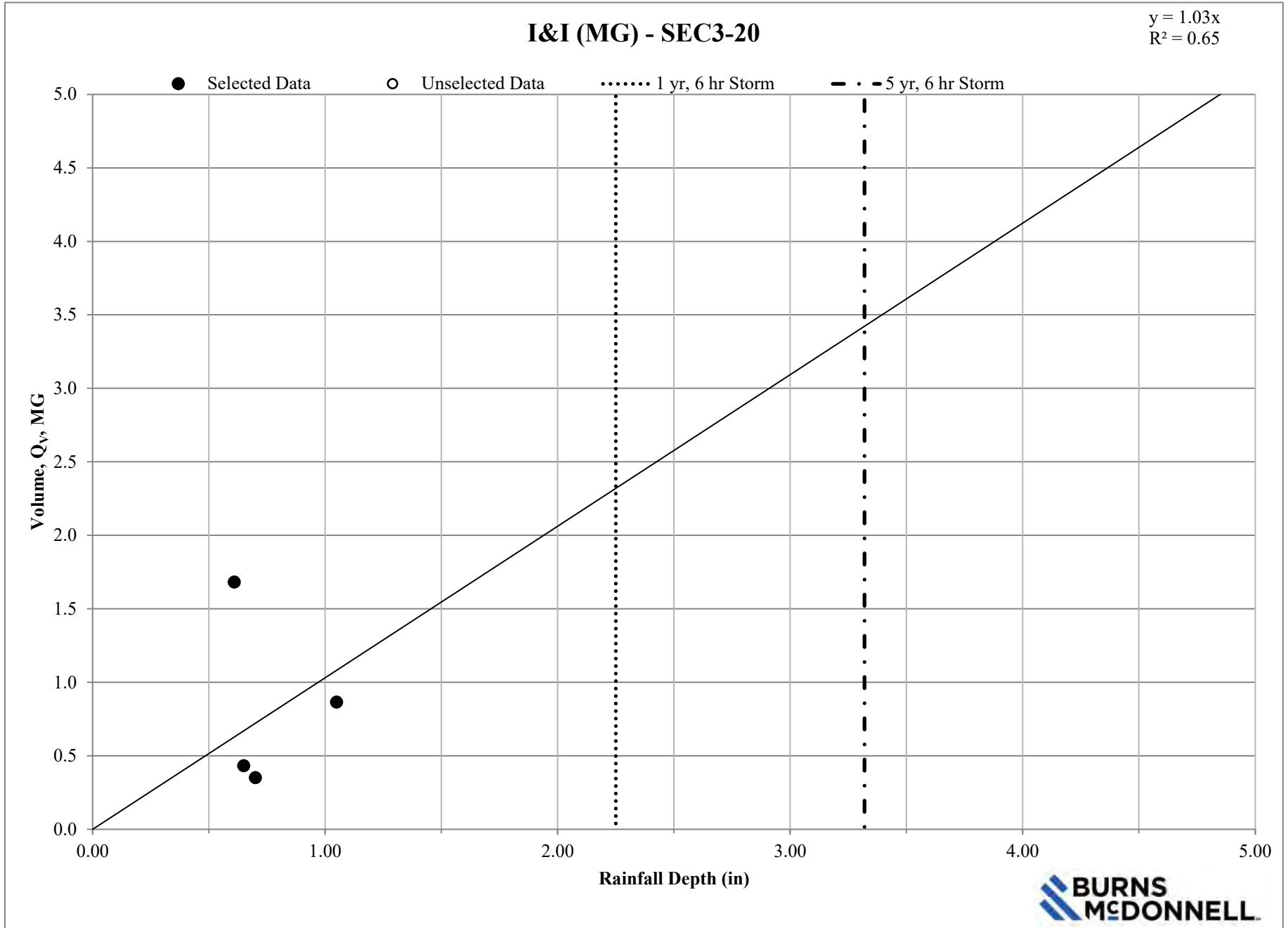


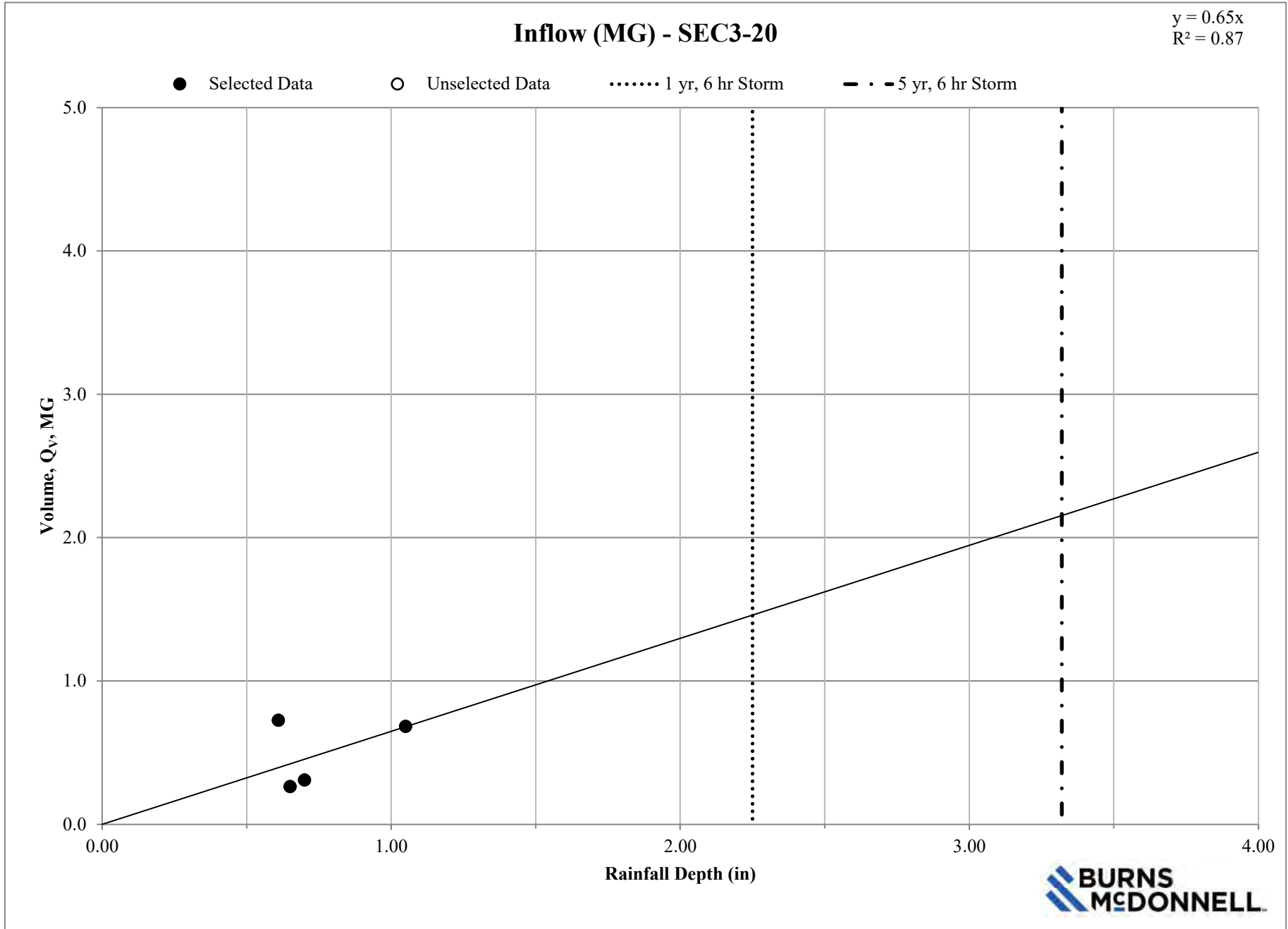


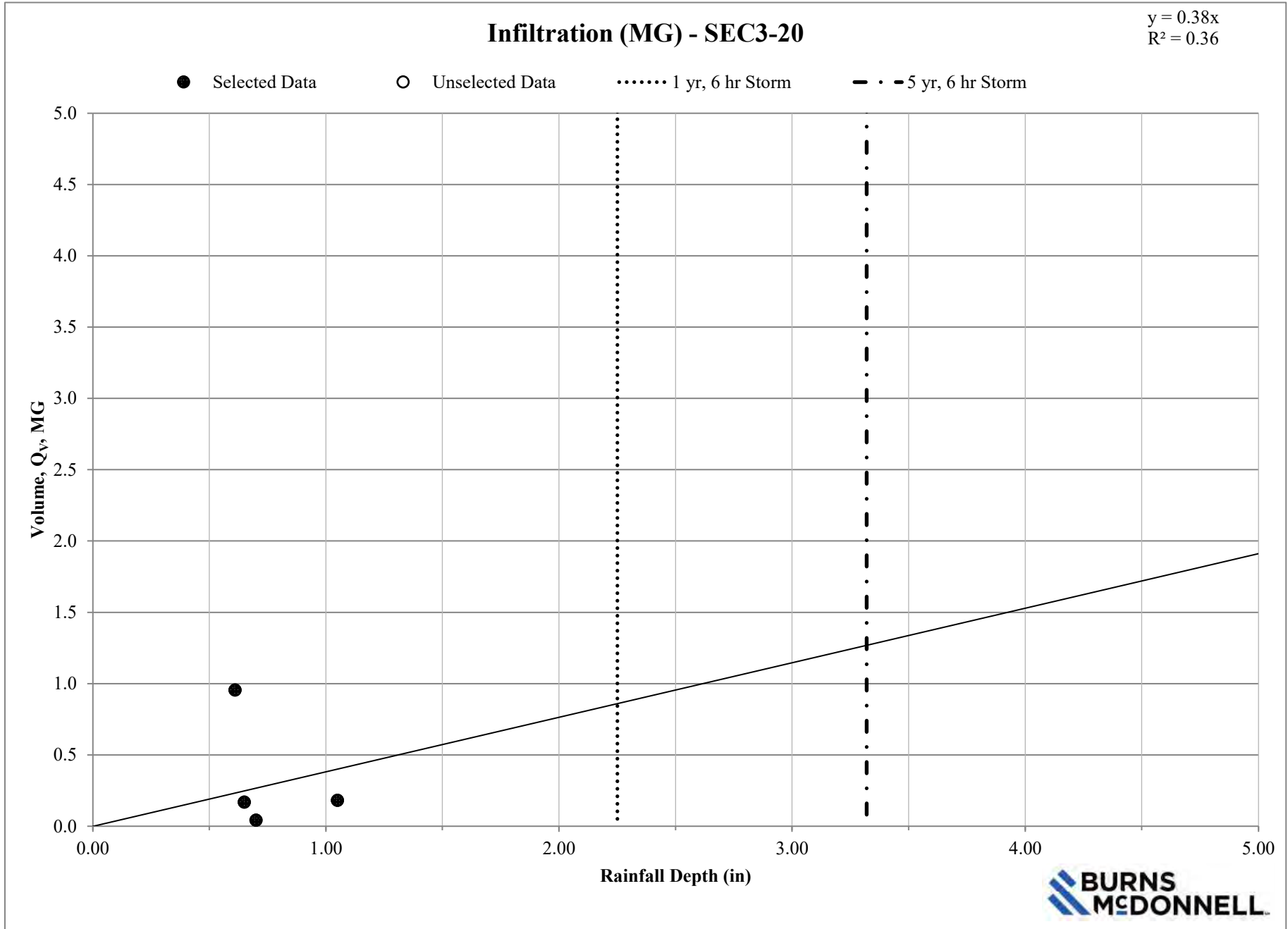


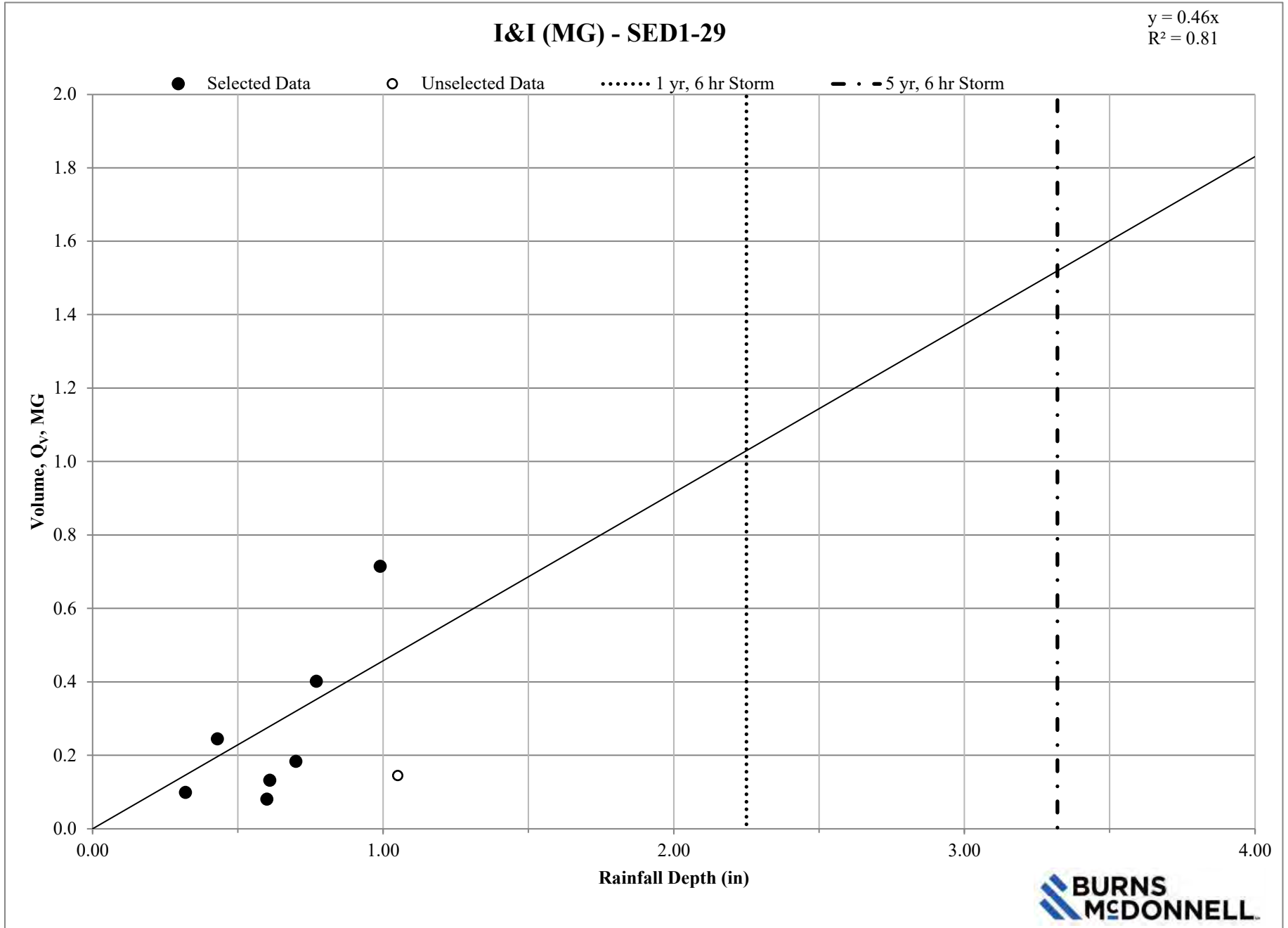


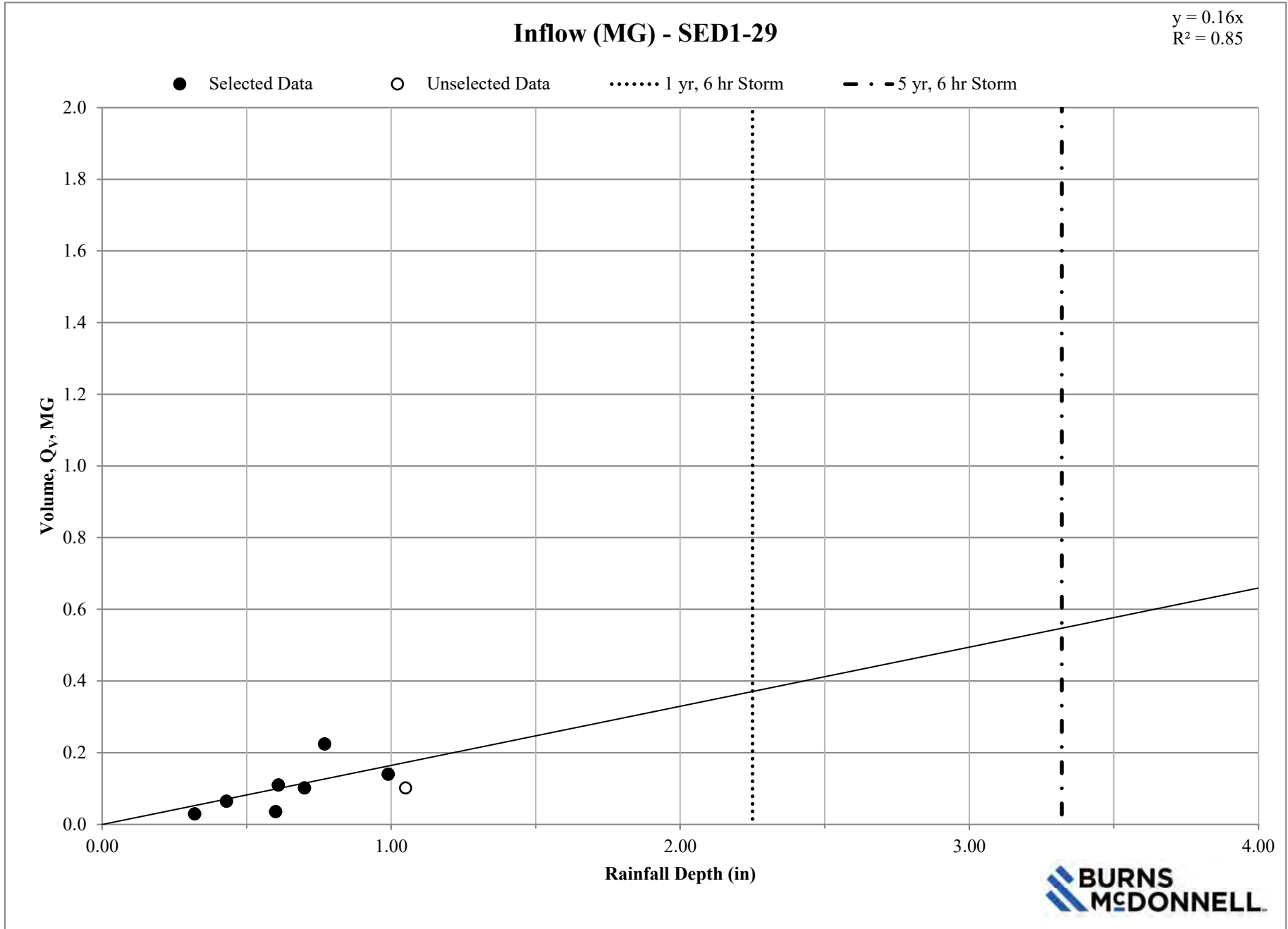


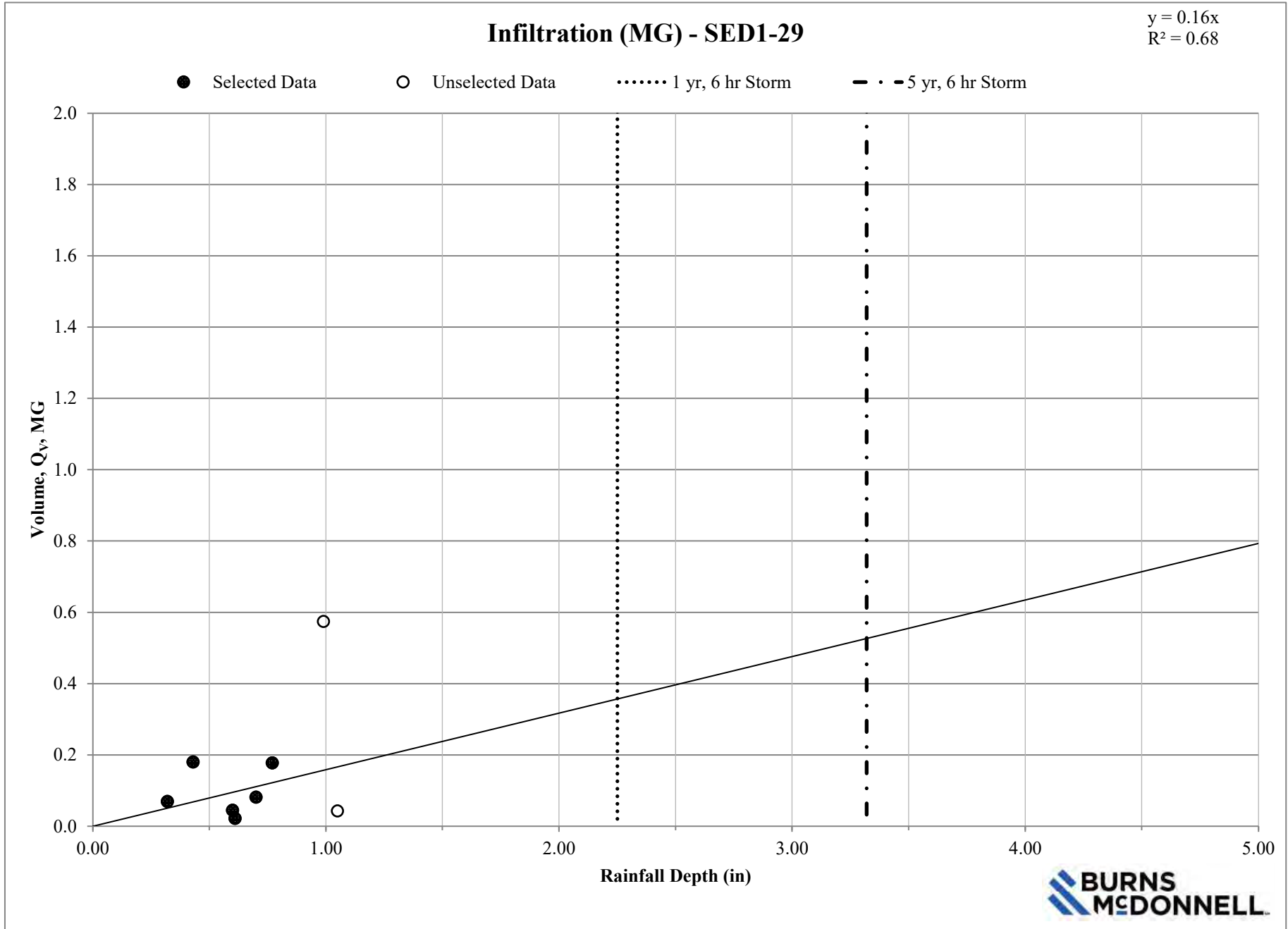


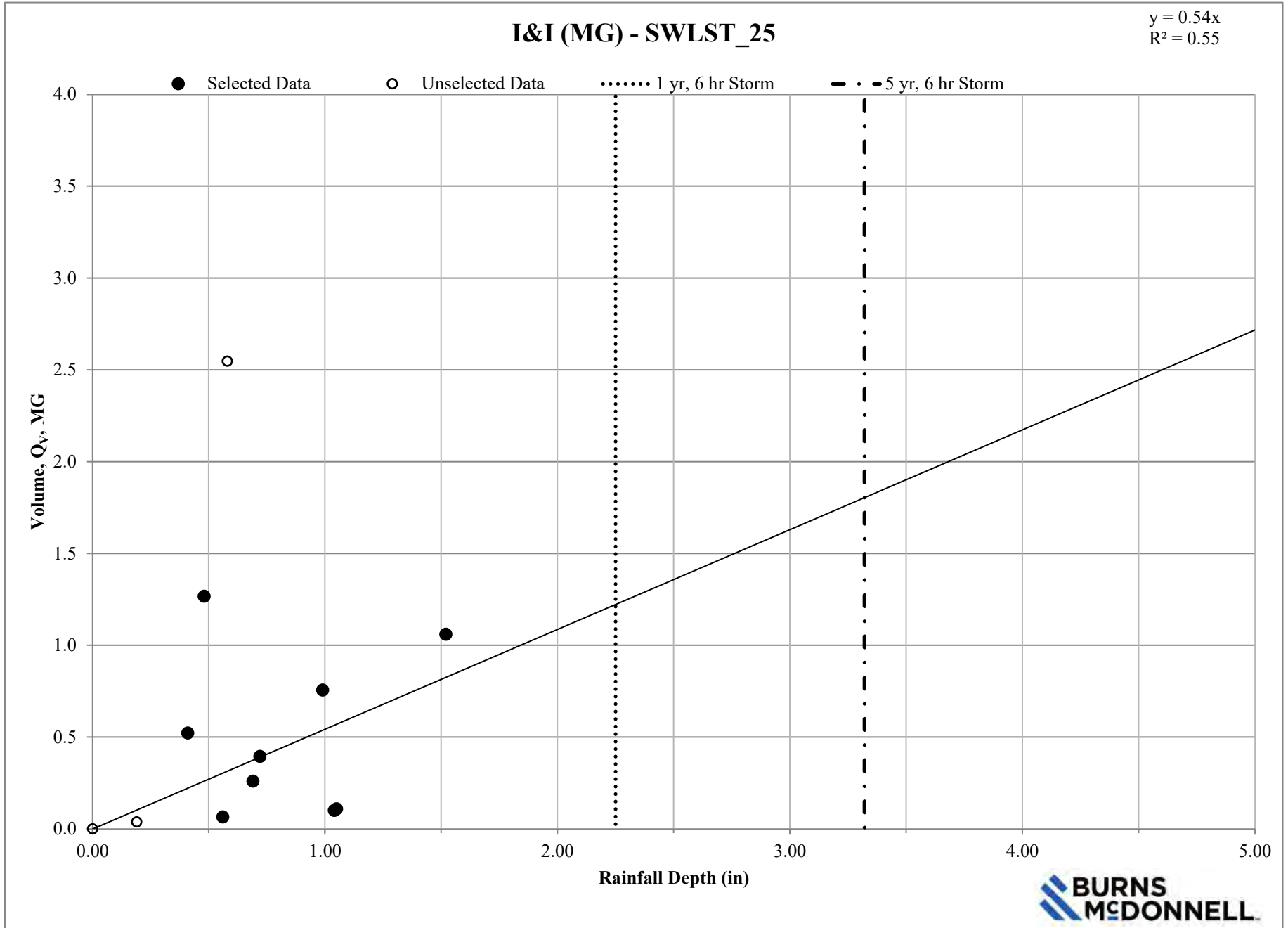


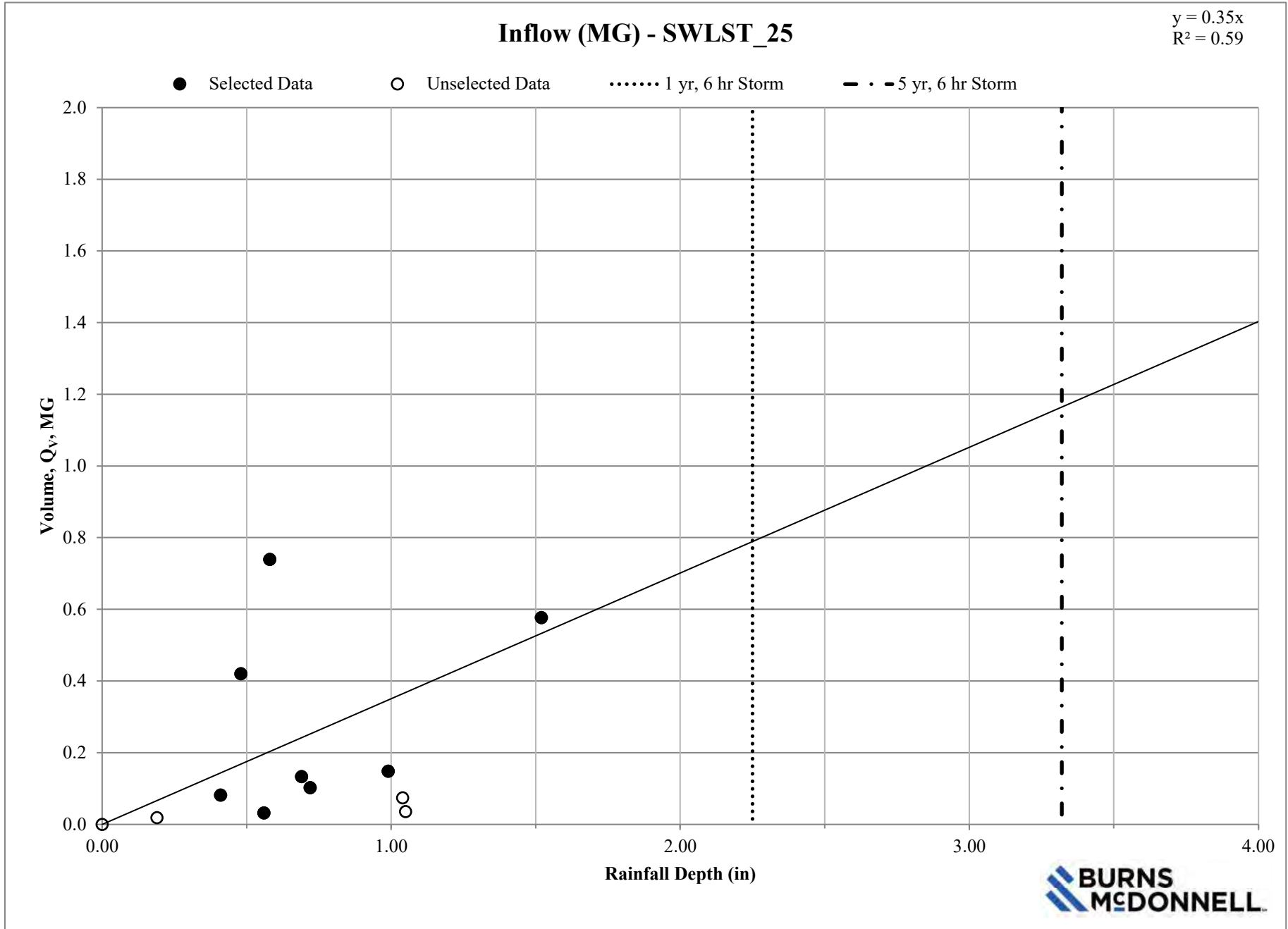


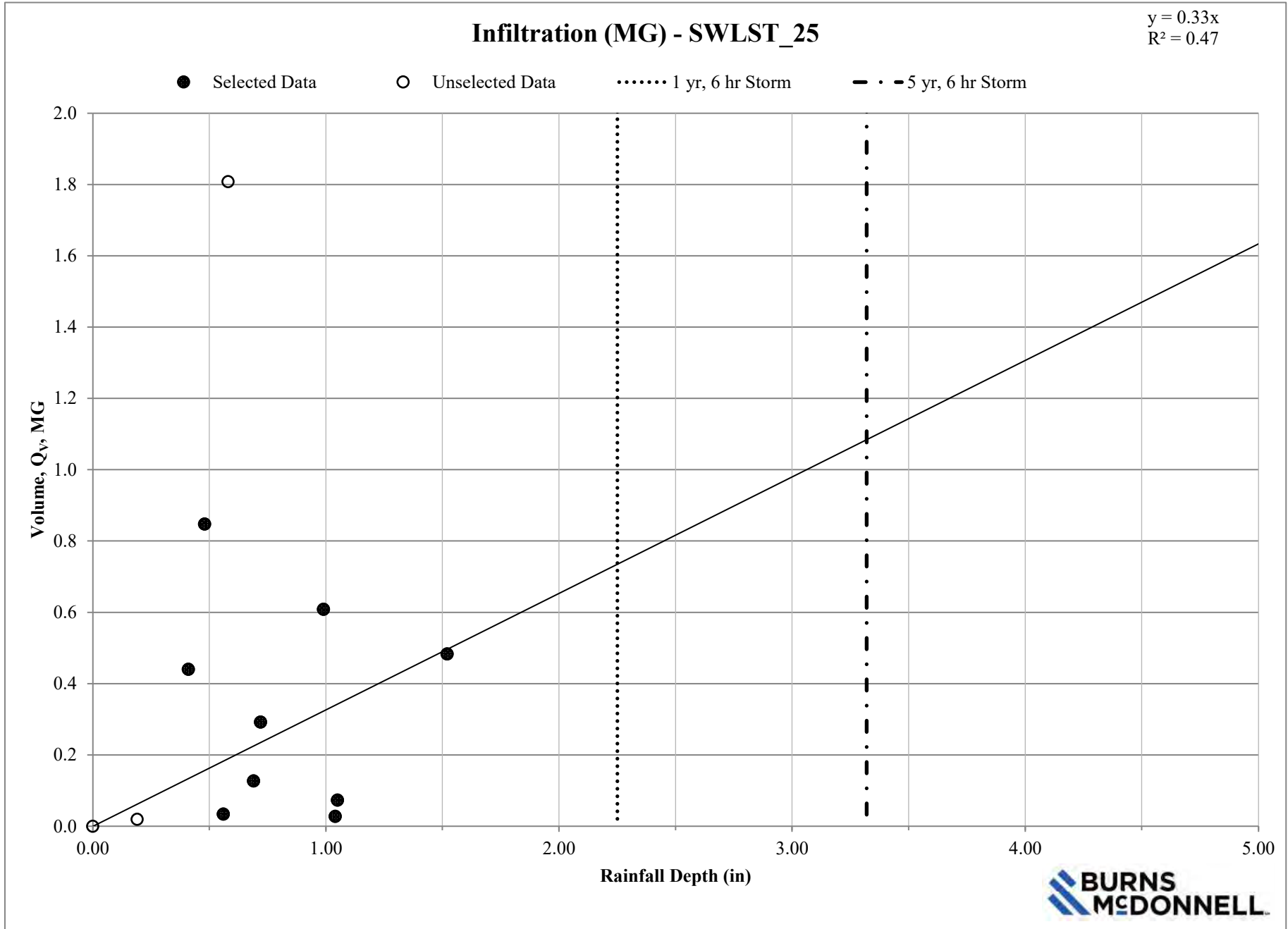






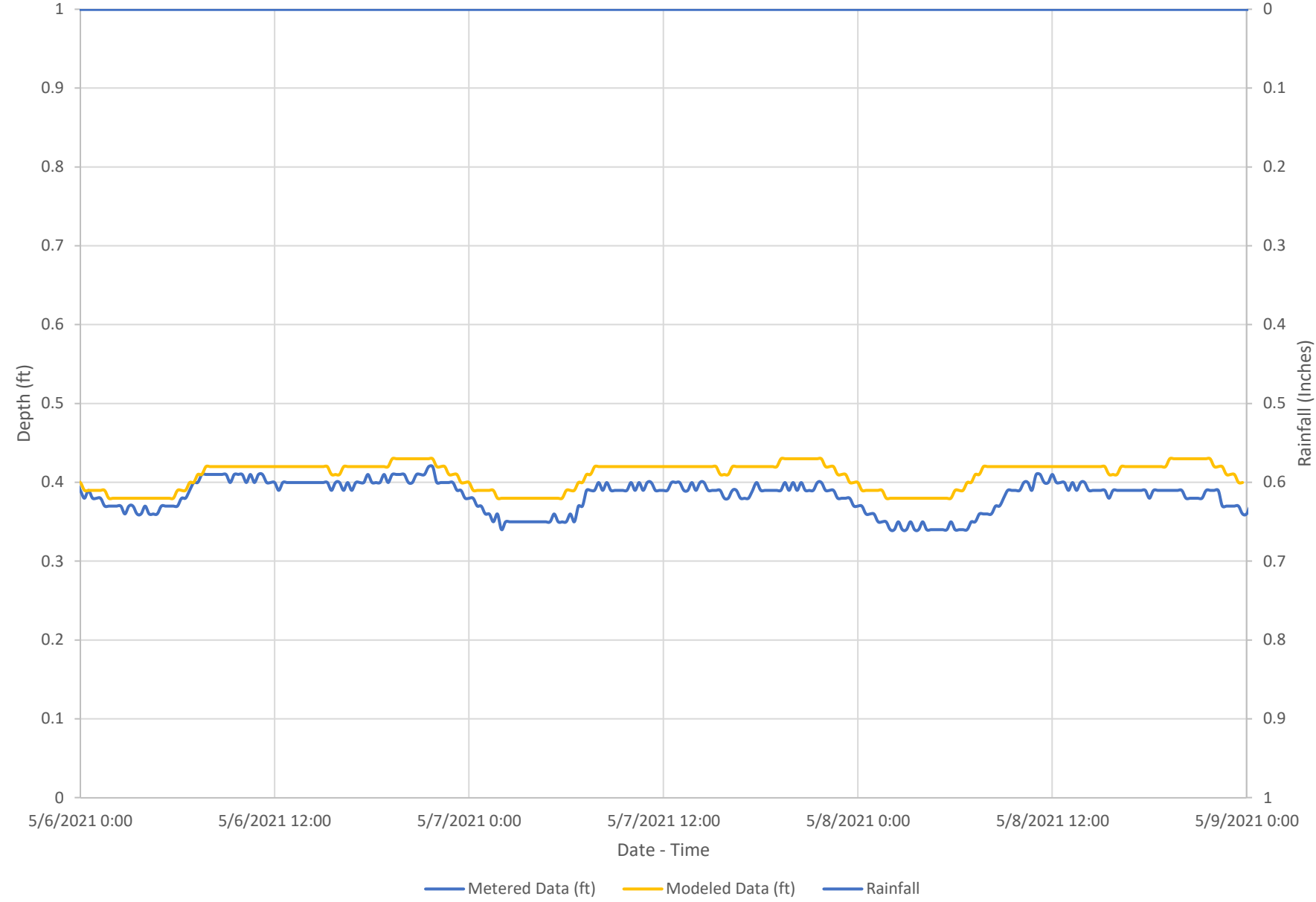




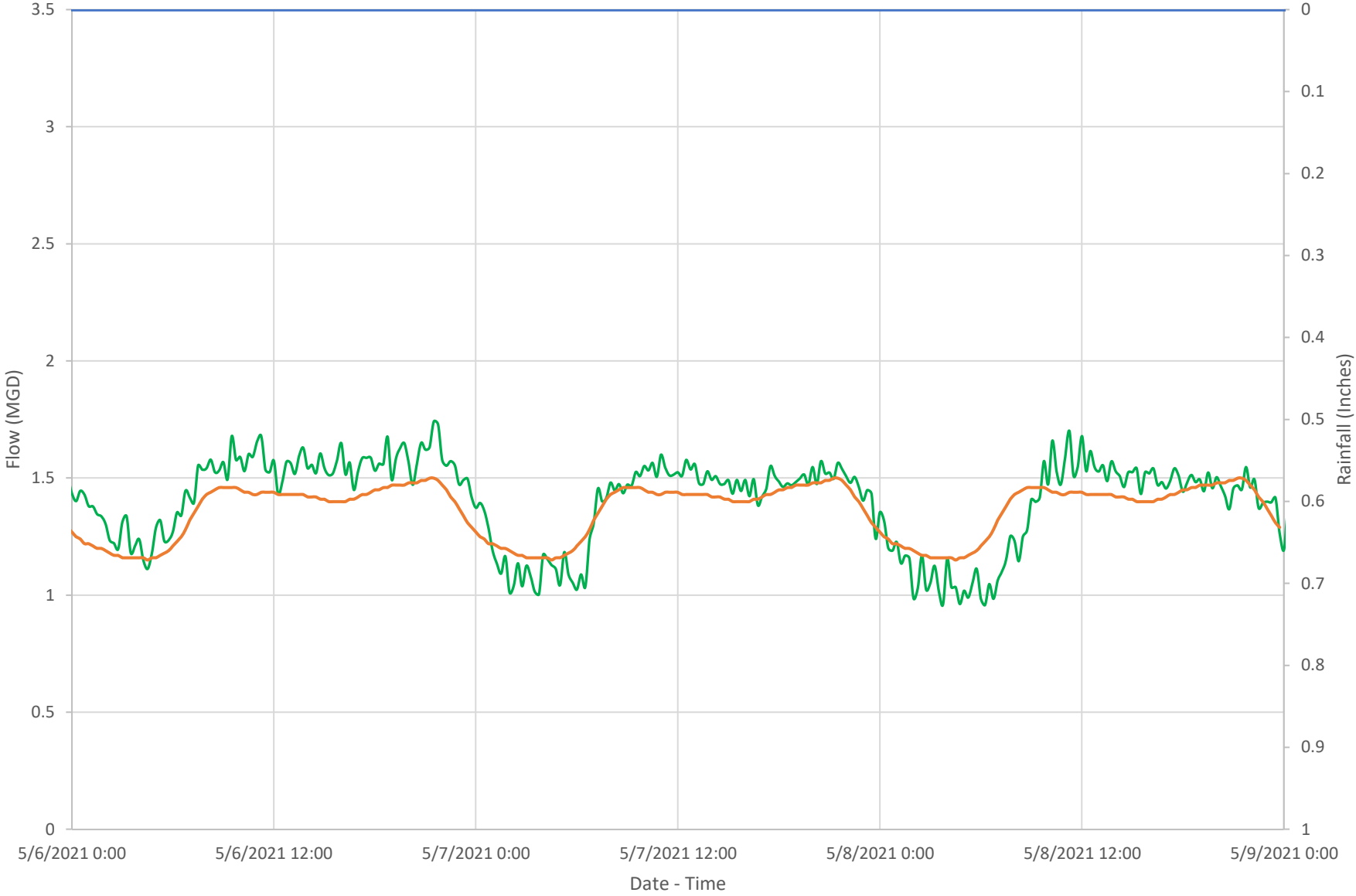


APPENDIX C – DRY WEATHER FLOW CALIBRATION GRAPHS

CE1-30

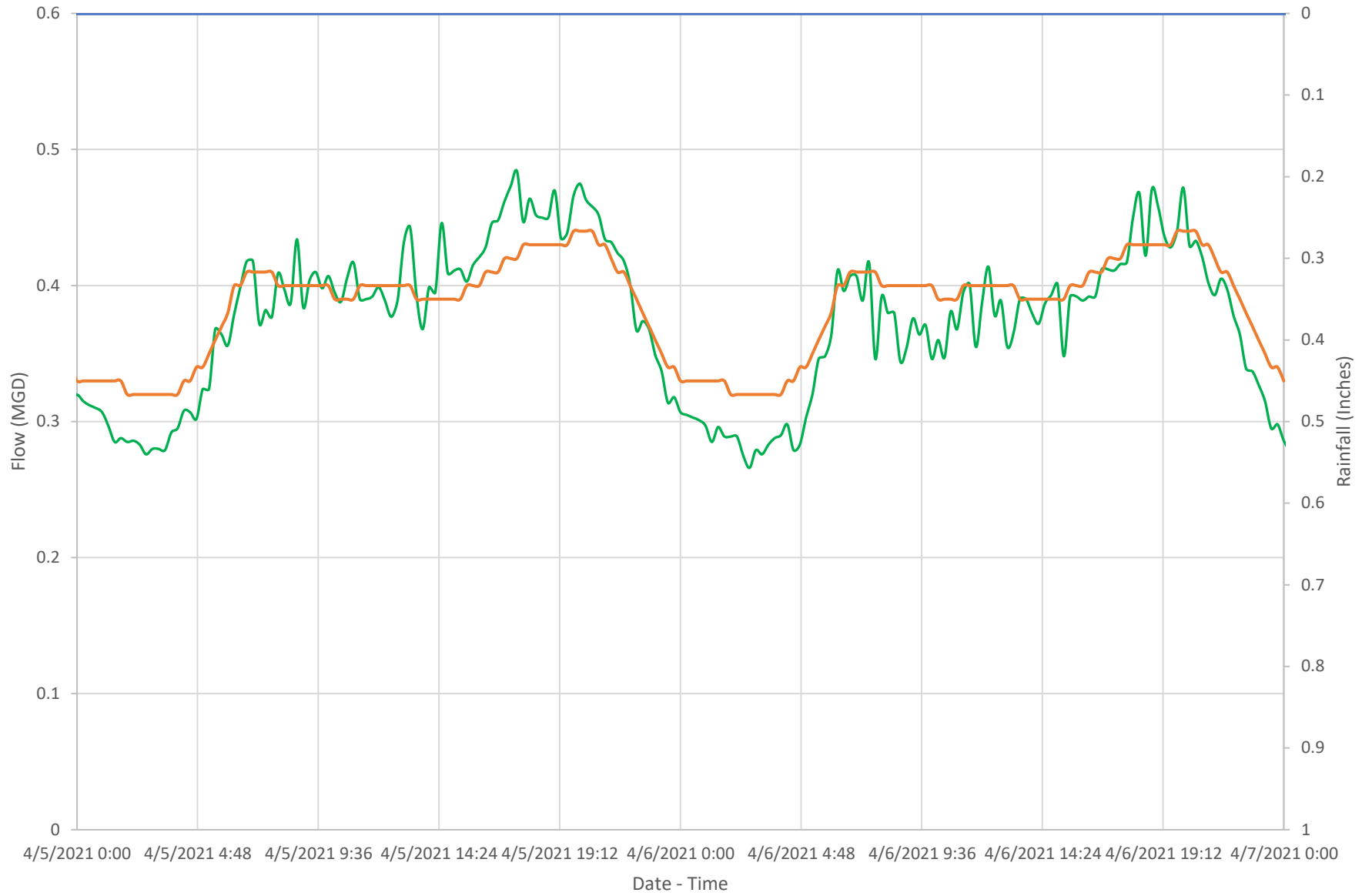


CE1-30



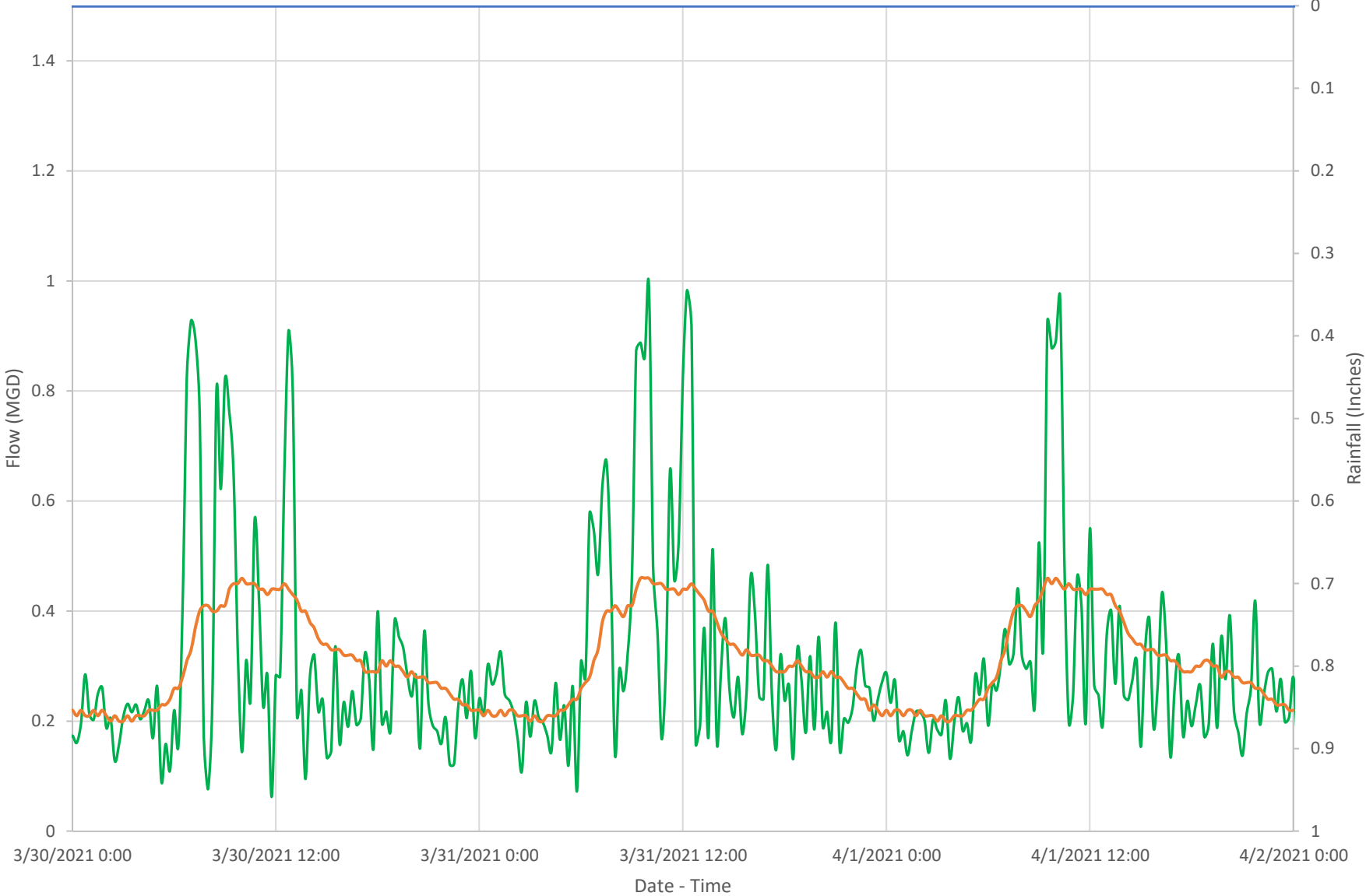
Metered Data (MGD) Modeled Data (MGD) Rainfall

CE7-1



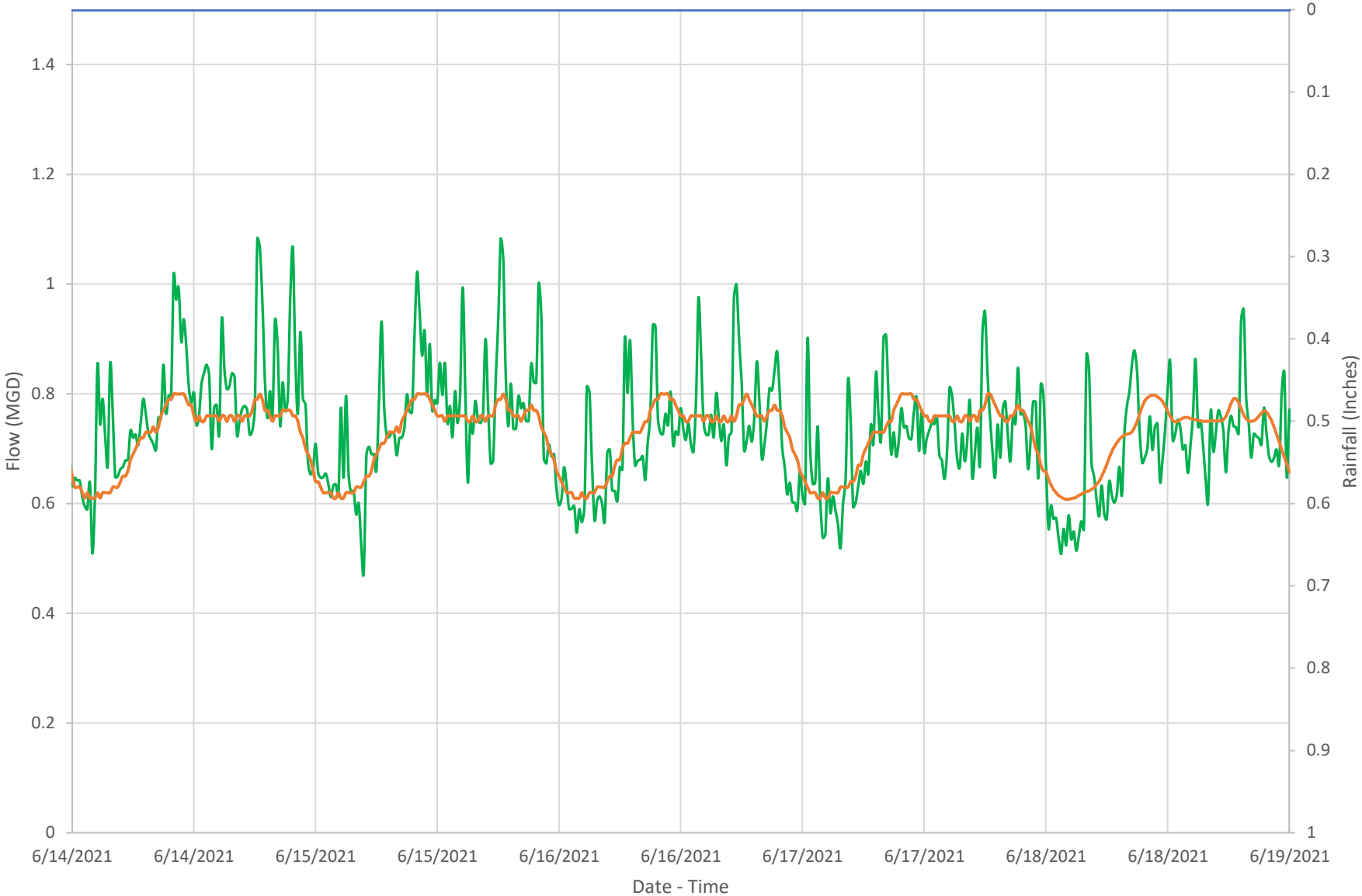
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CW1-3



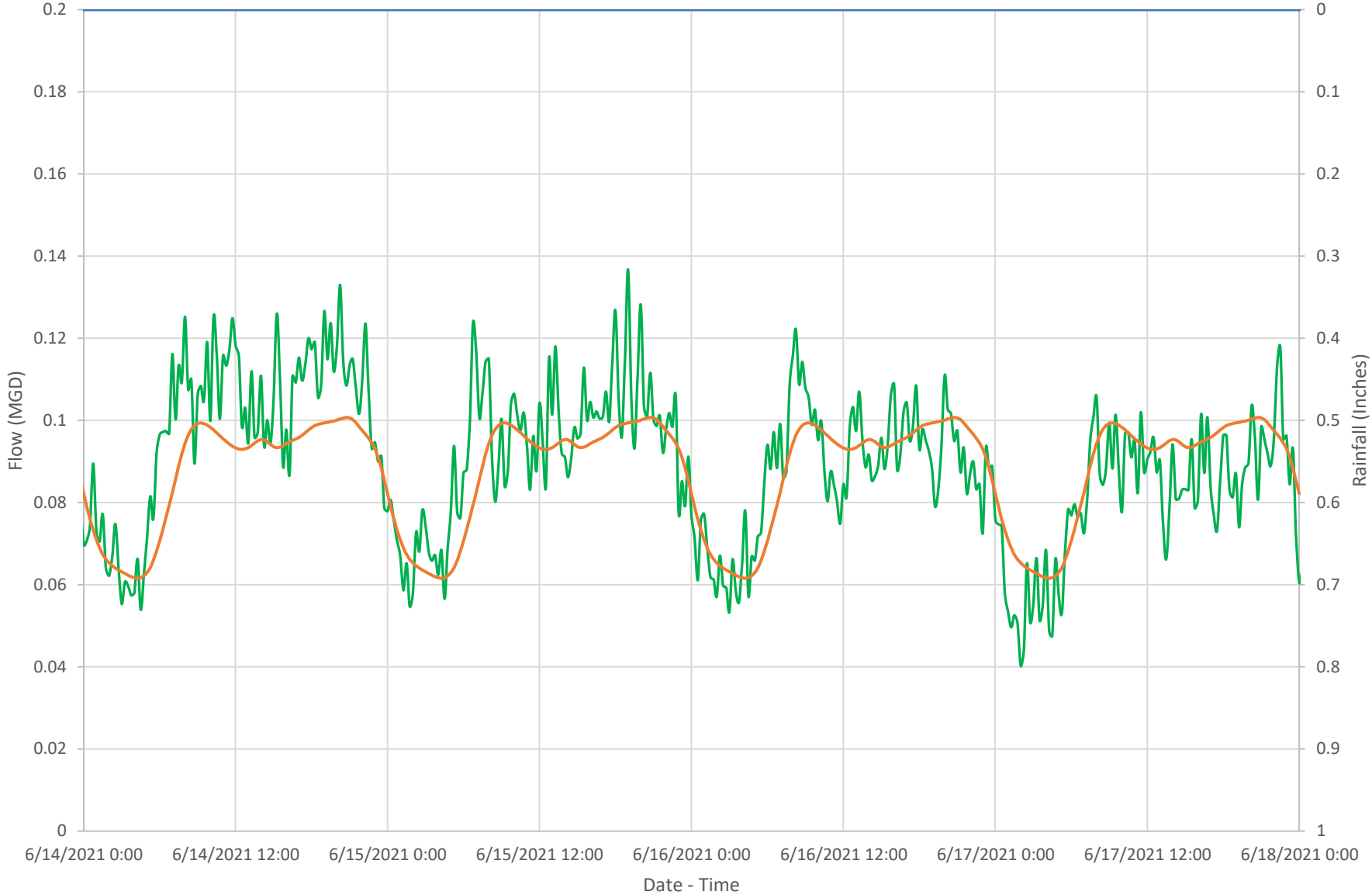
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N1-31



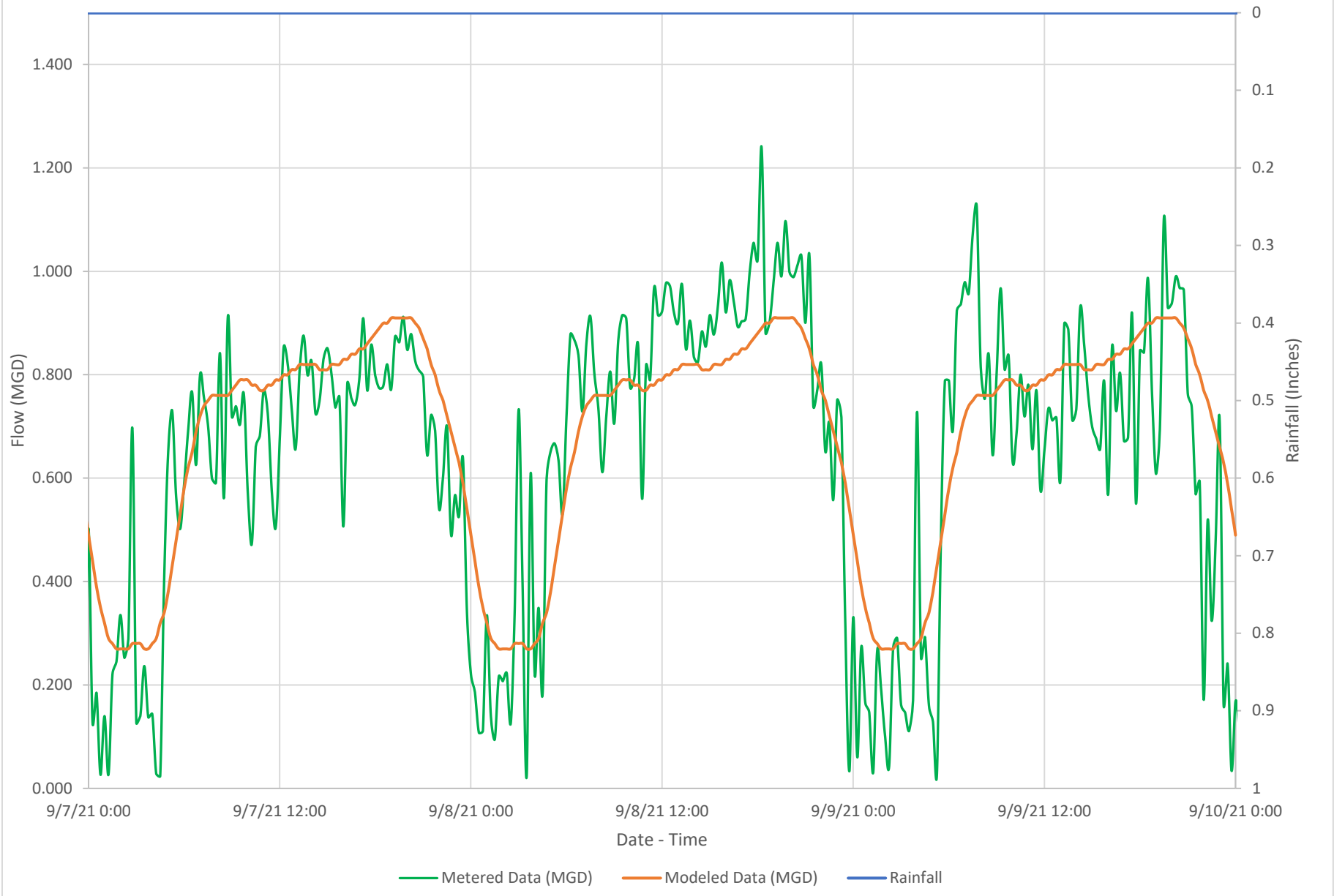
Metered Data (MGD) Modeled Data (MGD) Rainfall

SEB3-6

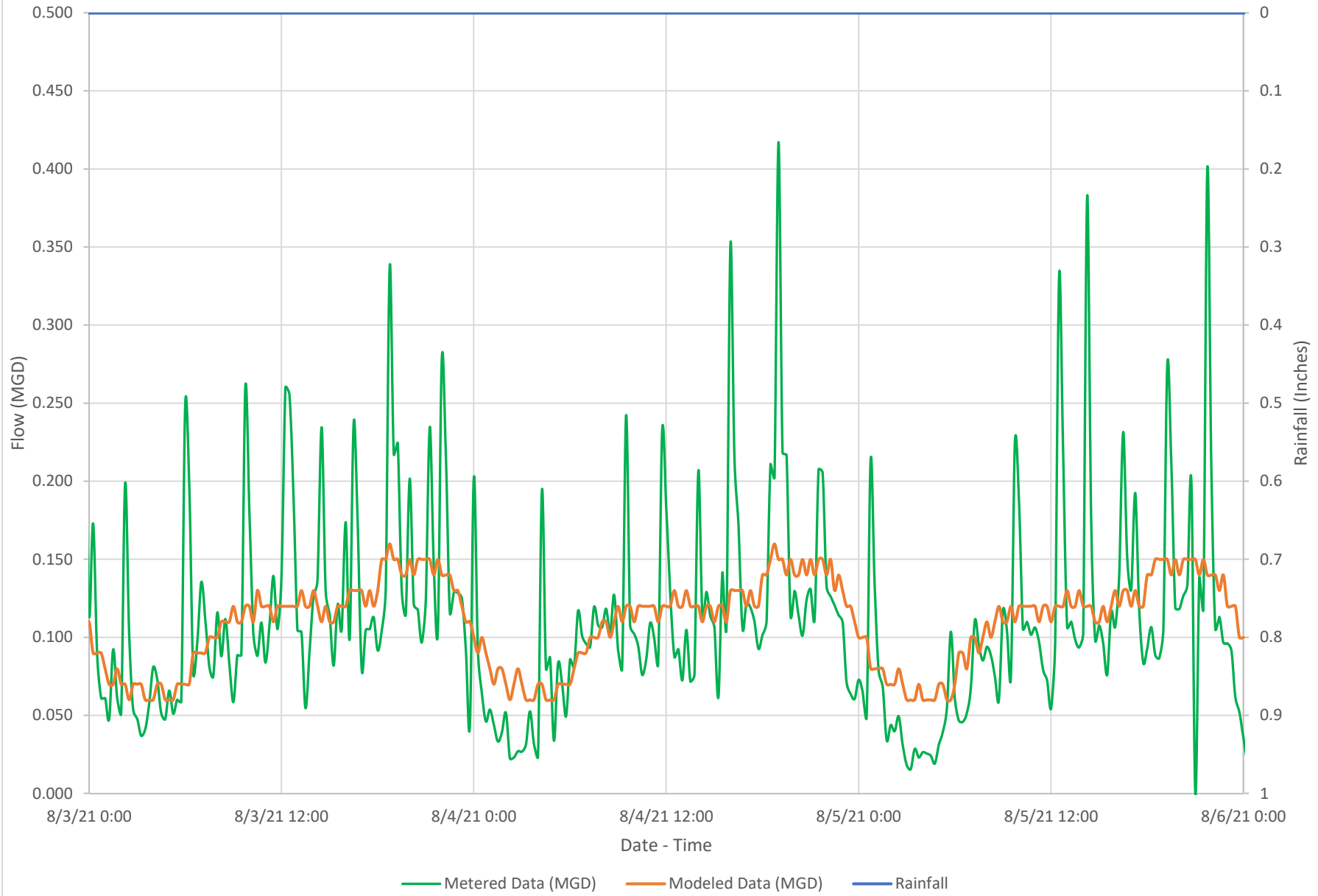


Metered Data (MGD) Modeled Data (MGD) Rainfall

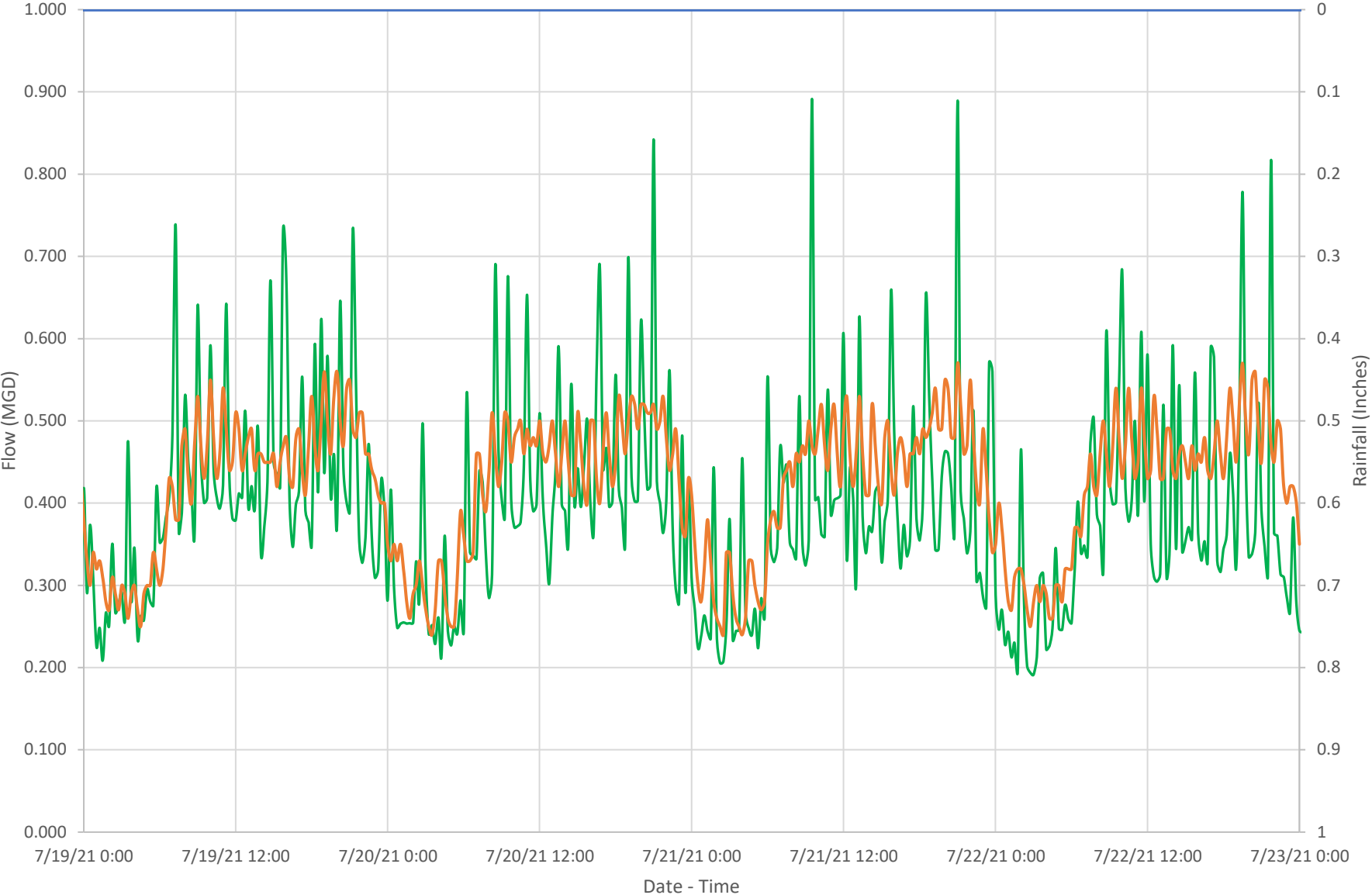
SEC3-20



SED1-29

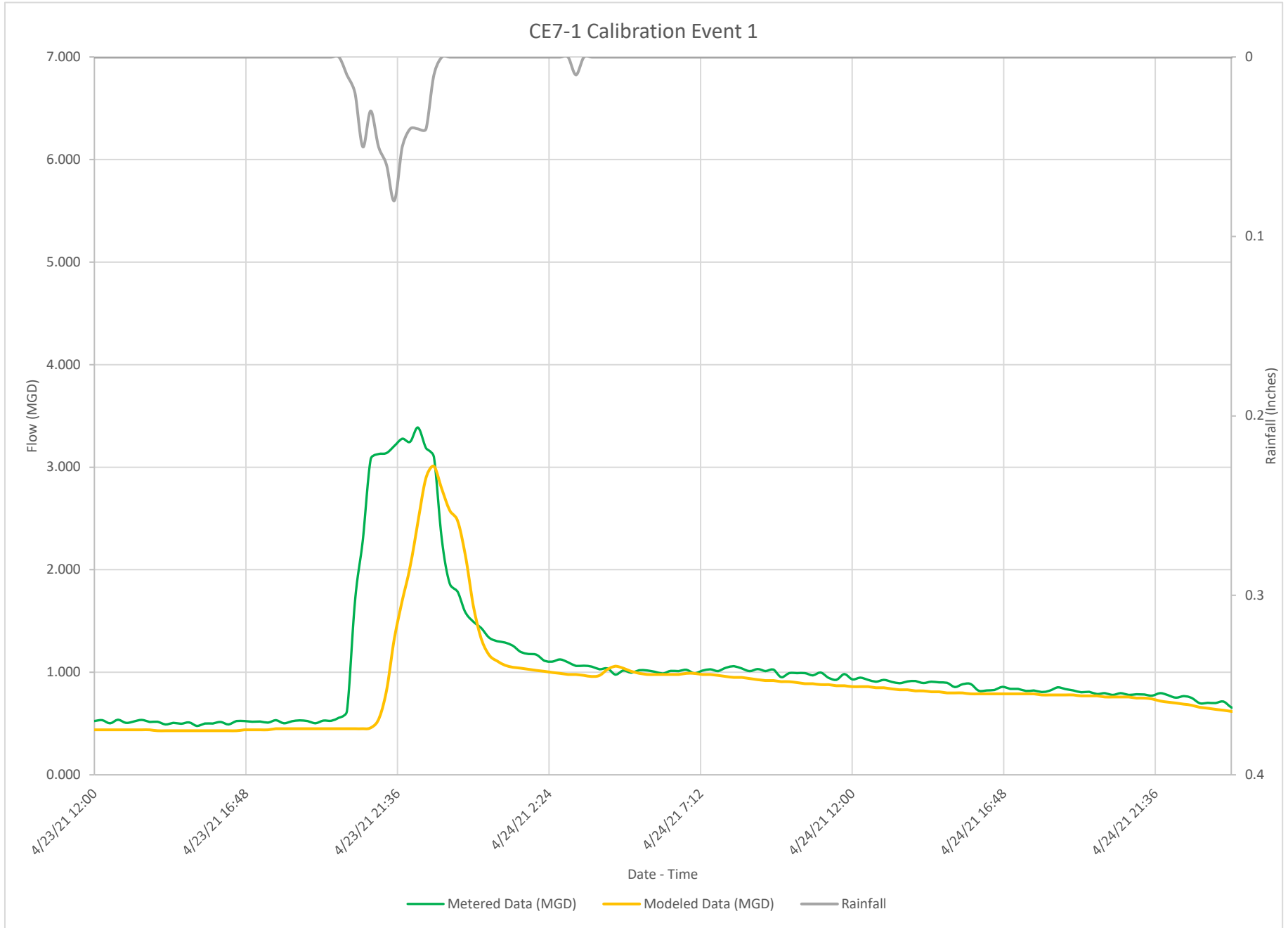


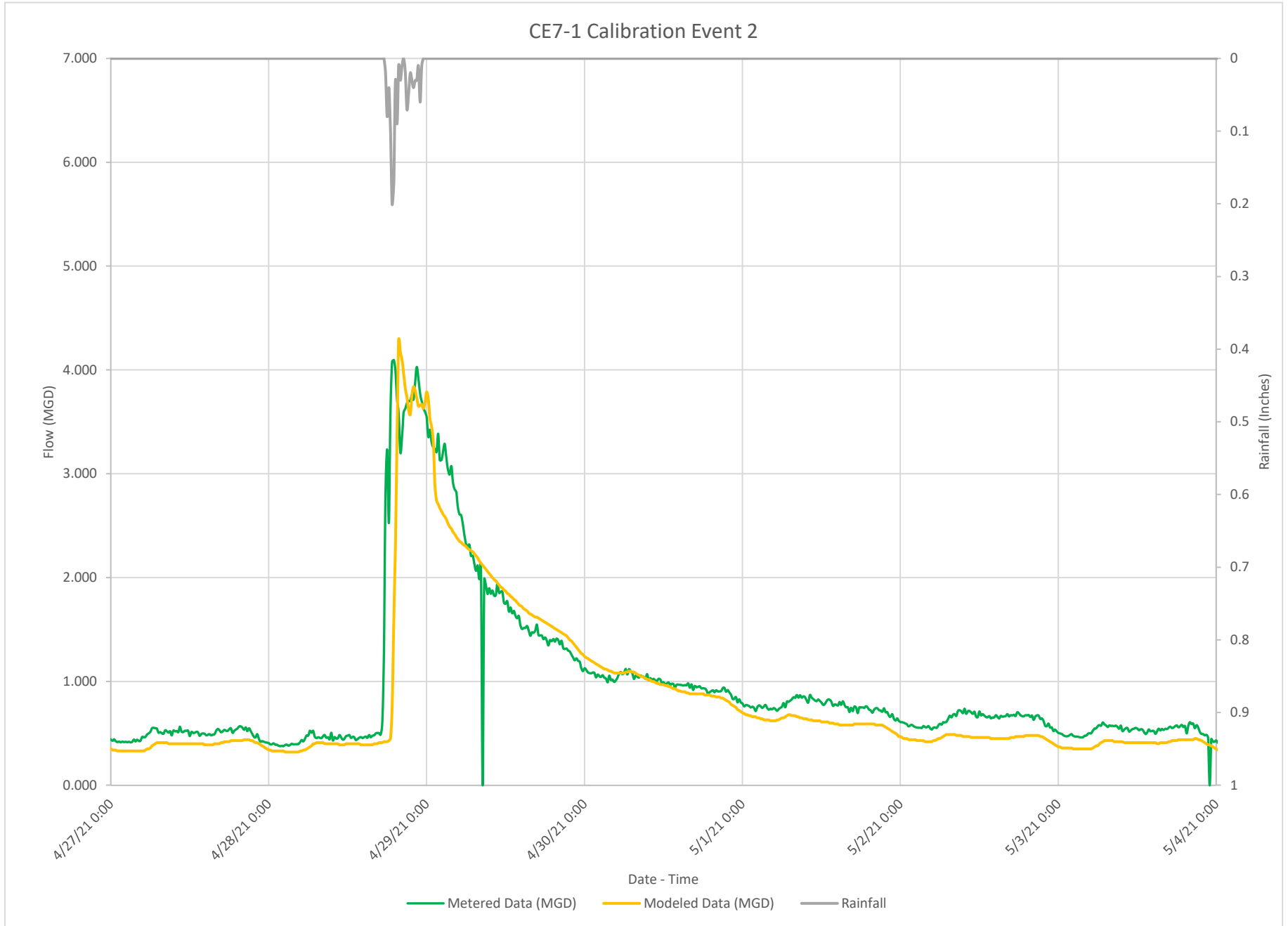
SWLST-25



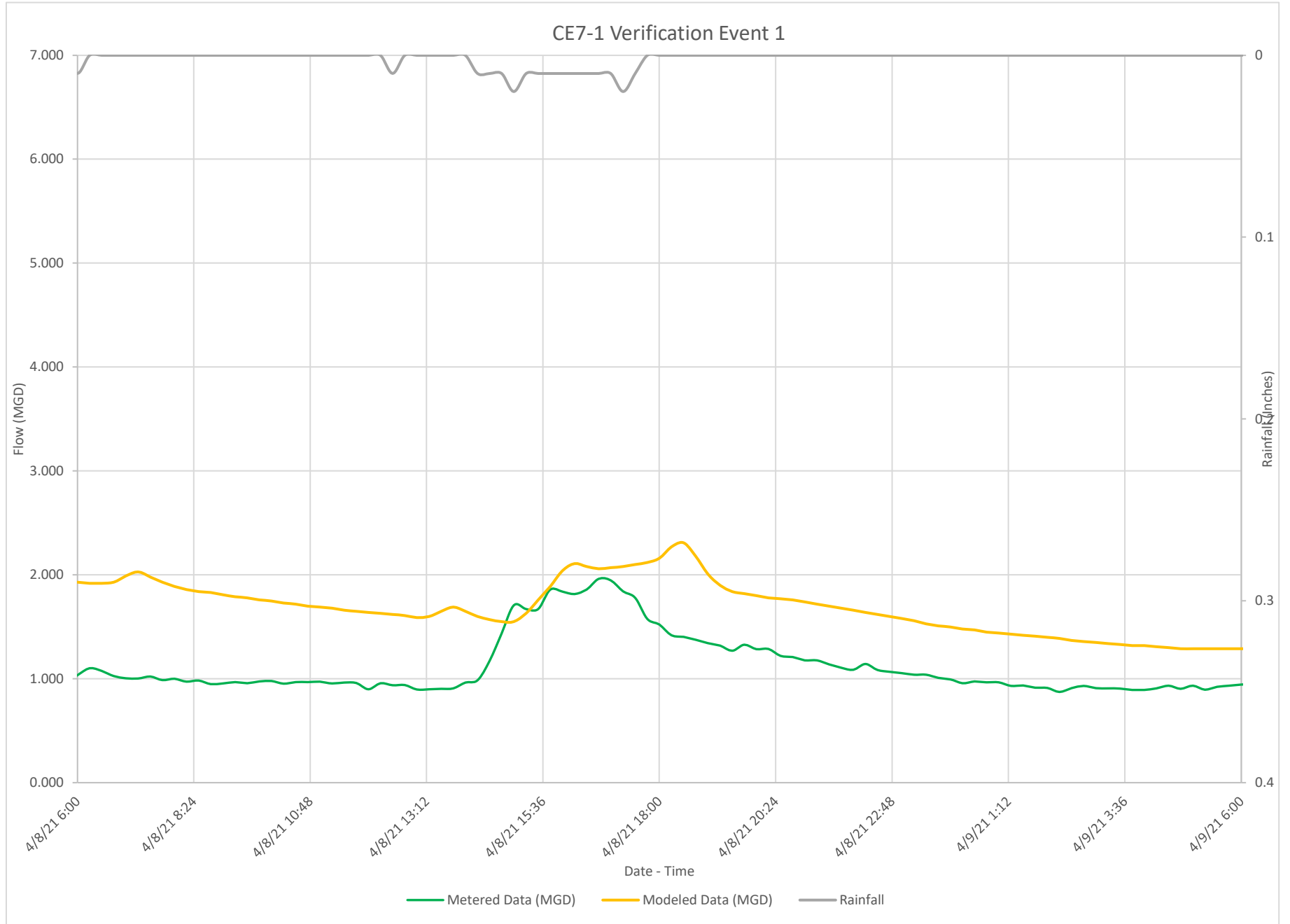
— Metered Data (MGD) — Modeled Data (MGD) — Rainfall

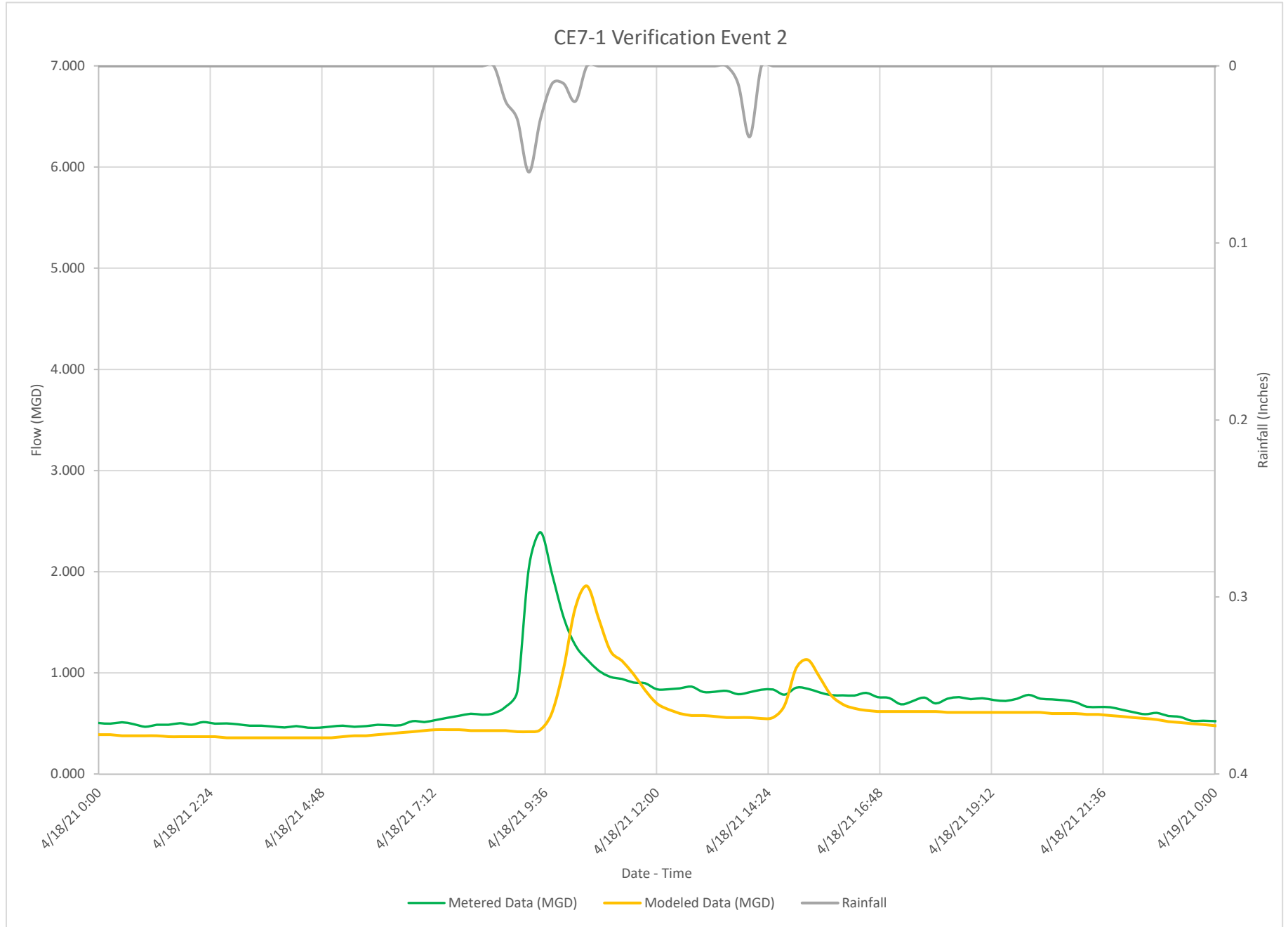
APPENDIX D – WET WEATHER FLOW CALIBRATION GRAPHS

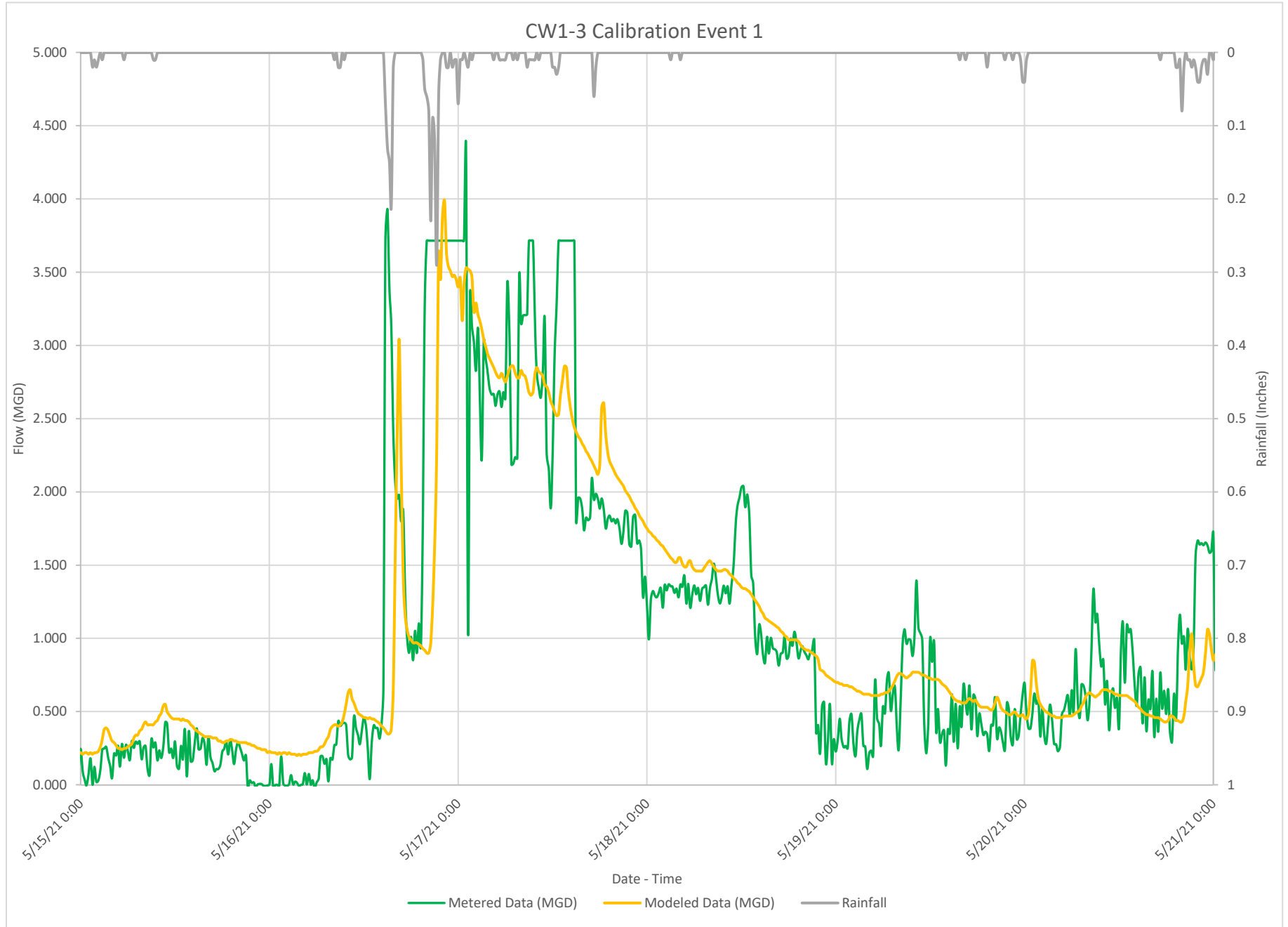


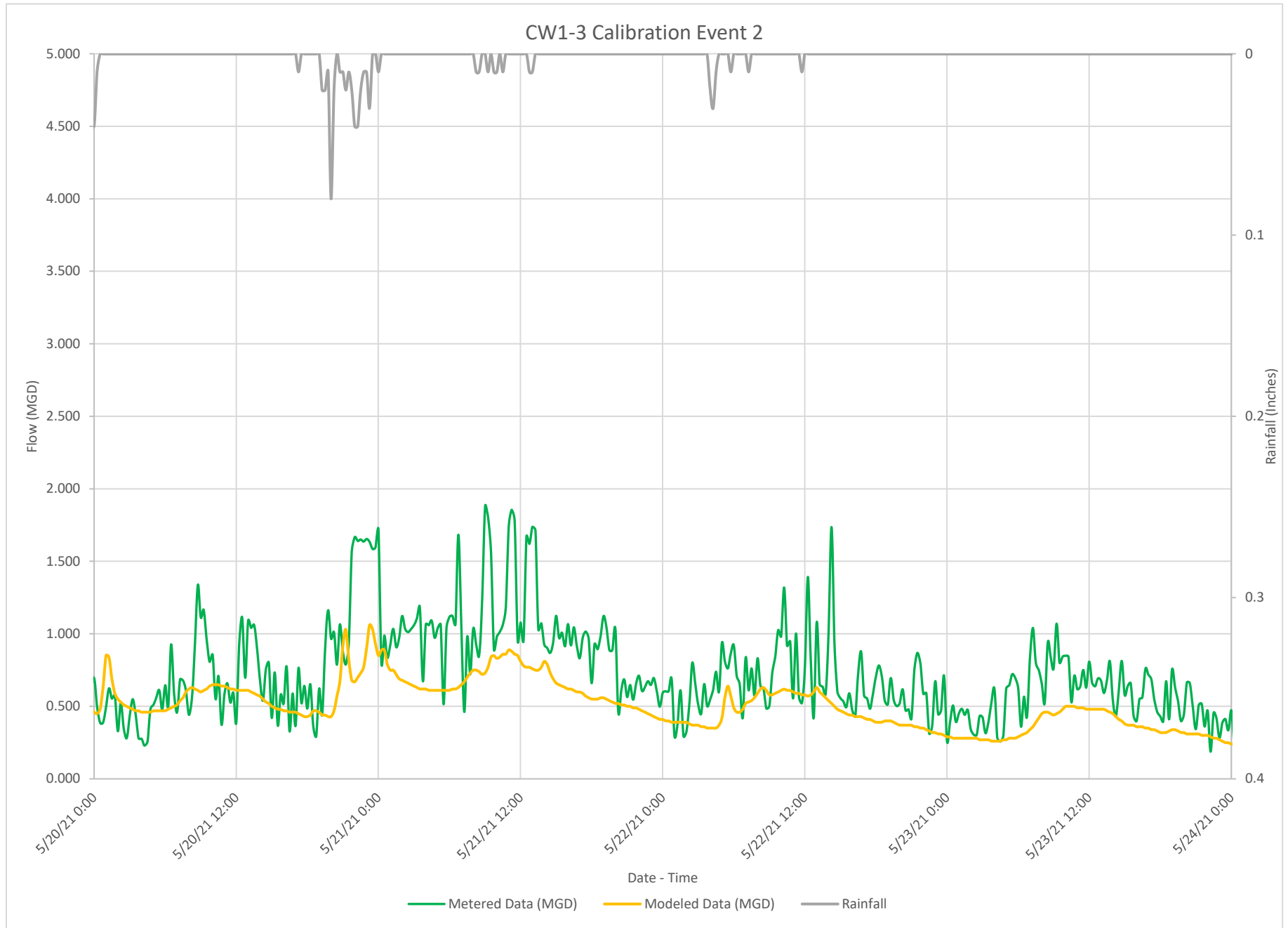


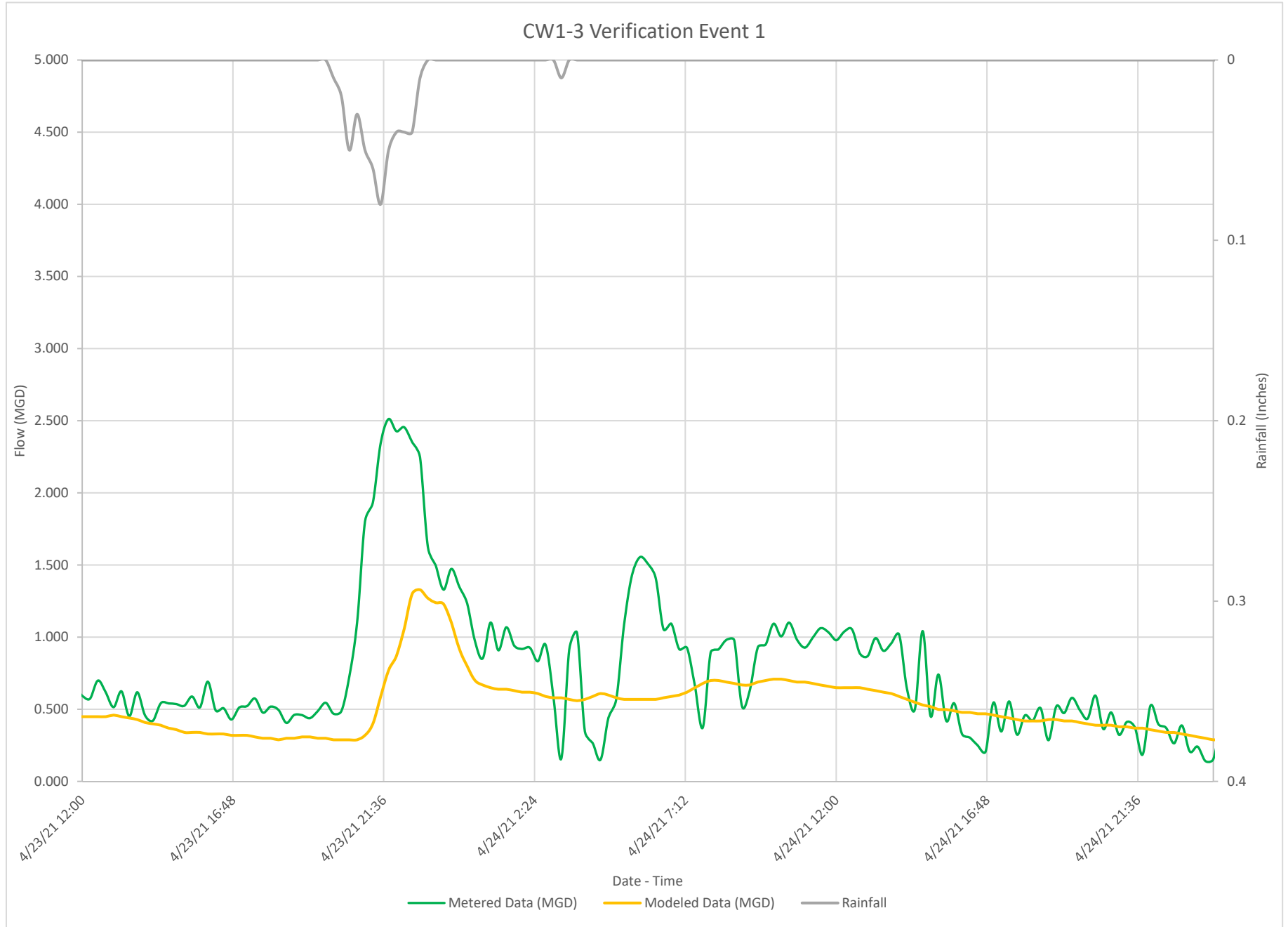
Sedalia WWMP - Draft



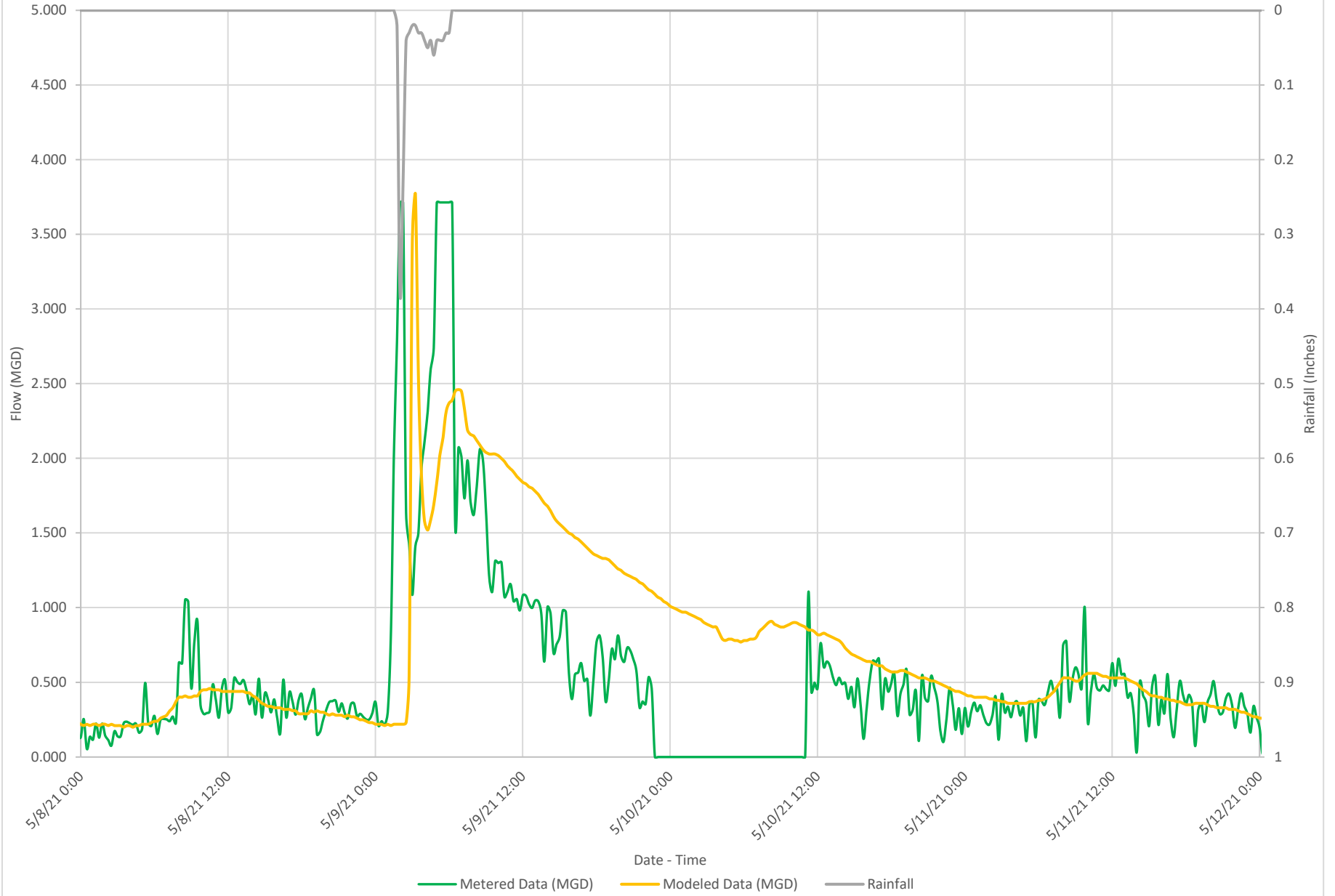




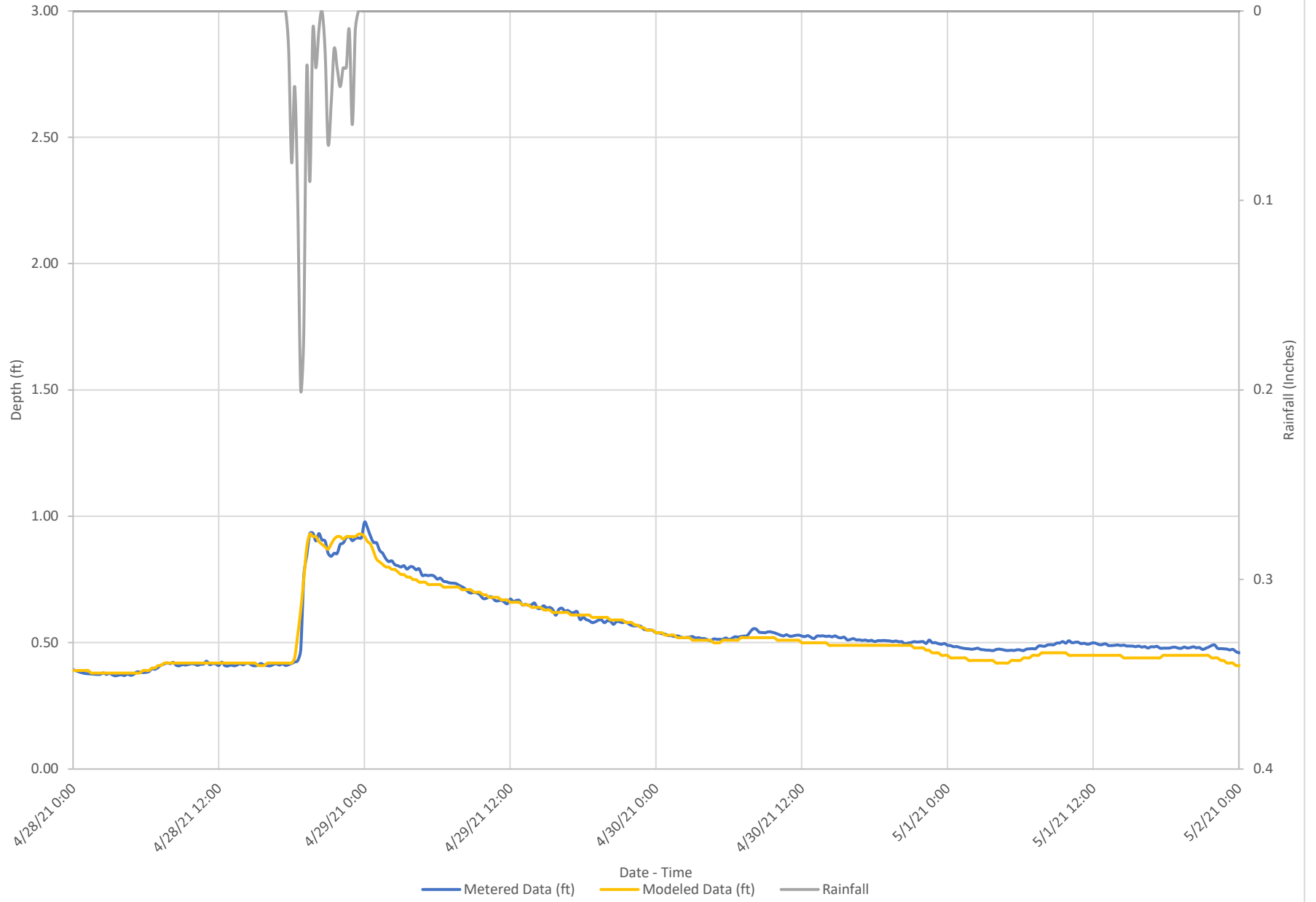




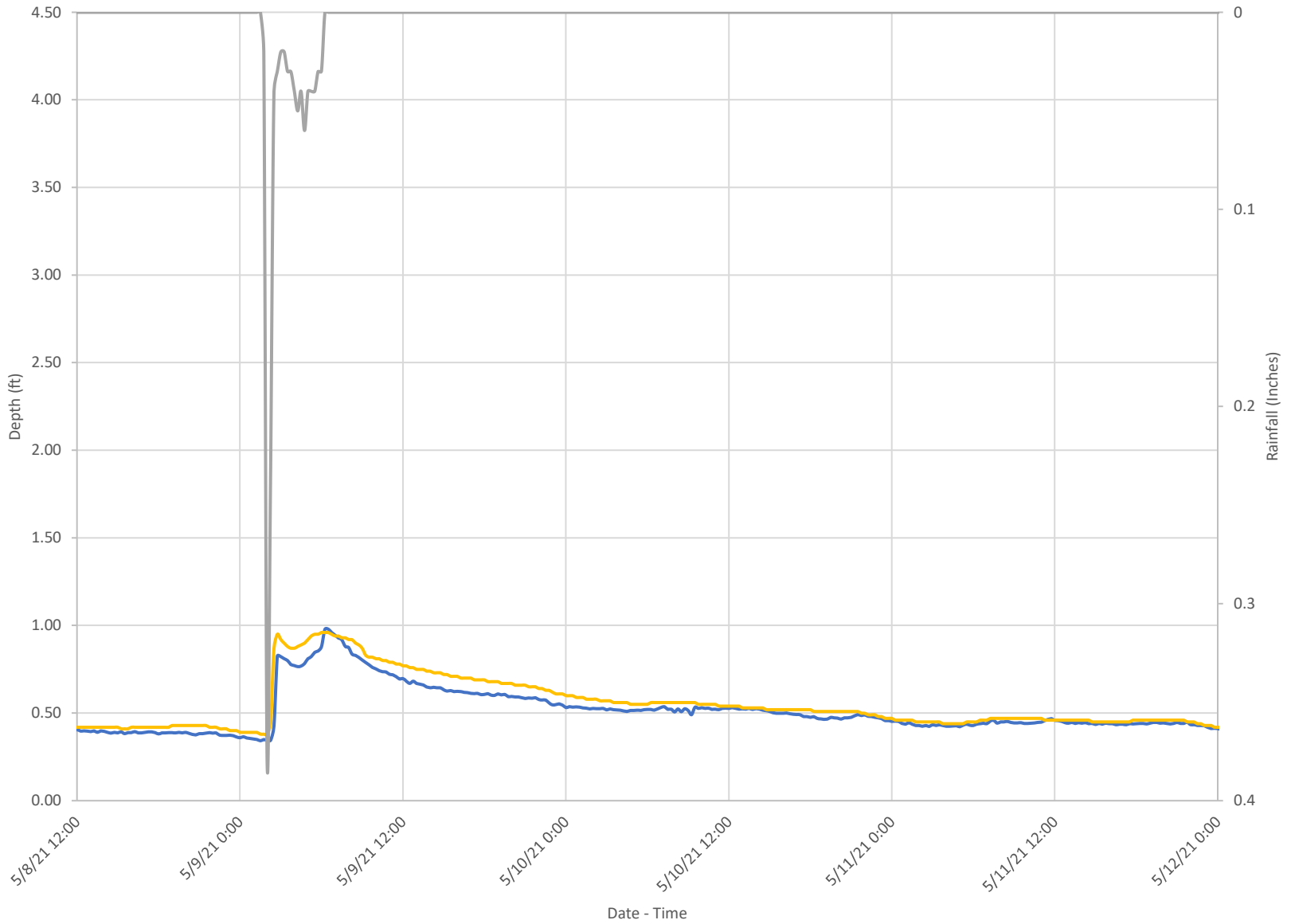
CW1-3 Verification Event 2



CE1-30 Calibration 1

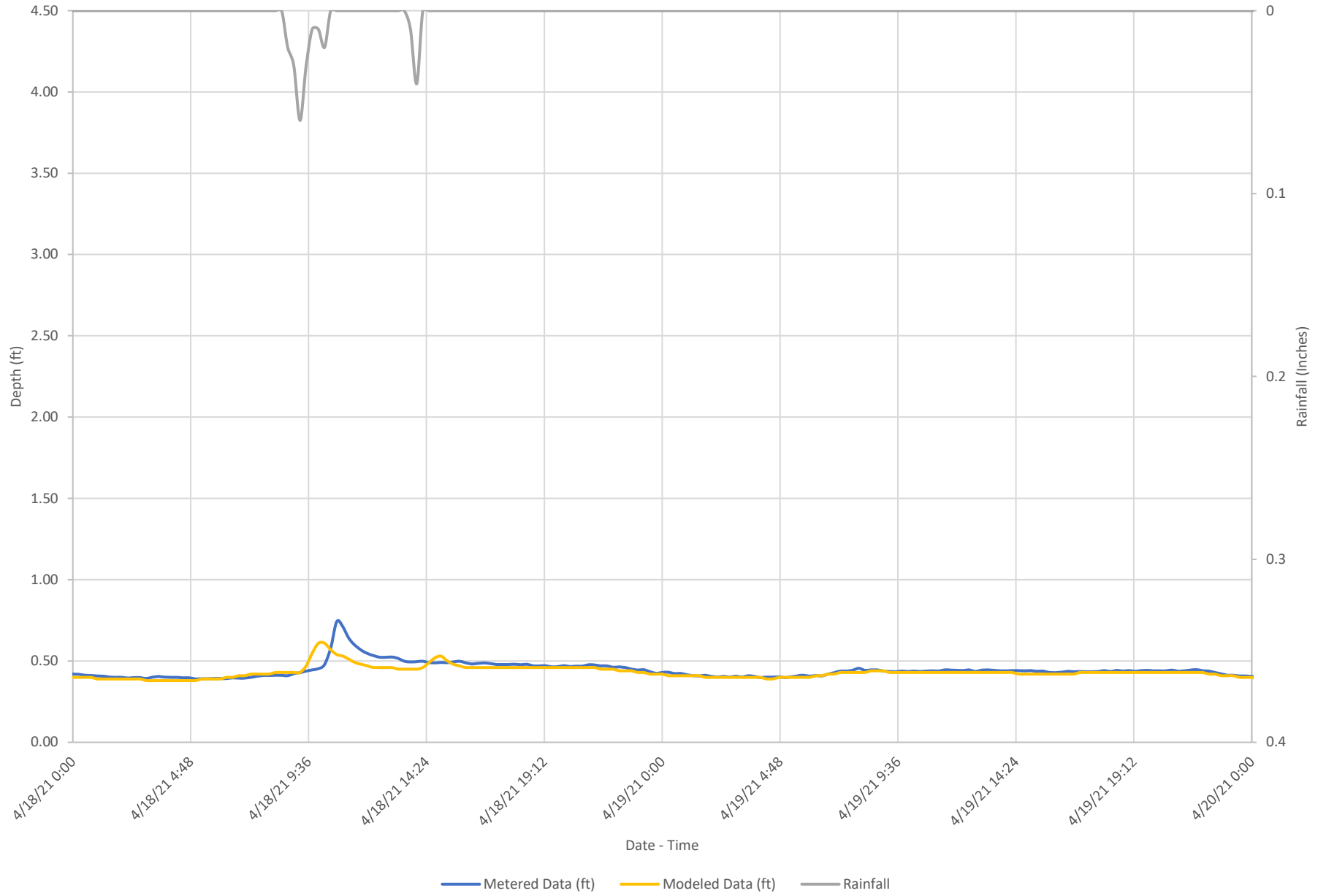


CE1-30 Calibration 2

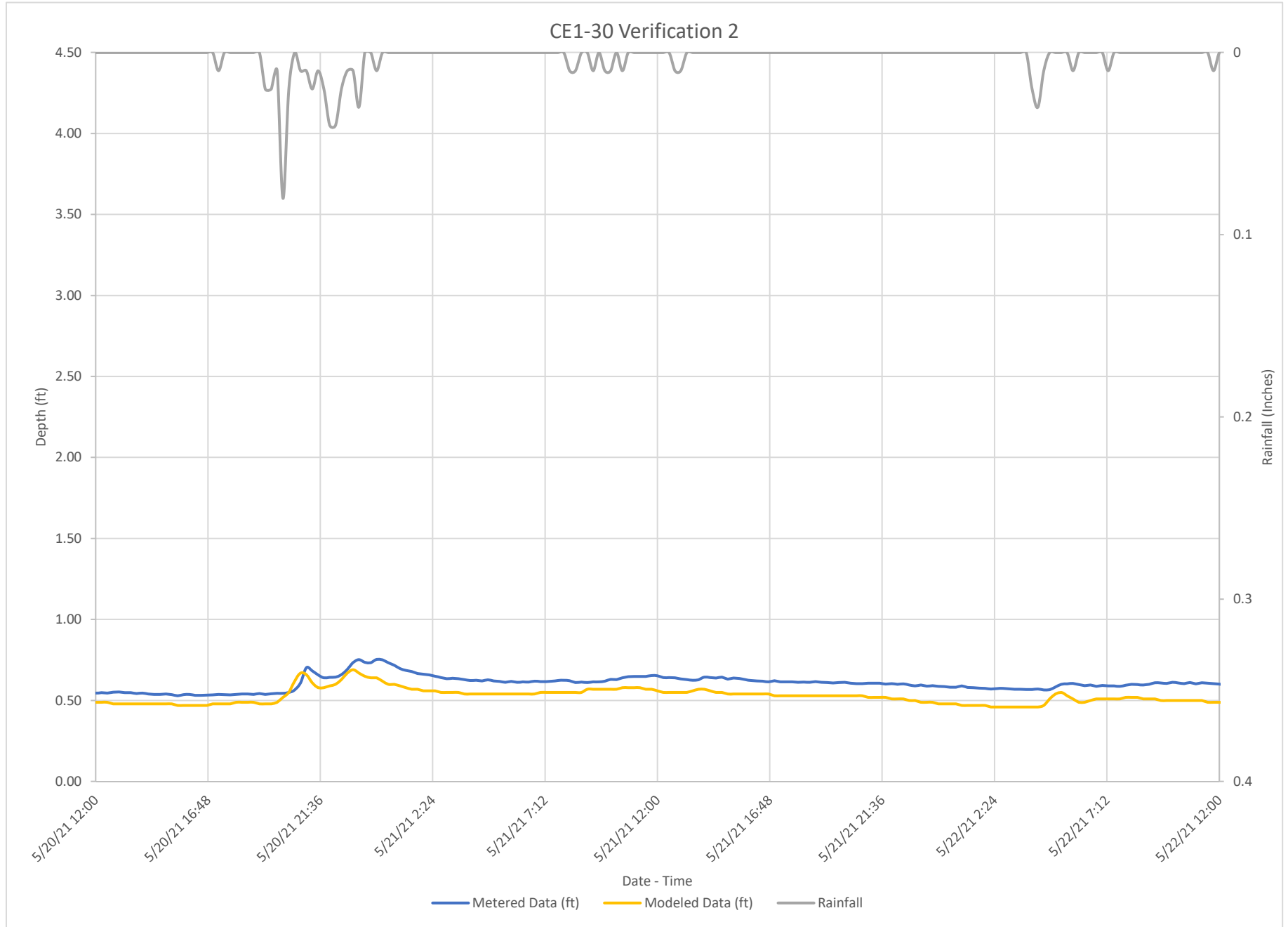


— Metered Data (ft) — Modeled Data (ft) — Rainfall

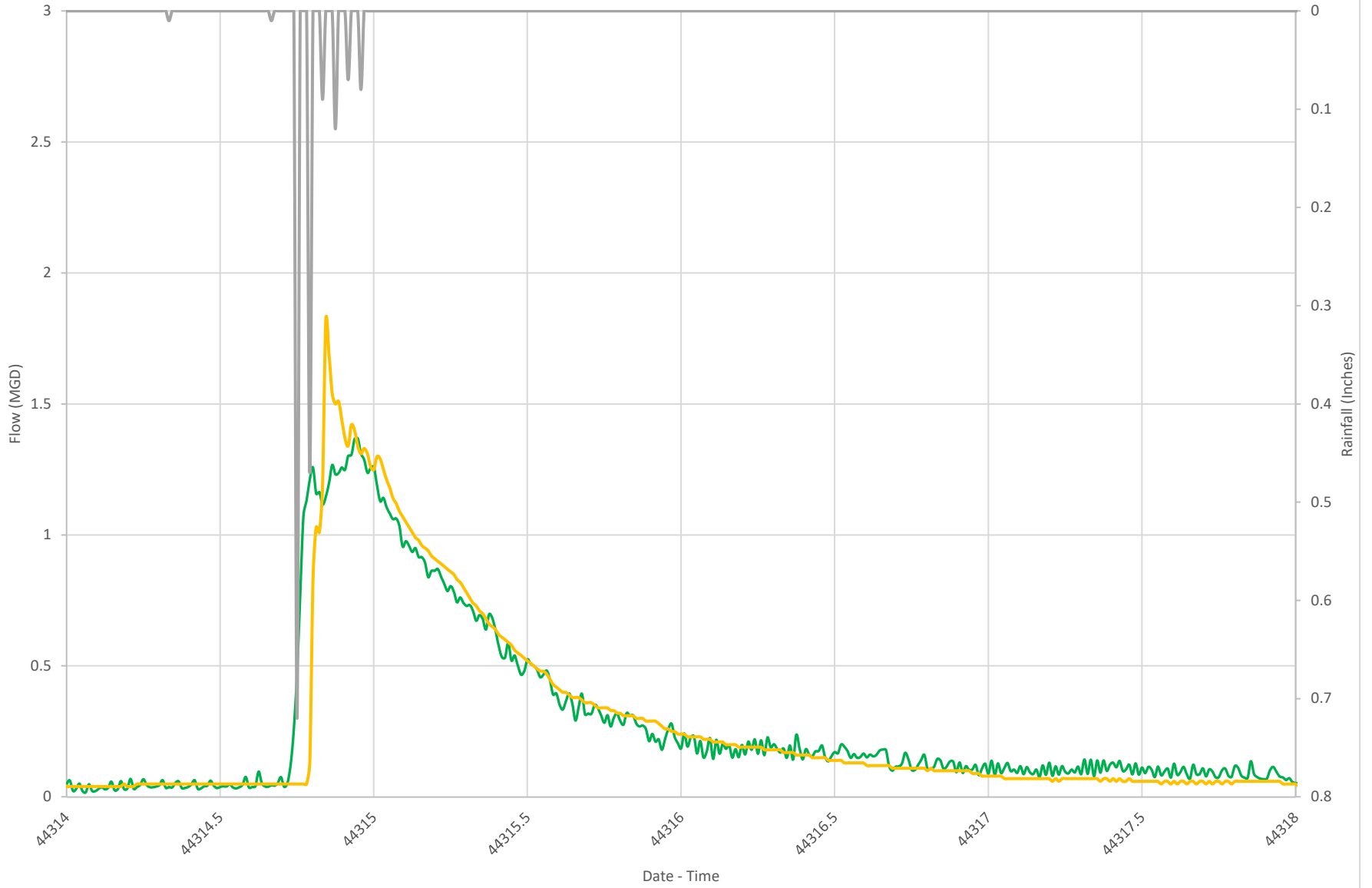
CE1-30 Verification 1



Sedalia WWMP - Draft

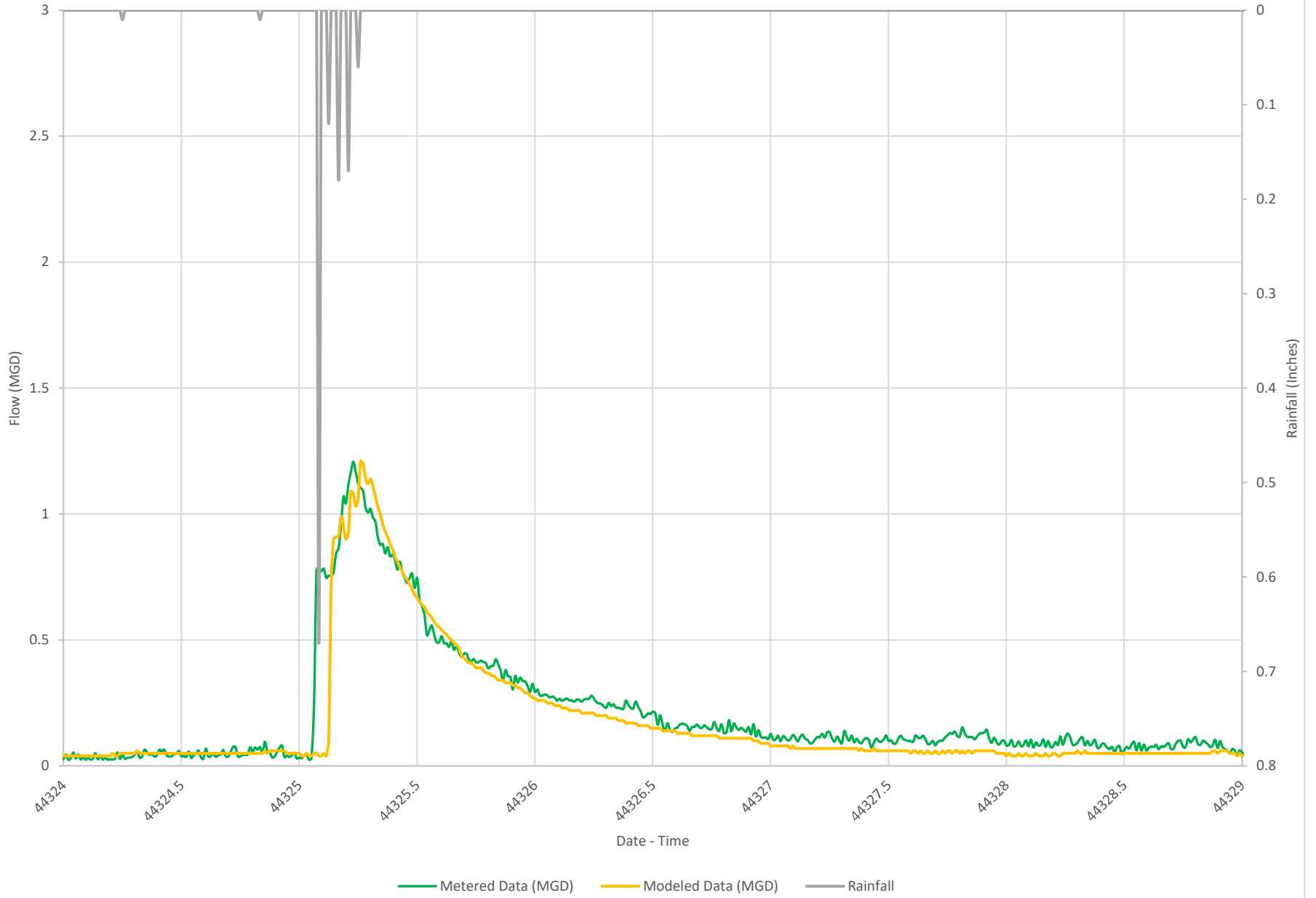


N3-5 Calibration Event 1

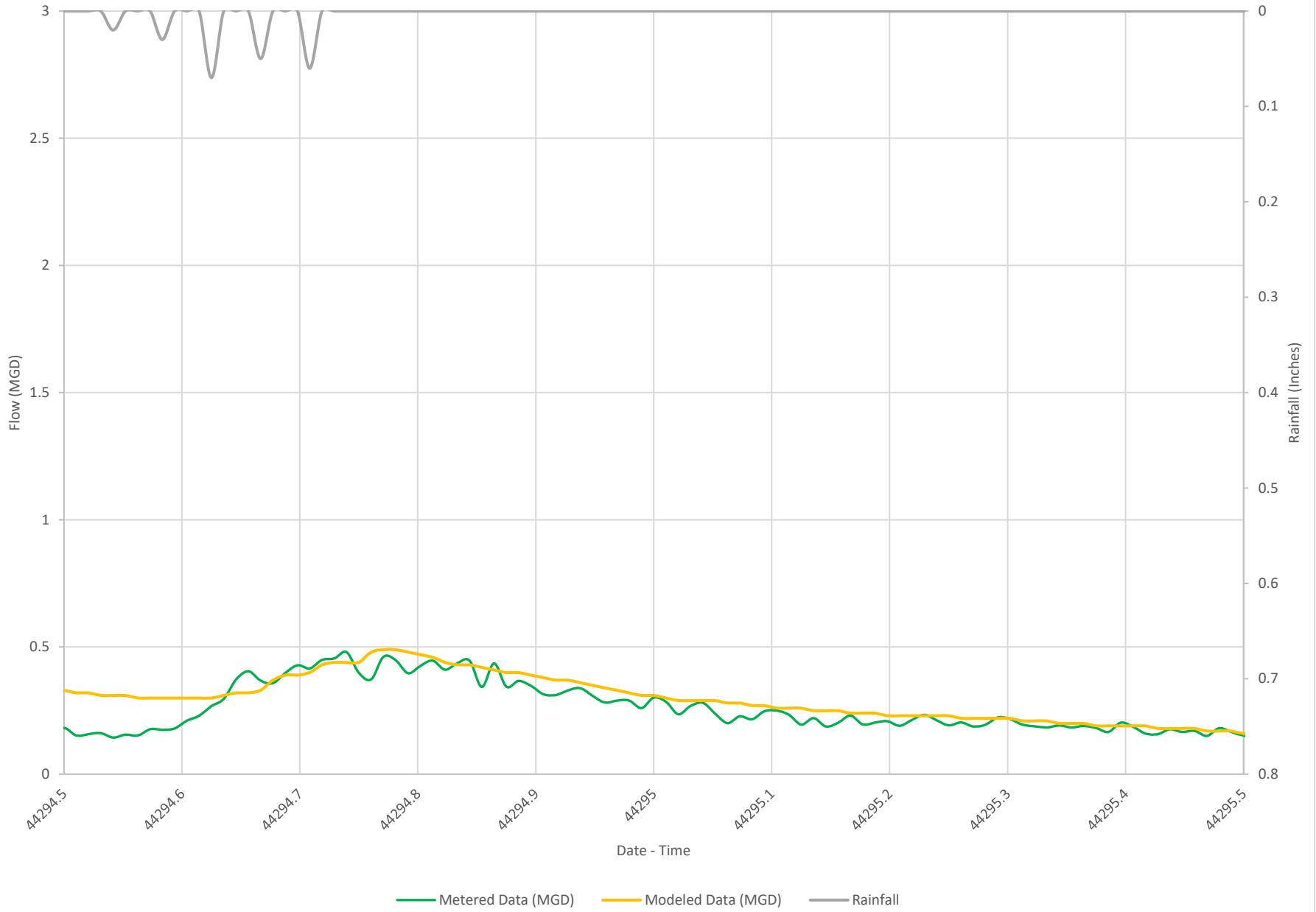


Metered Data (MGD) Modeled Data (MGD) Rainfall

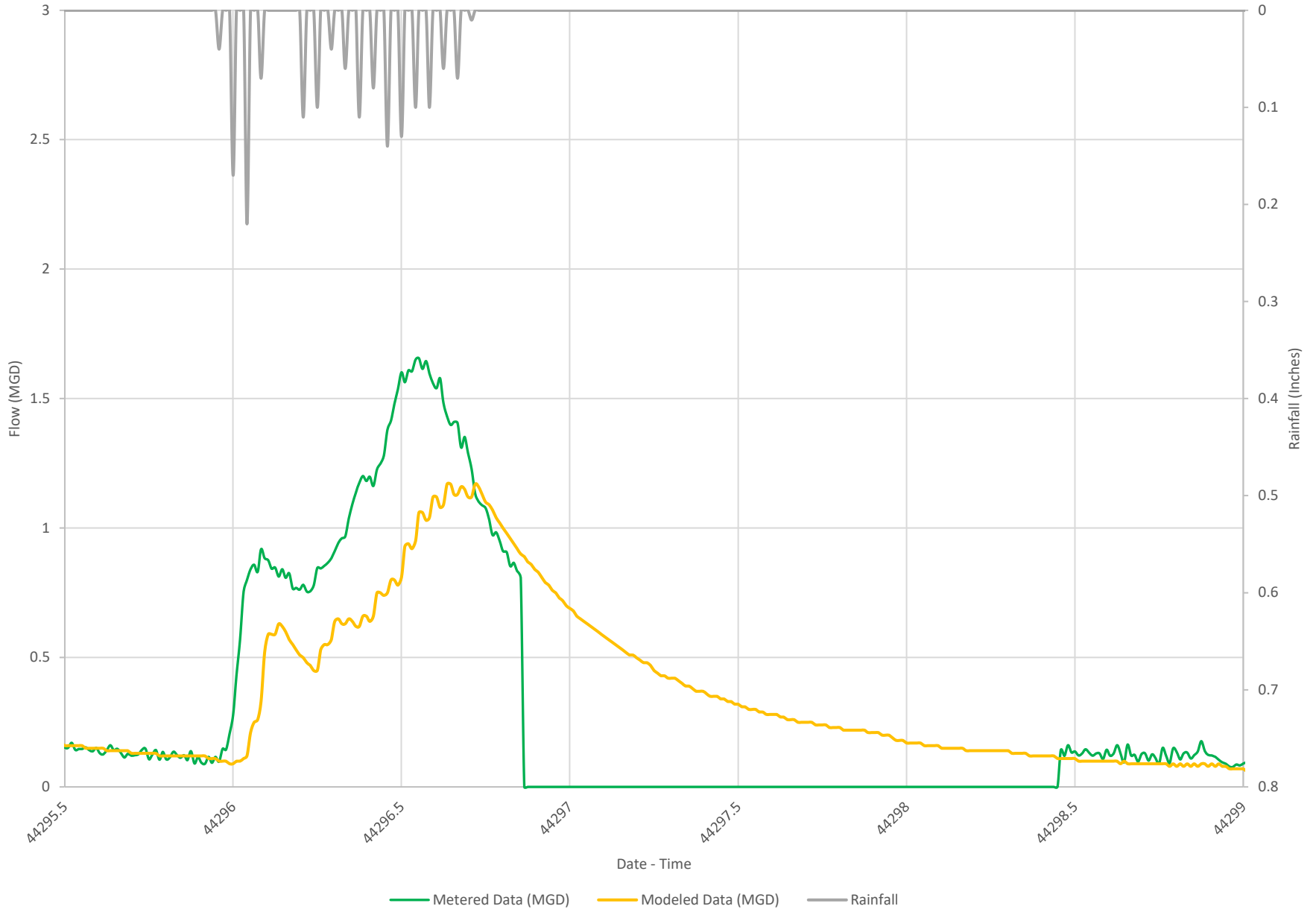
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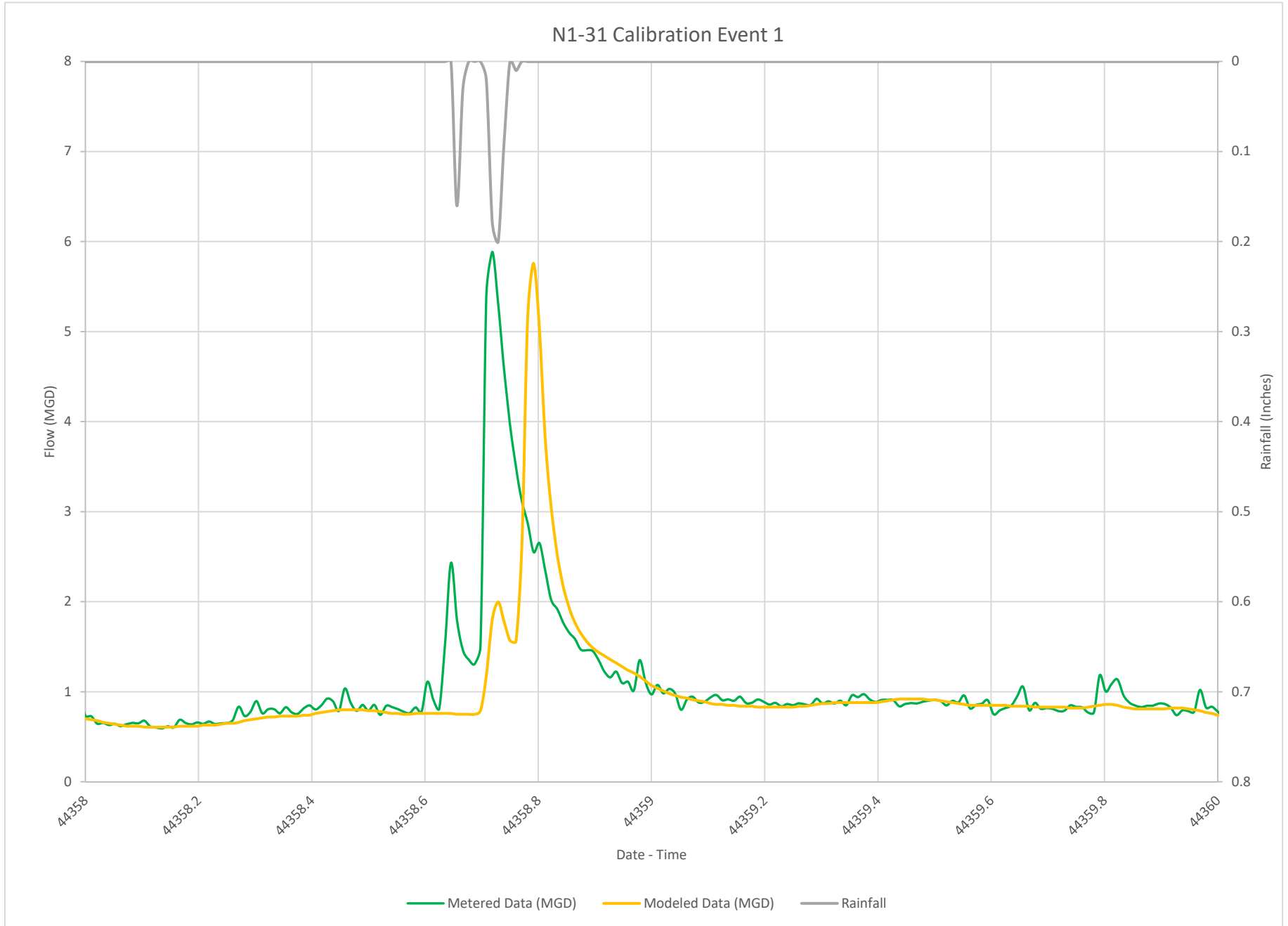
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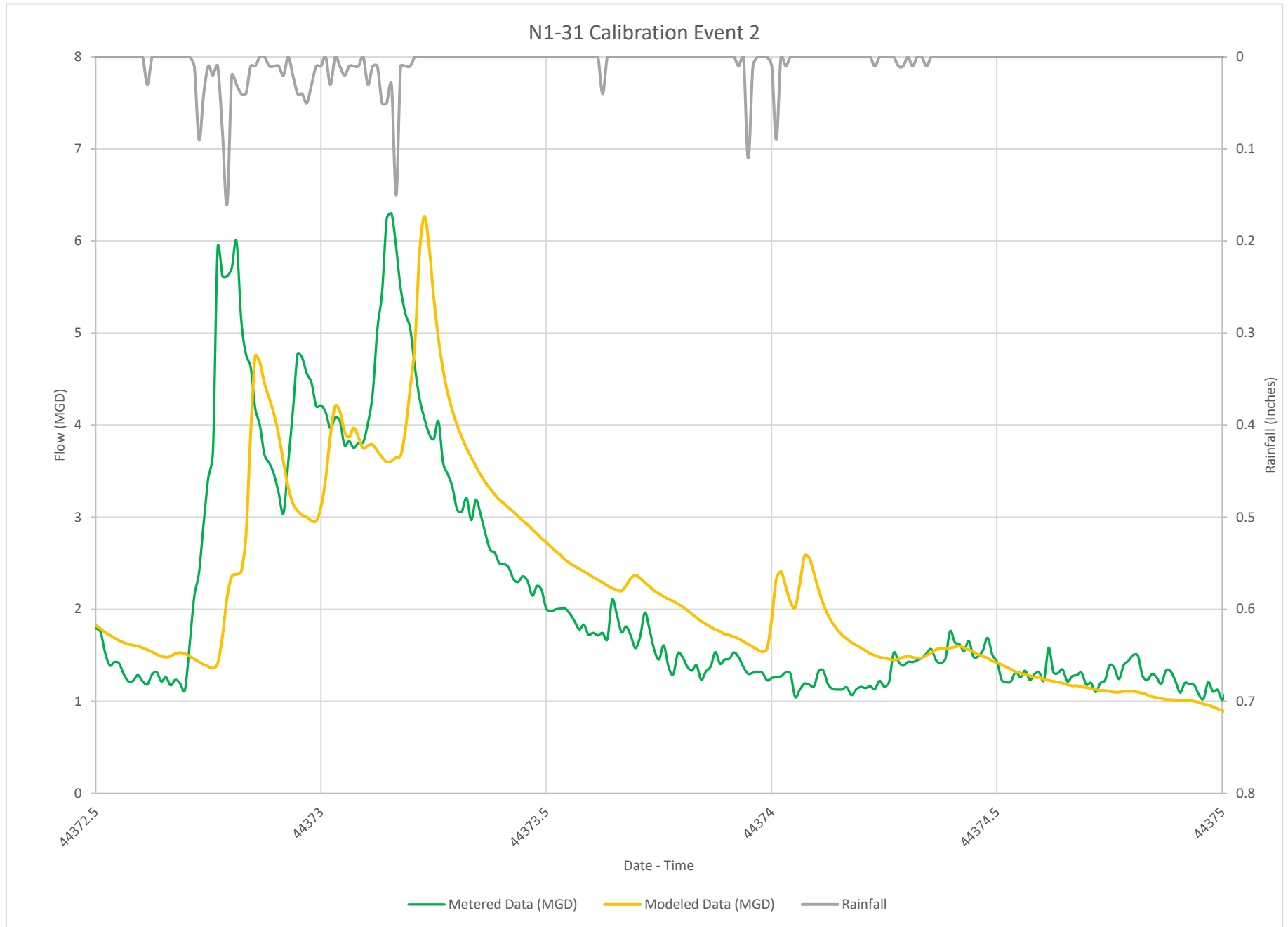


N3-5 Verification Event 2

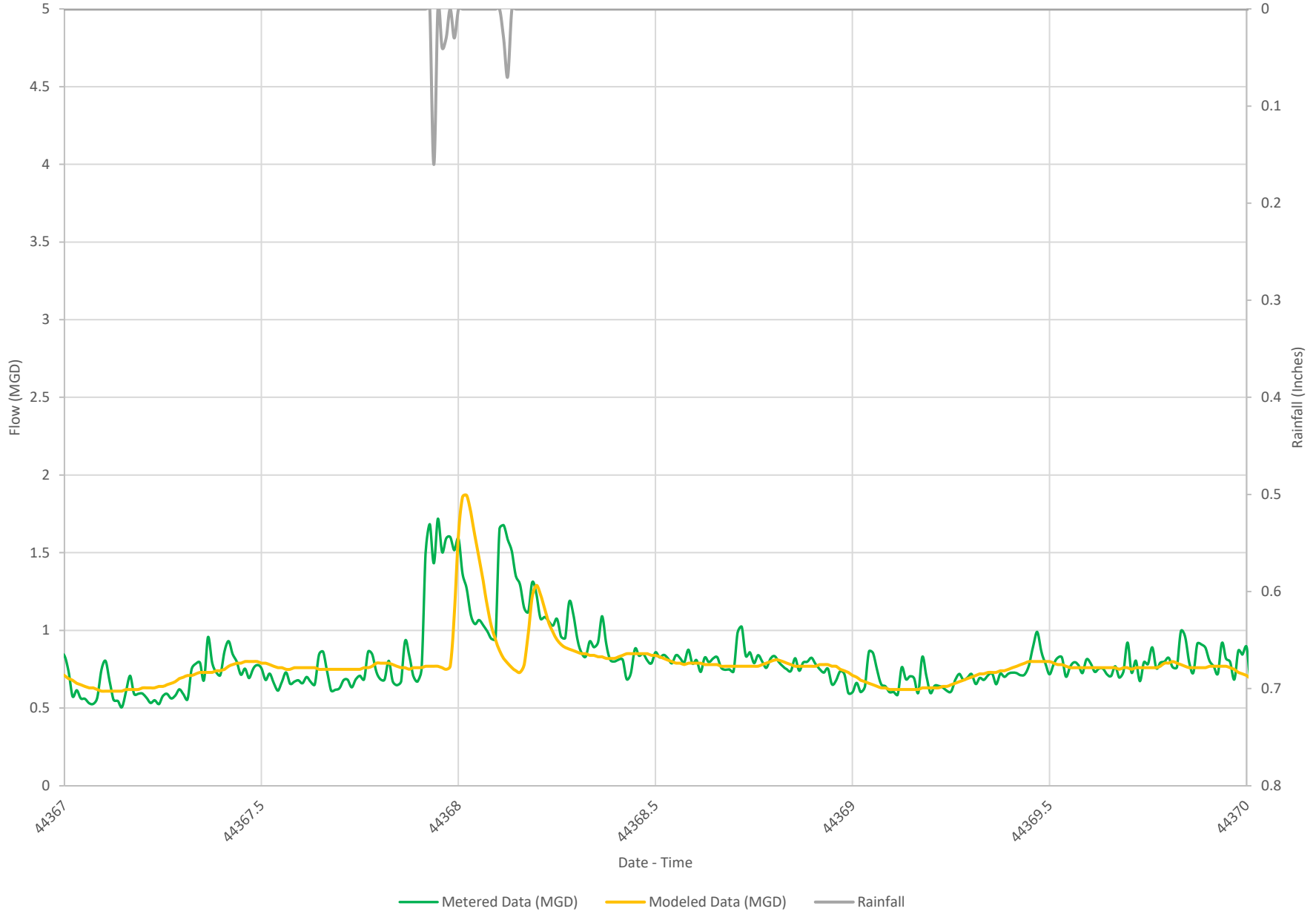


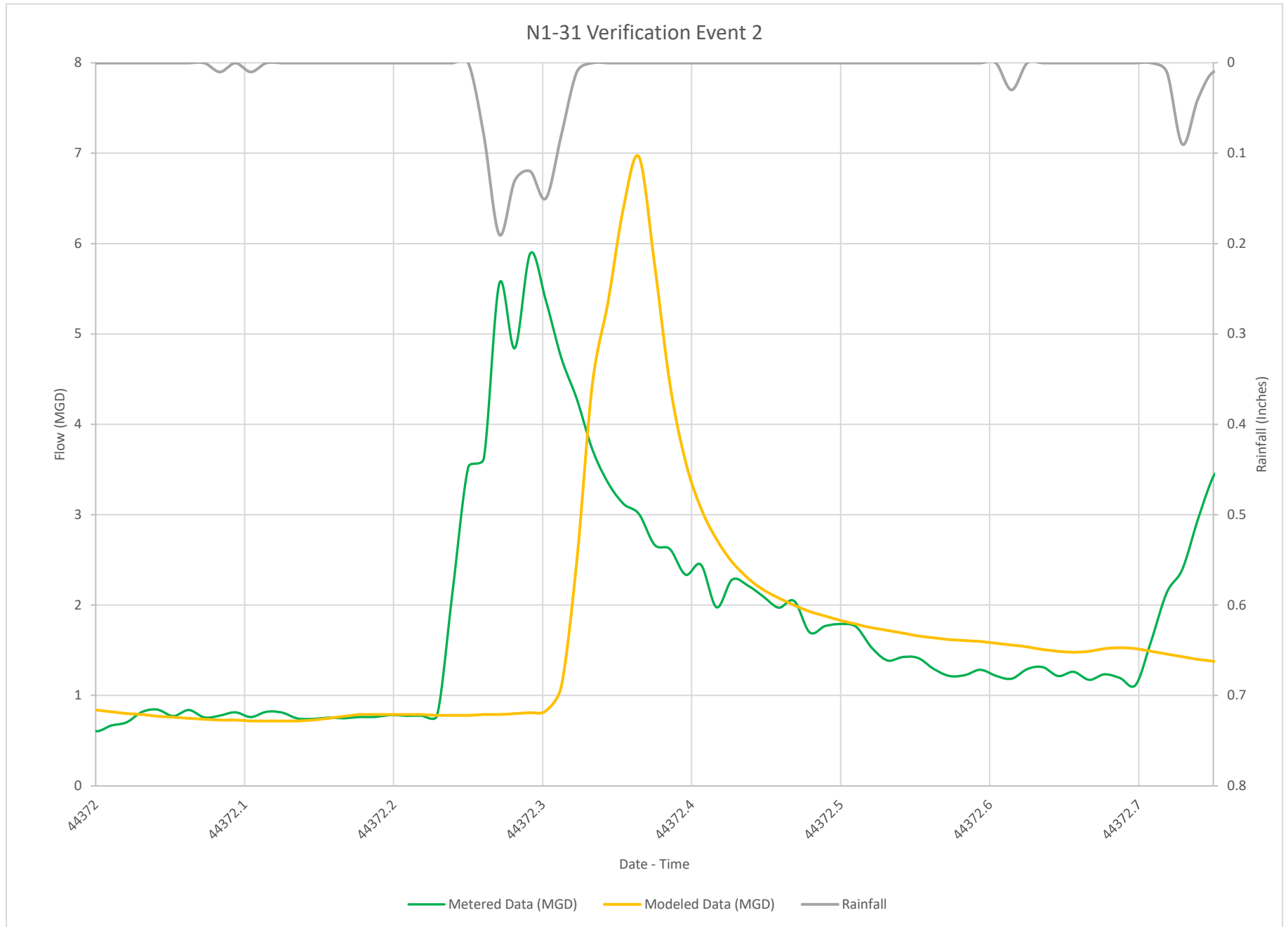
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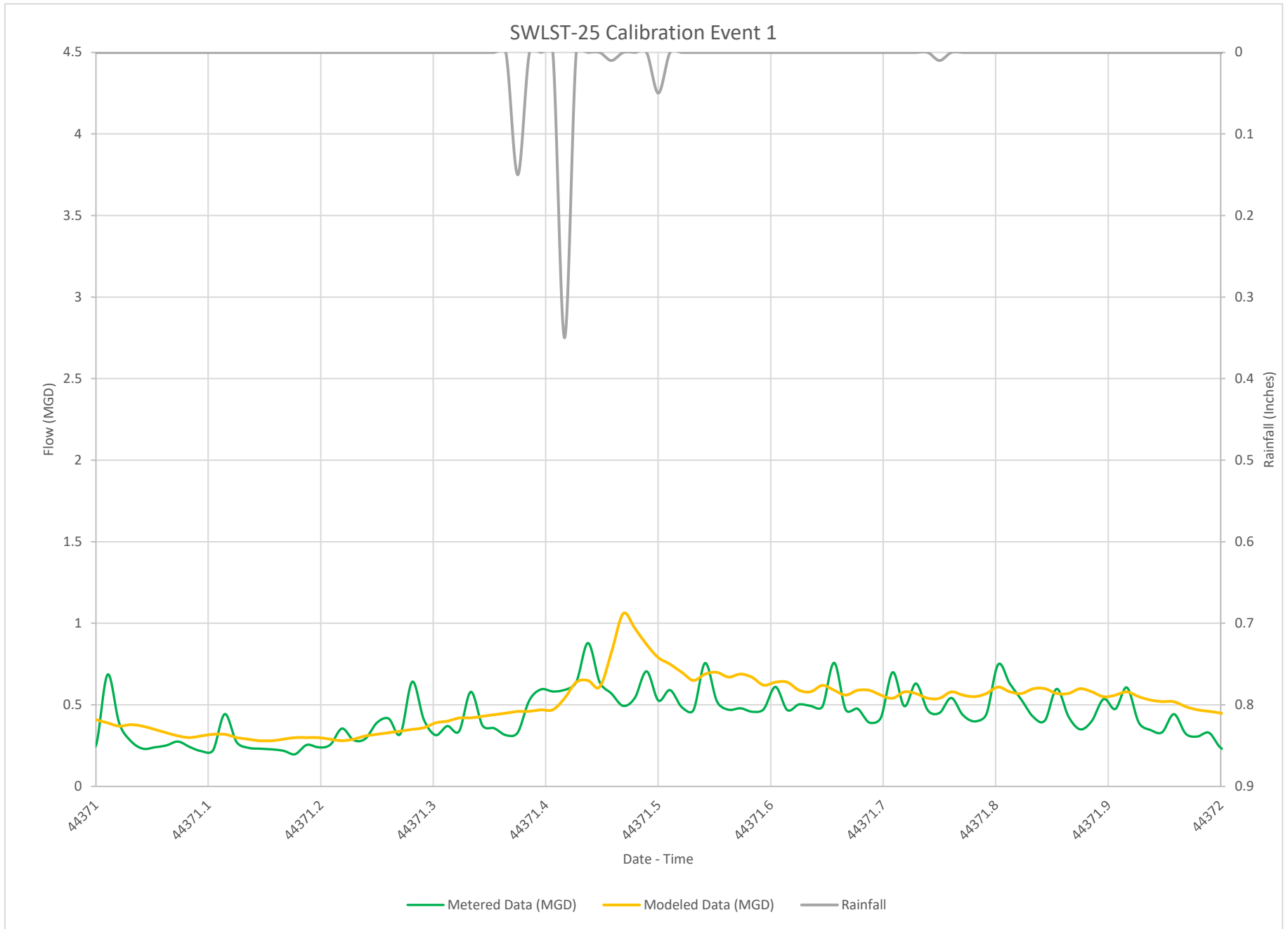


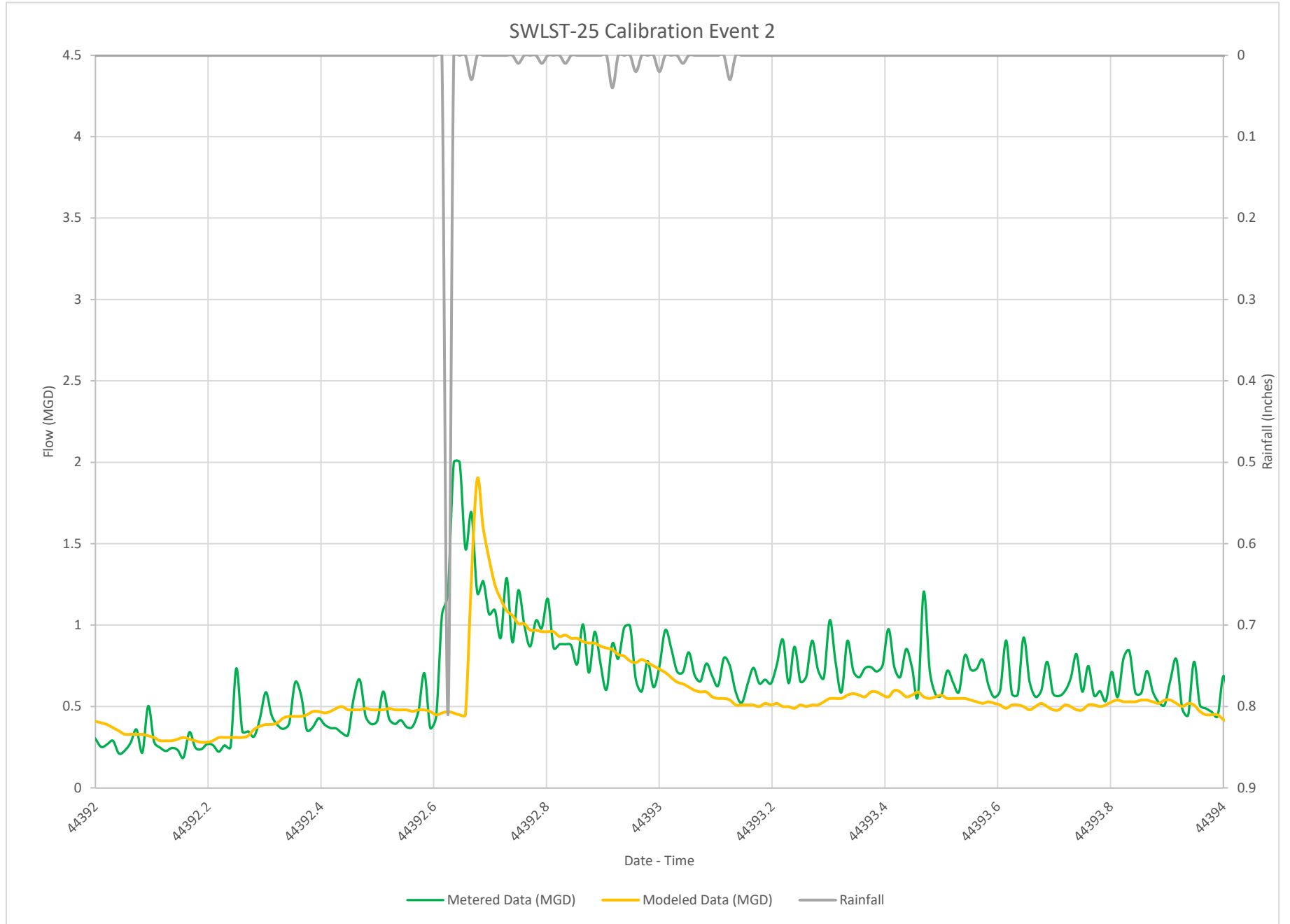
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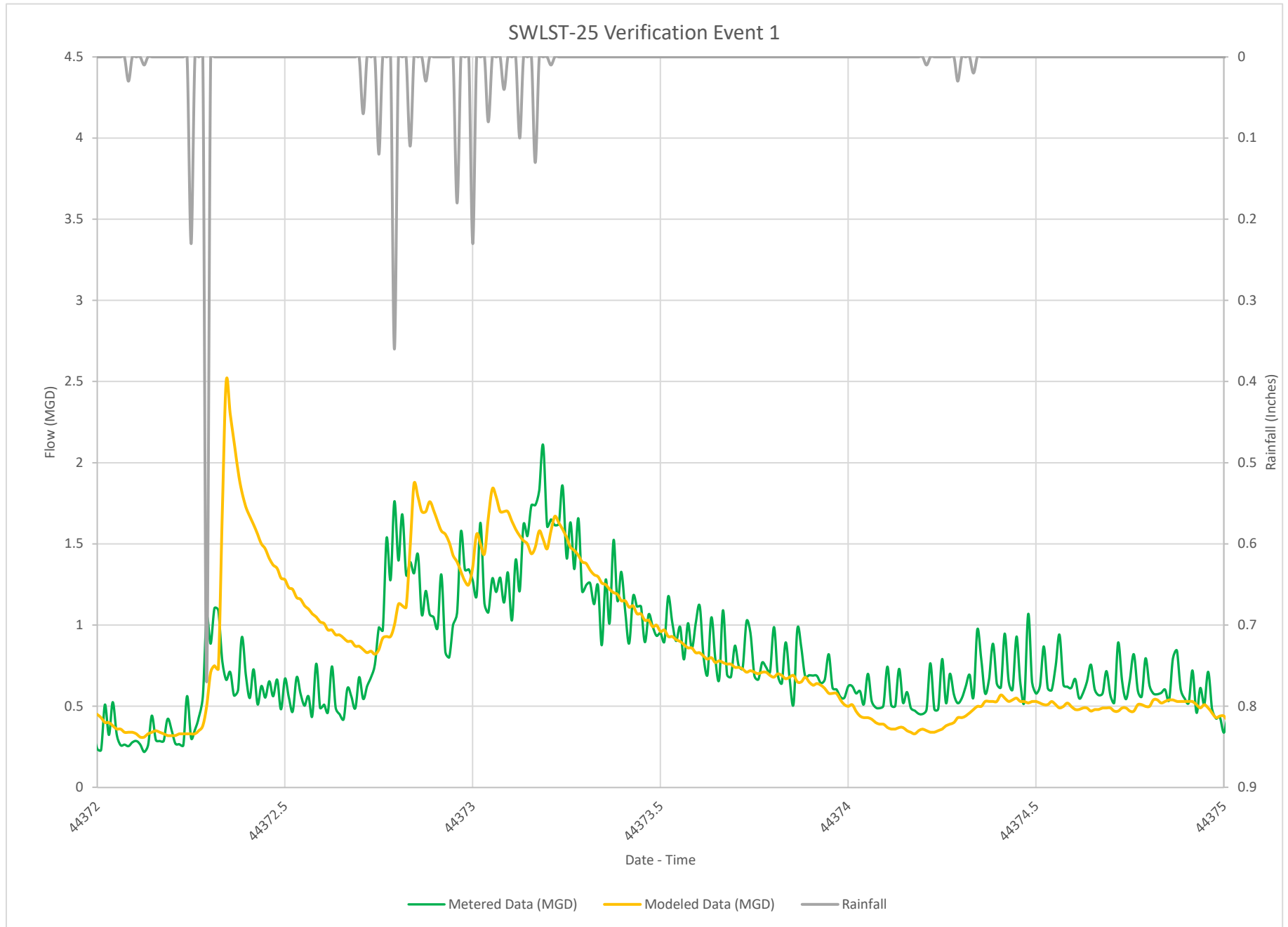


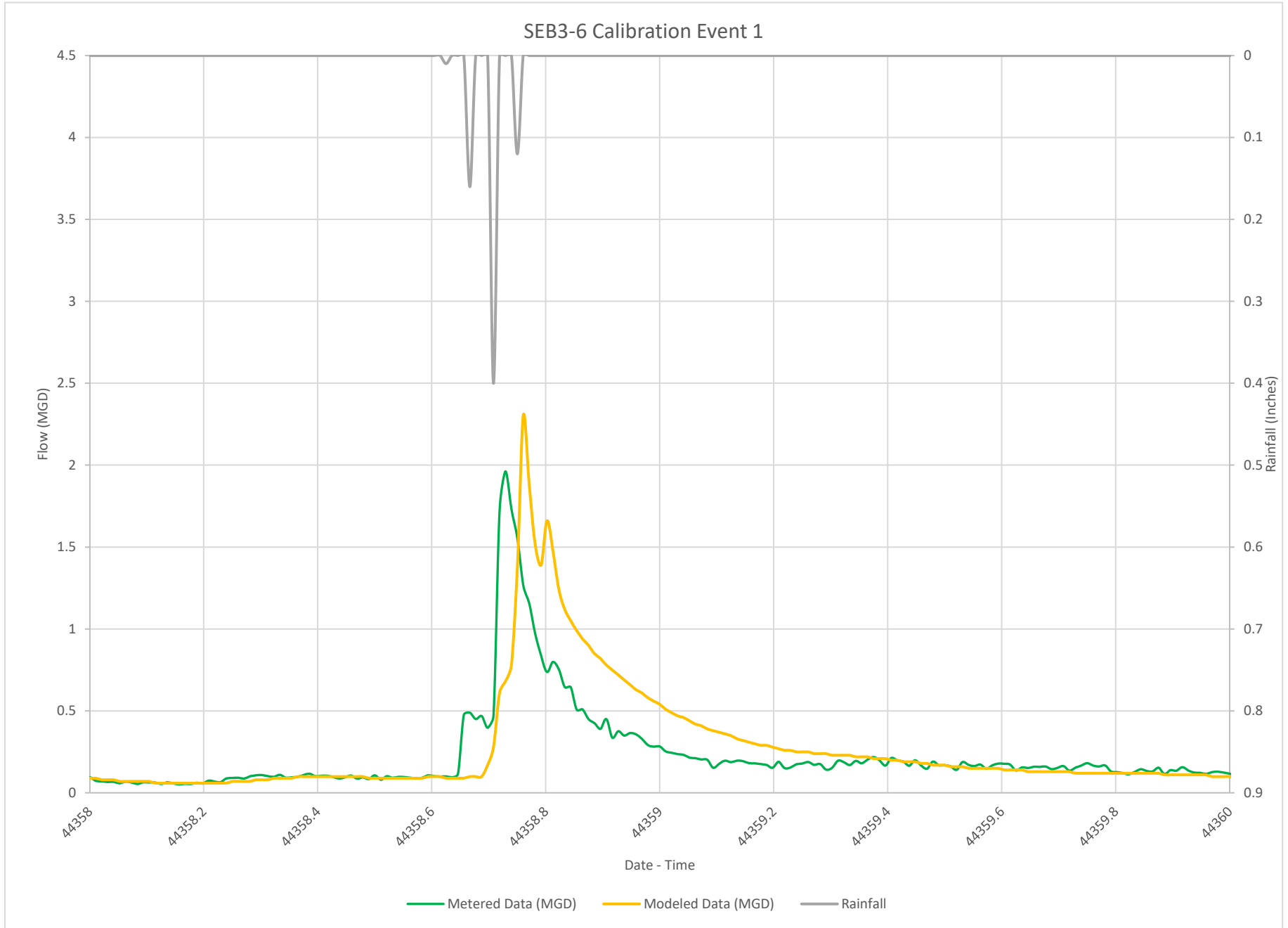


Sedalia WWMP - Draft

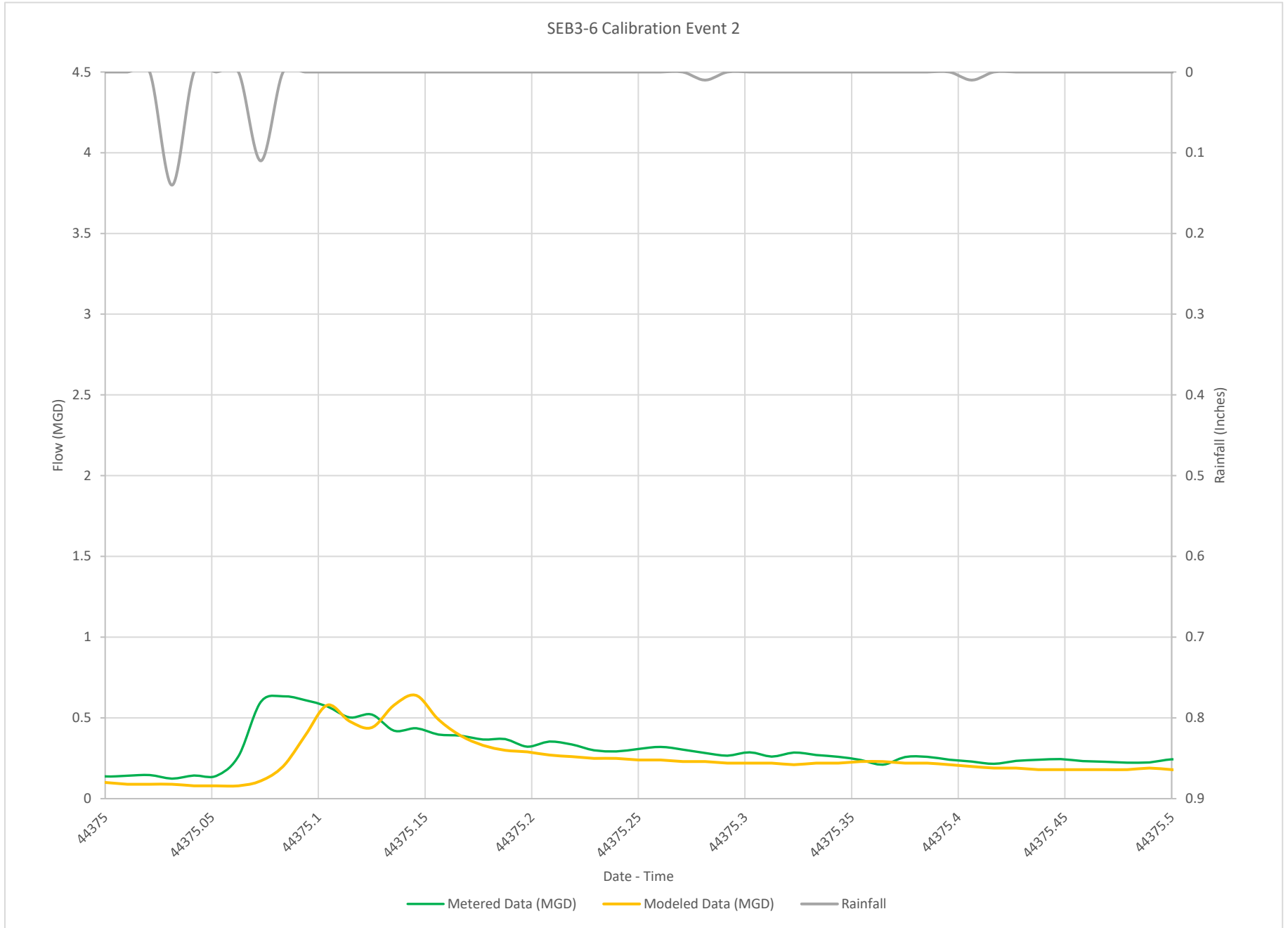




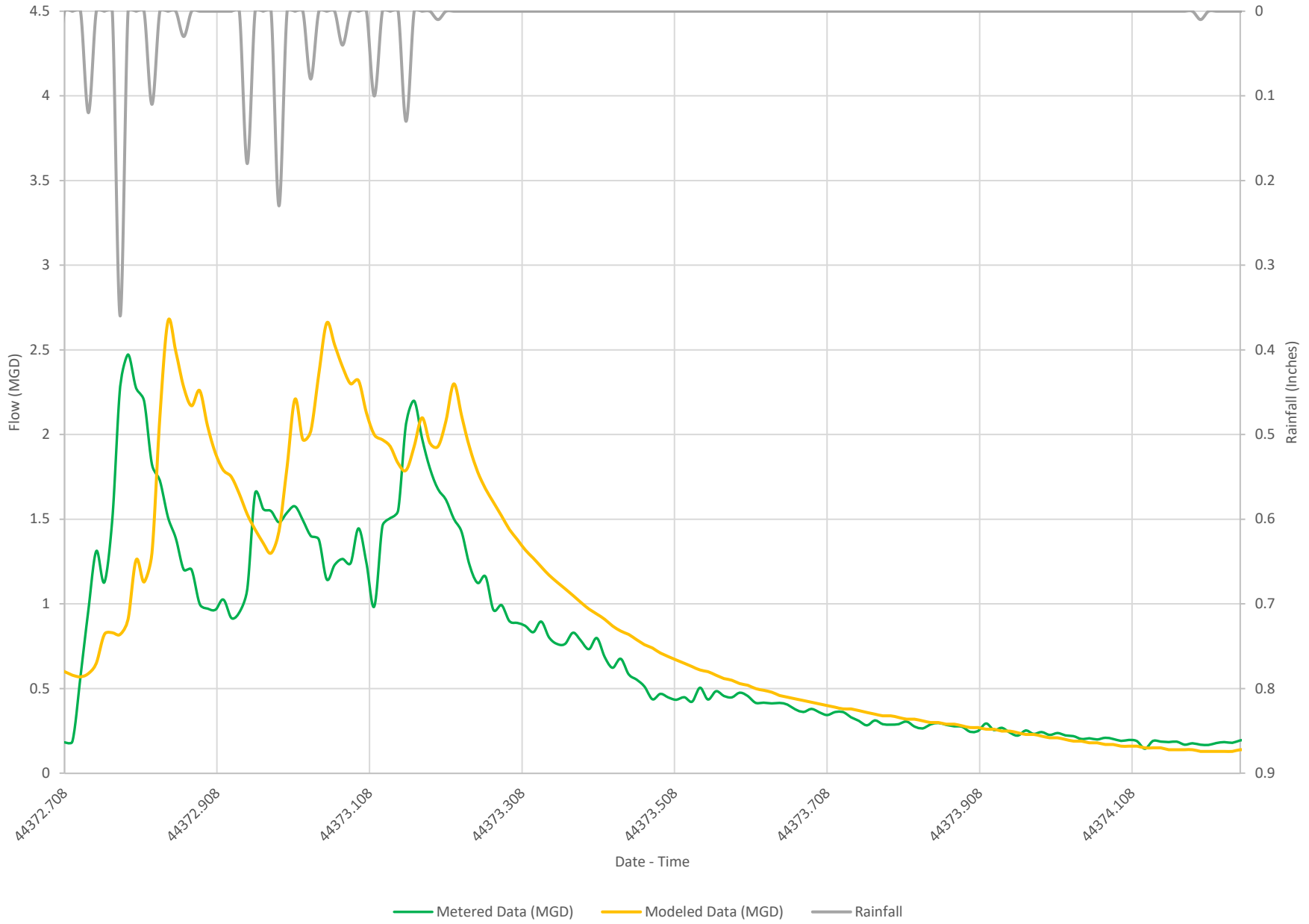




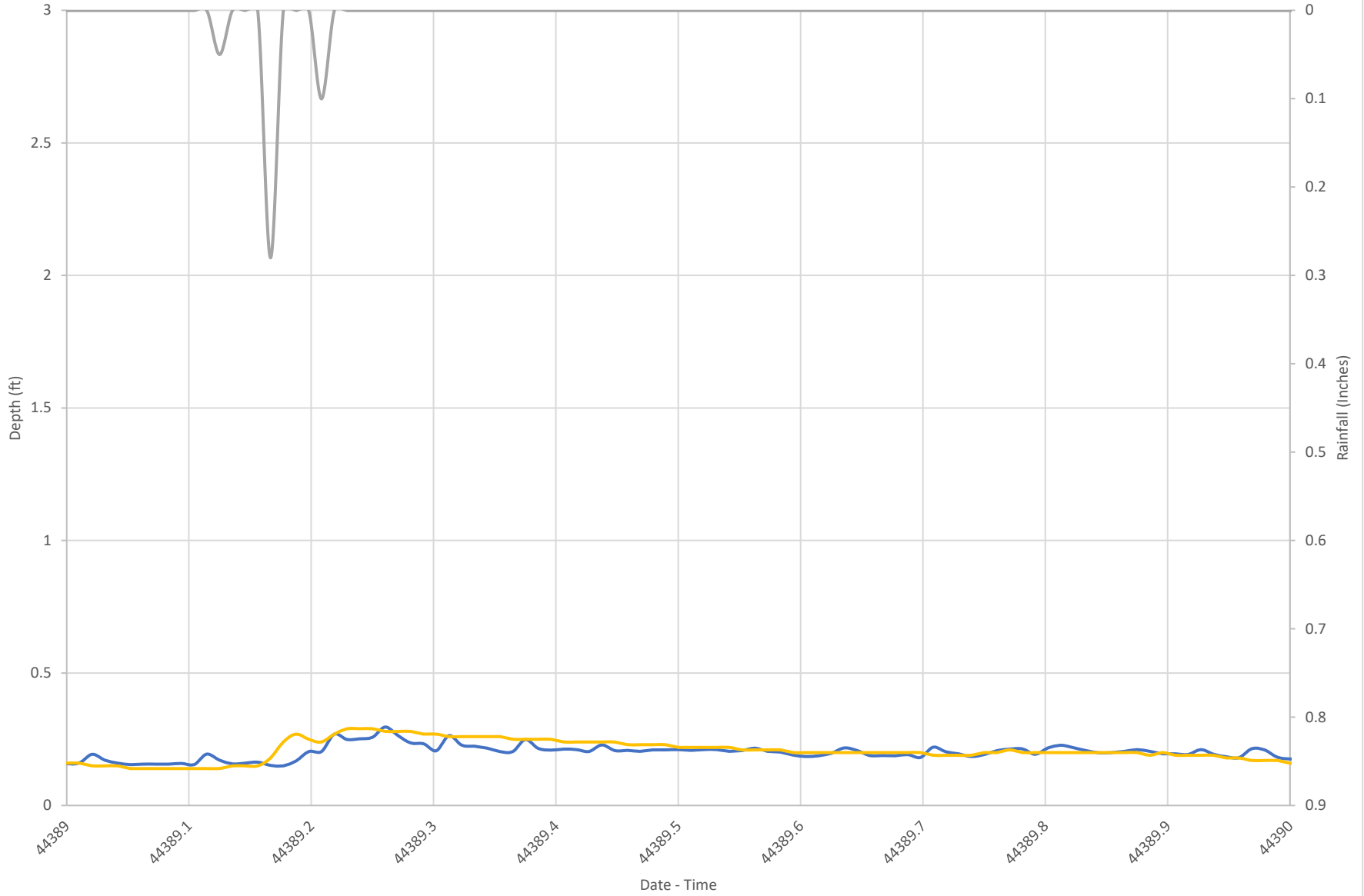
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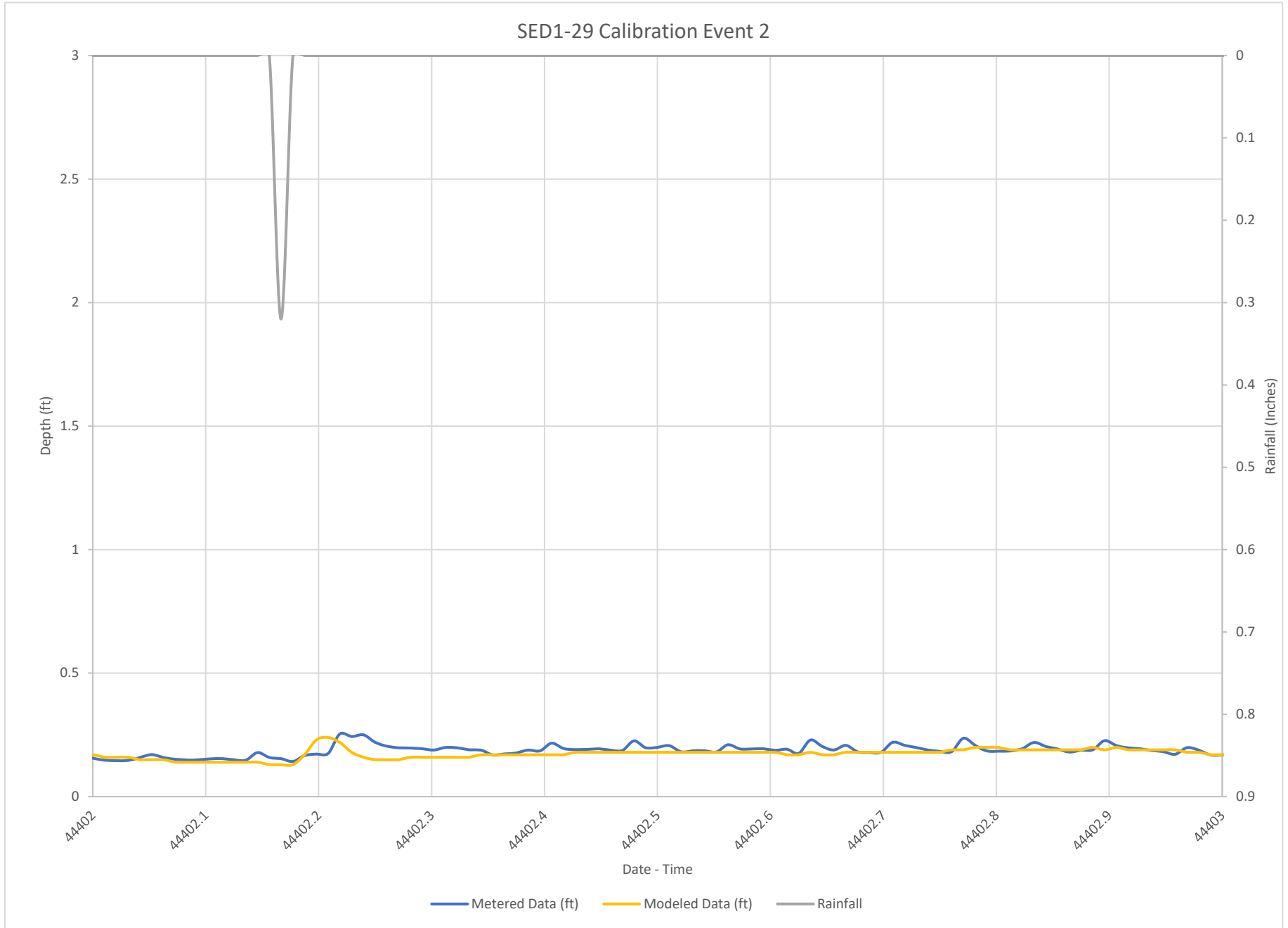
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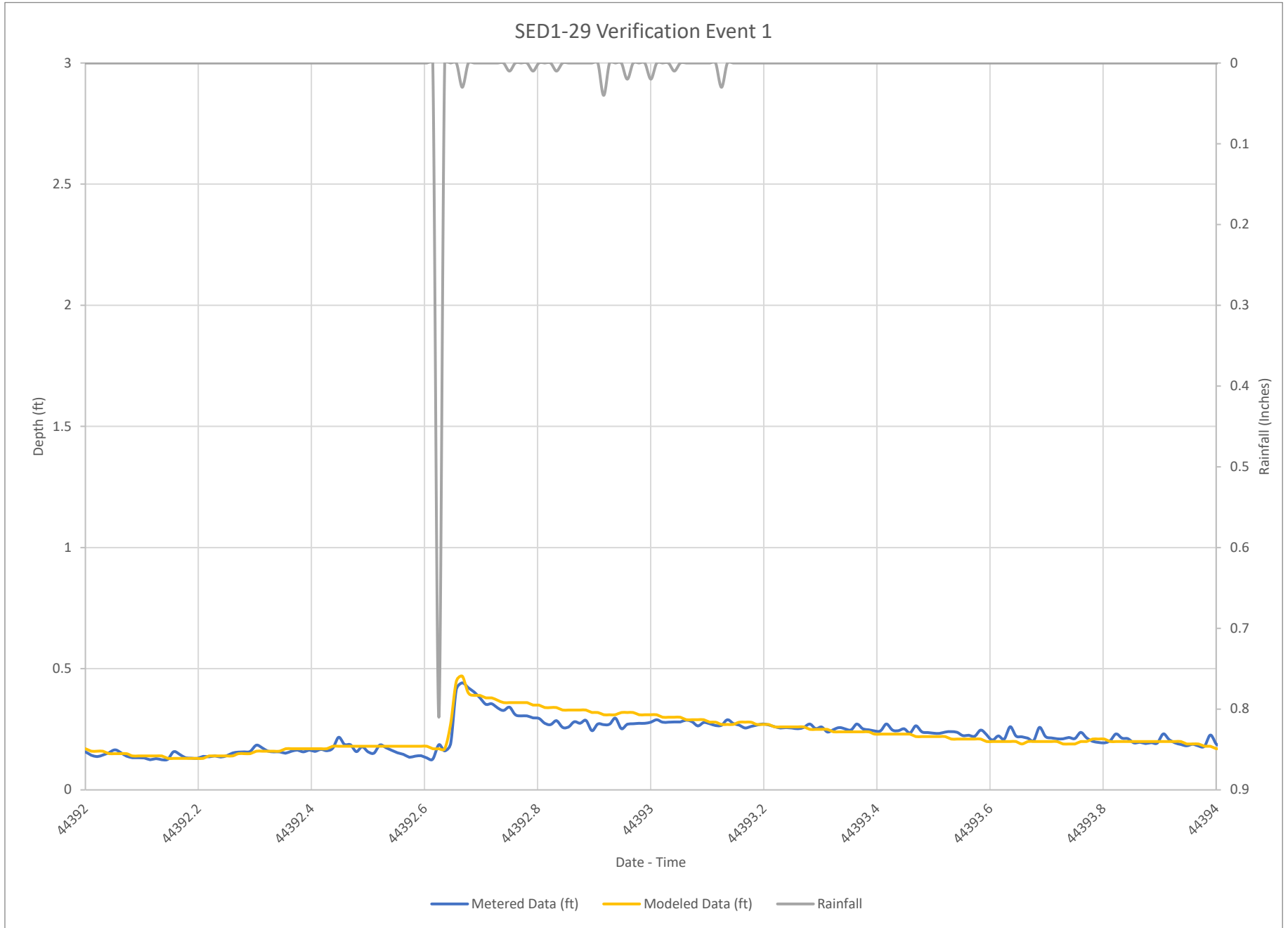


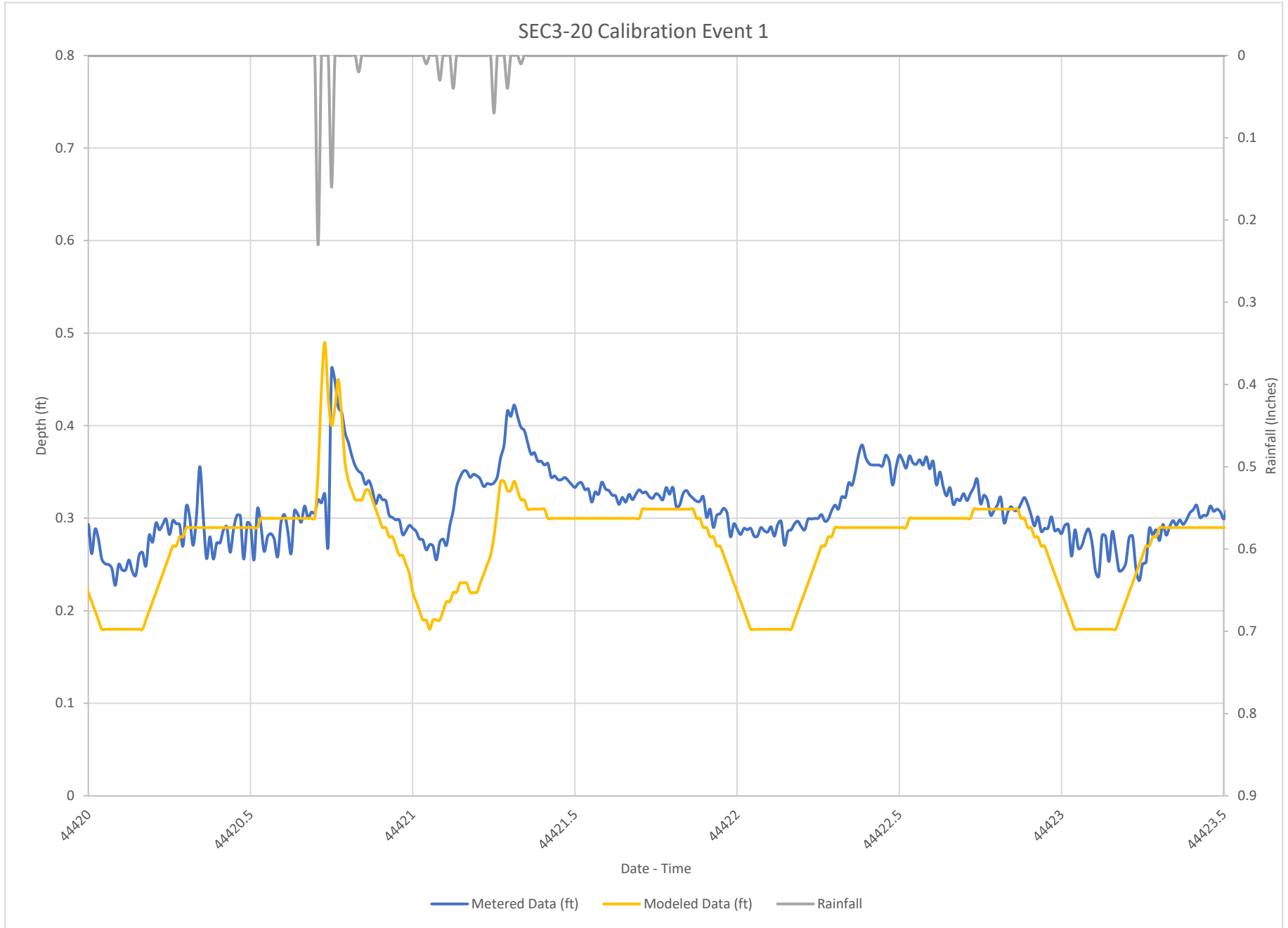
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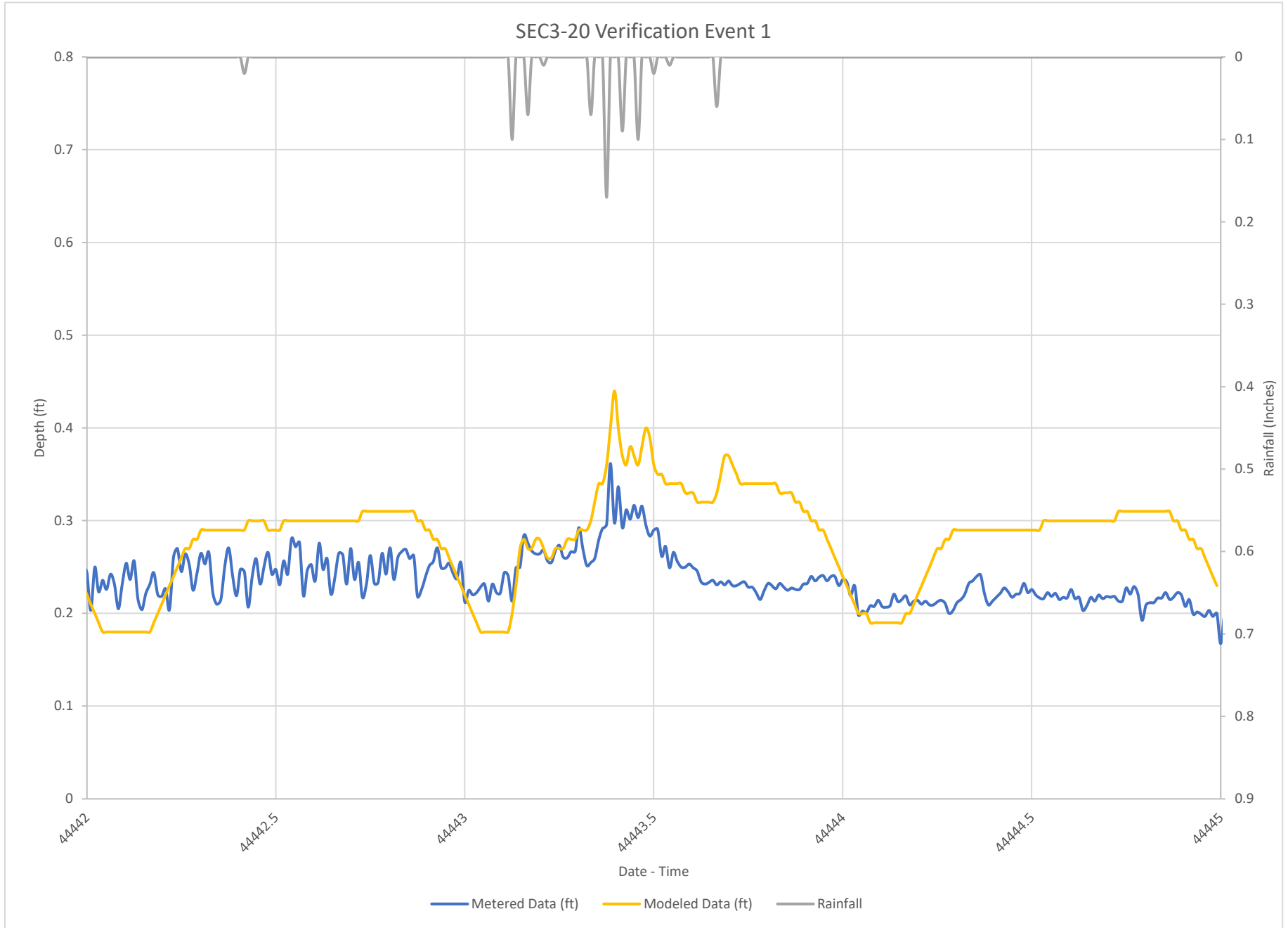


— Metered Data (ft) — Modeled Data (ft) — Rainfall









APPENDIX E – OPINION OF PROBABLE COSTS

Sedalia WWMP - Opinion of Probable Cost - DRAFT

CIP Opinion of Probable Cost - Backup					
CIP #	Item	Quantity	Units	Unit Cost	Opinion of Probable Cost
6	12" Relief Sewer in pavement	600	LF	\$ 820.00	\$ 492,000.00
8	36" Relief Sewer in pavement	1,200	LF	\$ 1,425.00	\$ 1,710,000.00
8	Weir Adjustment	1	EA	\$ 100,000.00	\$ 100,000.00
9	30" Relief Sewer in pavement	1,400	LF	\$ 1,250.00	\$ 1,750,000.00
9	36" Relief Sewer in pavement	4,700	LF	\$ 1,425.00	\$ 6,697,500.00
10	15" Relief Sewer in pavement	600	LF	\$ 895.00	\$ 537,000.00
10	18" Relief Sewer in pavement	700	LF	\$ 984.00	\$ 688,800.00
11	15" Relief Sewer in pavement	2,300	LF	\$ 895.00	\$ 2,058,500.00
12	21" Relief Sewer in pavement	300	LF	\$ 1,005.00	\$ 301,500.00
12	24" Relief Sewer in pavement	1,400	LF	\$ 1,040.00	\$ 1,456,000.00
12	30" Relief Sewer in pavement	3,000	LF	\$ 1,250.00	\$ 3,750,000.00
12	36" Relief Sewer in pavement	400	LF	\$ 1,425.00	\$ 570,000.00
13	18" Relief Sewer in pavement	800	LF	\$ 984.00	\$ 787,200.00
14	21" Relief Sewer in pavement	750	LF	\$ 1,005.00	\$ 753,750.00
14	24" Relief Sewer in pavement	400	LF	\$ 1,040.00	\$ 416,000.00
15	15" Relief Sewer in pavement	4,100	LF	\$ 895.00	\$ 3,669,500.00
15	18" Relief Sewer in pavement	450	LF	\$ 984.00	\$ 442,800.00
15	21" Relief Sewer in pavement	60	LF	\$ 1,005.00	\$ 60,300.00
16	15" Relief Sewer in pavement	1,200	LF	\$ 895.00	\$ 1,074,000.00
16	18" Relief Sewer in pavement	1,300	LF	\$ 984.00	\$ 1,279,200.00
17	15" Relief Sewer in pavement	580	LF	\$ 895.00	\$ 519,100.00
18	0.5 MGD Package Pump Station	1	EA	\$ 1,100,000.00	\$ 1,100,000.00
19	18" Relief Sewer in pavement	1,500	LF	\$ 984.00	\$ 1,476,000.00
20	15" Relief Sewer in pavement	1,200	LF	\$ 895.00	\$ 1,074,000.00
27	8" New Sewer	2,600	LF	\$ 163.00	\$ 423,800.00
27	0.5 MGD Pump Station	1	EA	\$ 150,000.00	\$ 150,000.00
27	6" Forcemain	6,300	LF	\$ 128.00	\$ 806,400.00
28	8" New Sewer	3,500	LF	\$ 163.00	\$ 570,500.00
28	0.5 MGD Pump Station	1	EA	\$ 150,000.00	\$ 150,000.00
28	6" Forcemain	4,900	LF	\$ 128.00	\$ 627,200.00
29	10" New Sewer	4,500	LF	\$ 181.00	\$ 814,500.00
29	1 MGD Pump Station	1	EA	\$ 300,000.00	\$ 300,000.00
29	6" Forcemain	6,000	LF	\$ 128.00	\$ 768,000.00
29	Decommision 1.5 MGD Pump Station	1	EA	\$ 125,000.00	\$ 125,000.00
30	6 MGD Pump Station	1	EA	\$ 1,250,000.00	\$ 1,250,000.00
30	12" Forcemain	9,500	LF	\$ 188.00	\$ 1,786,000.00
31	18" Relief Sewer in grass	5,300	LF	\$ 759.00	\$ 4,022,700.00
31	21" Relief Sewer in grass	4,500	LF	\$ 790.00	\$ 3,555,000.00
31	3 MGD Pump Station	1	EA	\$ 1,400,000.00	\$ 1,400,000.00
31	12" Forcemain	7,900	LF	\$ 188.00	\$ 1,485,200.00

Summary	
CIP #	Total Opinion of Probable Cost
6	\$ 490,000
8	\$ 1,810,000
9	\$ 8,450,000
10	\$ 1,230,000
11	\$ 2,060,000
12	\$ 6,080,000
13	\$ 790,000
14	\$ 1,170,000
15	\$ 4,170,000
16	\$ 2,350,000
17	\$ 520,000
18	\$ 1,100,000
19	\$ 1,480,000
20	\$ 1,070,000
27	\$ 1,380,000
28	\$ 1,350,000
29	\$ 2,010,000
30	\$ 3,040,000
31	\$ 10,460,000

Opinion of Probable Cost - Unit Prices					
Item	Size/Capacity	Units		Unit Cost	Additional Details
New Gravity Mains	6	Inches	6" New Sewer	\$ 146.00	Green space
	8	Inches	8" New Sewer	\$ 163.00	Green space
	10	Inches	10" New Sewer	\$ 181.00	Green space
	18	Inches	18" New Sewer	\$ 432.00	Green space
	21	Inches	21" New Sewer	\$ 445.00	Green space
	36	Inches	36" New Sewer	\$ 1,038.00	Green space
New pump stations (wet well, valve vault, pumps, etc)	0.5	MGD	0.5 MGD Pump Station	\$ 150,000.00	Green space
	1	MGD	1 MGD Pump Station	\$ 300,000.00	Green space
	3	MGD	3 MGD Pump Station	\$ 1,400,000.00	Green space
	4	MGD	4 MGD Pump Station	\$ 1,250,000.00	Green space
	6	MGD	6 MGD Pump Station	\$ 1,250,000.00	Green space
New Forcemains (HDPE)	6	Inches	6" Forcemain	\$ 128.00	Green space
	12	Inches	12" Forcemain	\$ 188.00	Green space
Remove & Replace Existing Gravity Mains (PVC)	12	Inches	12" Relief Sewer in grass	\$ 635.00	Green space
	15	Inches	15" Relief Sewer in grass	\$ 710.00	
	18	Inches	18" Relief Sewer in grass	\$ 759.00	
	21	Inches	21" Relief Sewer in grass	\$ 790.00	
	24	Inches	24" Relief Sewer in grass	\$ 845.00	
	27	Inches	27" Relief Sewer in grass	\$ 890.00	
	12	Inches	12" Relief Sewer in pavement	\$ 820.00	Pavement
	15	Inches	15" Relief Sewer in pavement	\$ 895.00	
	18	Inches	18" Relief Sewer in pavement	\$ 984.00	
	21	Inches	21" Relief Sewer in pavement	\$ 1,005.00	
	24	Inches	24" Relief Sewer in pavement	\$ 1,040.00	
	27	Inches	27" Relief Sewer in pavement	\$ 1,095.00	
	30	Inches	30" Relief Sewer in pavement	\$ 1,250.00	
	36	Inches	36" Relief Sewer in pavement	\$ 1,425.00	
New package pump stations	0.5	MGD	0.5 MGD Package Pump Station	\$ 1,100,000.00	
	1.5	MGD	1.5 MGD Package Pump Station	\$ 1,100,000.00	
Decommission Small Pump Station	1.5	MGD	Decommission 1.5 MGD Pump Station	\$ 125,000.00	
Weir Adjustment	1	EA	Weir Adjustment	\$ 100,000.00	

Note: Unit costs are based on the year 2022.

APPENDIX F - LIFT STATION ASSESSMENT

Technical Memorandum

To: Cliff Cate and Rachelle Lowe, Burns & McDonnell
From: Bryan Oakley and Andrea Collier, Barr Engineering Co.
Subject: Lift Station Condition Assessments, 16th and 32nd Street
Date: April 7, 2022
Project: Sedalia Wastewater Master Plan

1.0 Introduction

This memorandum summarizes a condition assessment performed on lift stations for the City of Sedalia (City). The City operates a municipal wastewater collection system which includes 16 lift stations. Lift stations selected for a condition assessment included:

- 16th Street Lift Station
- 32nd Street Lift Station

The City selected these lift stations for condition assessments because there were no capital improvements completed recently or planned at these locations and needs at these locations were unknown.

The condition assessment included the following tasks:

- Review existing information including:
 - Flow and run-time records including:
 - 16th Street Lift Station runtimes from 01/01/2018 to 09/16/2021
 - 32nd Street Lift Station runtimes from 01/01/2018 to 09/16/2021
 - Construction record drawings including:
 - 16th Street Lift Station
 - Generator drawings (no indication of review by original design engineer) printed 12/26/2001
 - Pump parts list dated 10/18/2001
 - Pump shop drawings (reviewed by original design engineer) dated 04/2001
 - 32nd Street Lift Station
 - Pump Specification dated 09/21/1994
 - Pump shop drawings (no indication of review by original design engineer) dated 09/21/1994
 - Note: *Record drawings were not received for either lift station*
 - Operation and maintenance data including:
 - 16th Street Lift Station
 - Generator Maintenance logs from 03/29/2012 to 03/20/2020

- Pump Maintenance logs from 03/29/2012 to 03/20/2020
- Maintenance comments from 04/05/2012 to 11/30/2017
- Other miscellaneous maintenance documents
- 32nd Street Lift Station
 - Generator Maintenance logs from 09/12/2013 to 03/20/2020
 - Pump Maintenance logs from 09/12/2013 to 03/20/2020
 - Maintenance comments from 09/12/2013 to 04/13/2018
 - Other miscellaneous maintenance documents
- Site visits to observe existing conditions – completed on 10/08/2021
- Discussions with City staff to review operation and maintenance challenges

2.0 Lift Station Condition Assessments

2.1 16th Street Lift Station

Figure 1 shows the 16th Street Lift Station and Attachment A shows the Condition Assessment form completed in October 2021.



Figure 1 16th Street Lift Station

The 16th Street Lift Station is a duplex submersible pump lift station with a separate valve vault.

Table 1 16th Street Lift Station Design and Operating Parameters

Parameter	Value
Lift Station Type	Submersible
Year Constructed	2001
Pump Replacement	NA
Control Modifications	NA
Manufacturer	Flygt 3127
Motor Horsepower	7.5
Firm Capacity	252 gallons per minute (gpm) at 45 feet total dynamic head (ft-TDH)
Average Day of Minimum Month¹	6,000 gpd
Peak Day Flow 2019-2021	52,100 gpd
Average Day of Maximum Month²	22,500 gpd

1 – Based on lowest 30-day average daily flow from 1/1/2019 through 9/16/2021

2 – Based on highest 30-day average daily flow from 1/1/2019 through 9/16/2021

2.1.1 Site General Arrangement

The 16th Street Lift Station is secured within a chain-link fence which offers good security and easy access from the near-by right-of-way. Maintenance vehicles equipped with hoists can easily access the wet well for pump removal.

The valve vault is a permit-required confined space entry. The vault does not require frequent access; however, the valves should be exercised periodically to maintain function.

The site has one yard light and a plug-in at the control panel for a work light.

2.1.2 Pumping Equipment

The flow records provided assume that the pumps are currently operating at the original design capacity. This may not be accurate. “Blow by” (leakage between the pump discharge and the base discharge elbow) can reduce the actual flow volume by requiring a portion of the flow to be pumped multiple times. This is a common problem with Flygt metal-to-metal discharge elbows that have been in service for many years. Impeller wear typically reduces pump capacity as pumps age. There is no evidence in the operational records provided by the City that this is an issue.

The pumps are operated using a four-float level control system. Based on past run time and start records, the lead pump on and pump off floats are adjusted periodically.

The pumps were furnished when the lift station was originally constructed in 2001. Pumps have an expected service life of 20 years but commonly remain in service longer than that if the duty is light and the system is well maintained. The Flygt pumps likely have some remaining service life, but the replacement should be expected within the next 10 years.

Pumps are maintained annually by a pump maintenance contractor.

2.1.3 Piping and Valves

The isolation valves and check valves are located in a separate valve vault. The isolation valves should be operated every six months to make sure that they remain operable. Because the vault is a permit-required confined space entry requiring multiple staff for an entry, this may not be done as frequently as commonly recommended.

The valve vault includes a flanged fitting to allow the launch of a pig for cleaning the force main.

2.1.4 Electrical, Instrumentation, and Controls

The control panel has experienced relatively frequent failure of some components.

- The alternator has been replaced multiple times and has not functioned reliably in recent years.
- The generator control panel board had failed, and a replacement had been ordered at the time of the site visit.
- The lift station control panel board had failed and been replaced multiple times in the past.

The lift station power supply is rated at 480 volts but commonly operates at 504 to 505 volts. This is at the top end of acceptable voltage supply ranges and may contribute to reduced service life of some electrical components.

2.1.5 Reliability

The pump alternator has not reliably operated for many years. It has been replaced multiple times. Operators have addressed this by manually alternating pumps. As an example, pumps were manually alternated at intervals ranging up to six weeks from December 11, 2020, to July 15, 2021.

2.2 32nd Street Lift Station

Figure 2 shows the 32nd Street Lift Station, and Attachment B shows the Condition Assessment form completed in October 2021.



Figure 2 32nd Street Lift Station

The 32nd Street Lift Station is a duplex submersible pump lift station with a separate valve vault.

Table 2 32nd Street Lift Station Design and Operating Parameters

Parameter	Value
Lift Station Type	Submersible
Year Constructed	1994
Pump Replacement	NA
Control Modifications	NA
Manufacturer	ABS AFP1001
Motor Horsepower	36
Firm Capacity	390 gpm at 117 ft-TDH
Average Day of Minimum Month¹	26,900 gpd
Peak Day Flow 2019-2021	537,800 gpd
2nd Highest Daily Flow 2019-2021	294,100 gpd
Average Day of Maximum Month²	92,500 gpd

1 – Based on lowest 30-day average daily flow from 1/1/19 through 9/16/21

2 – Based on highest 30-day average daily flow from 1/1/19 through 9/16/21

Based on the review of pump data, the 32nd Street Lift Station appears to have high inflow and infiltration (I/I) during storm events that should be investigated further. Figure 3 shows the daily flow in comparison with the 30-day rolling average flow. Based on this information, the peak daily flow is 20 times the minimum monthly flow and more than five times the peak monthly flow.

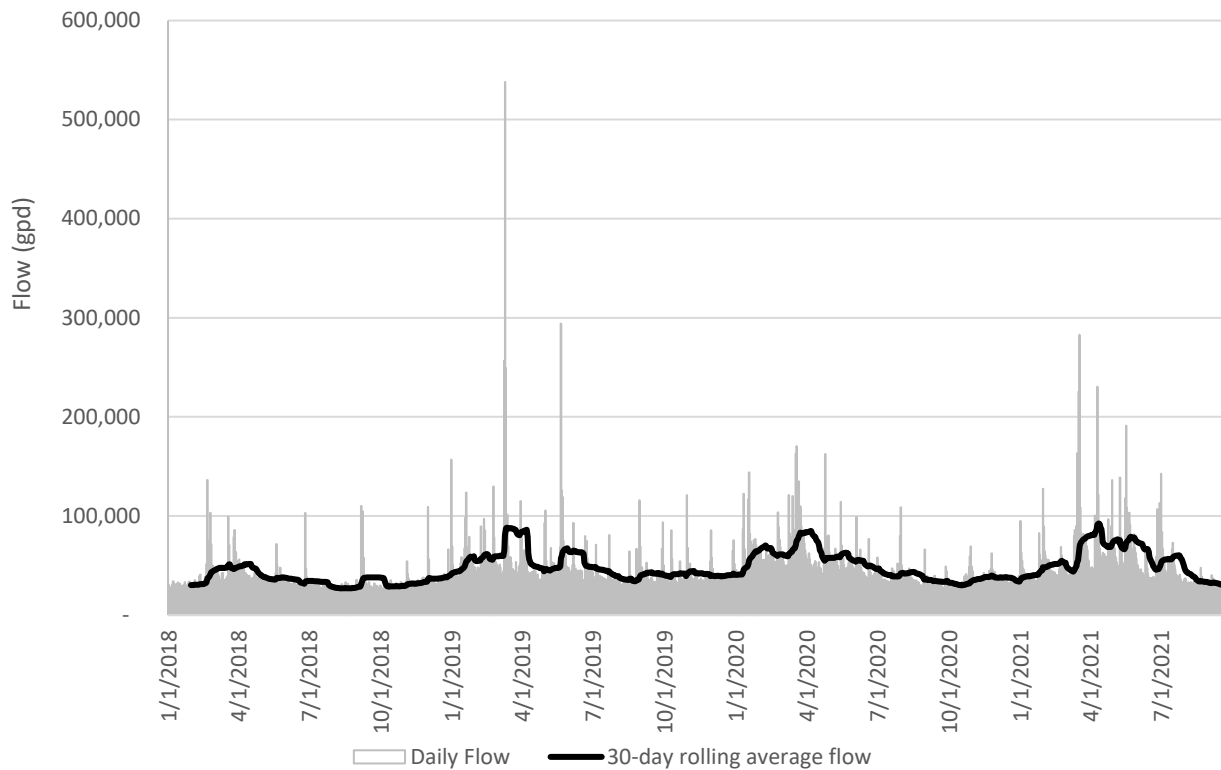


Figure 3 32nd Street Lift Station Flow

2.2.1 Site General Arrangement

The 32nd Street Lift Station is secured within a chain-link fence which offers good security and easy access from the nearby right-of-way. Maintenance vehicles equipped with hoists can easily access the wet well for pump removal.

The valve vault is a permit-required confined space entry. The vault does not require frequent access; however, the valves should be exercised periodically to maintain function.

The site has one yard light and a plug-in at the control panel for a work light.

2.2.2 Pumping Equipment

The flow records provided assume that the pumps are currently operating at the original design capacity. This may not be accurate. "Blow by" (leakage between the pump discharge and the base discharge elbow) can reduce the actual flow volume by requiring a portion of the flow to be pumped multiple times. This is not a common problem with ABS pumps equipped with well-maintained profile gaskets. Maintenance data indicates that the profile gasket is replaced periodically. Impeller wear typically reduces pump capacity as pumps age. There is no evidence in the operational records provided by the City that this is an issue.

The pumps are operated using a four-float level control system. Based on past run time and start records, the lead pump on and pump off floats are adjusted periodically.

The pumps were furnished when the lift station was originally constructed in 1994. Pumps have an expected service life of 20 years but commonly remain in service longer than that if the duty is light and the system is well maintained. The ABS pumps are likely near the end of their useful service life. Replacement should be expected within the next three years.

Pumps are maintained annually by a pump maintenance contractor.

2.2.3 Piping and Valves

The isolation valves and check valves are located in a separate valve vault. The isolation valves should be operated every six months to make sure that they remain operable. Because the vault is a permit-required confined space entry requiring multiple staff for an entry, this may not be done as frequently as commonly recommended.

The valve vault includes a flanged fitting to allow the launch of a pig for cleaning the force main.

2.2.4 Electrical, Instrumentation, and Controls

The control panel is mounted on a weathered plywood board. Information provided indicates relatively frequent failure of a variety of control panel components.

2.2.5 Reliability

The lift station receives an excessive amount of I/I (see Figure 3). During one extreme event, one of the pumps failed, and the other pump ran almost continuously for more than 24 hours. This may indicate the lift station is not sized adequately to handle the peak flows due to I/I. It is possible the pump ran continuously because of a failed float rather than excessive flow. The second-highest recorded flow during the study period was well below the lift station capacity.

Because of the high I/I, the lift station likely receives excessive amounts of grit and other solids in the flow. This increases the wear on the impeller and pump seals. The pumps have experienced seal failures multiple times.

The generator has failed to operate reliably numerous times for a variety of reasons including frozen fuel lines, transfer switch failure, overspeed alarm, battery failures, and starter failure.

3.0 Recommendations

3.1 16th Street Lift Station

- Investigate alternator failures. Repair to allow automatic alternation of pumps.
- Adjust level controls to provide at least two minutes of runtime during a typical cycle.
- Include replacement of pumps and controls in the next 10 years.
- Station runtime and future lift station capacity requirements.
- Consider adding a 5th level control float to provide a redundant pump-off and low-level alarm.

3.2 32nd Street Lift Station

- Evaluate collection system for potential I/I reduction projects to reduce lift station runtime and future lift station capacity requirements. The inflow to the lift station appears to exceed the pump capacity during some wet weather events. This causes extended run time but does not cause excessive frequency of starts.
- Include replacement of pumps and controls in the next 3 years.
- Include replacement of generator in the next 10 years.
- Consider adding a 5th level control float to provide a redundant pump-off and low-level alarm.

Attachments

Attachment A 16th Street Lift Station Condition Assessment Worksheet

Attachment B 32nd Street Lift Station Condition Assessment Worksheet

Attachment A

16th Street Lift Station Condition Assessment Worksheet

Lift Station Condition Assessment Notes

Facility Name:	Sedalia 16th Street Lift Station		
Date:	8-Oct-21		
By:	Bryan Oakley - Barr		
Others Onsite:	Dru Bloess, Dave Gerken, Bob Summers - City of Sedalia; Andrea Collier - Barr		
Year Built:	2001		
	Condition	Function	
Site Design Notes:			
Access:	2	2	
Turf/Landscaping:	2	2	
Expansion Area:	2	2	
Structural Observations:			
General:	2	2	Concrete in good condition
Top slab:	2	-	
Interior concrete:	2	-	
Hatch:	3	4	Does not include slam-proof operation.
Vent:	2	2	
Valve Vault:	4	1	Includes pig launcher.
Pumps:			
General:	3	2	Flygt 3127 pumps
Horsepower:	-	-	7.5
Capacity relative to rating:	-	-	252 gpm @45 TDH from pump curve. 250 gpm per City. Design point is left of peak efficiency.
Replacement parts available and in stock:	-	-	Yes. Regular maintenance performed by pump vendor.
Over-temp protection:	-	-	Yes
Max starts per hour:	-	4	4/hr max allowable, 9-10 calculated at critical flow
Pump-down time:	-	4	1.56 min at zero flow. 2-5 minutes typically recommended
Reliable alternation:	4	4	multiple past failures
Pipes and Valves:			
Valve vault accessible:	3	4	Visible rust. Not regularly exercised.
Actuator on shut-off valve:	3	4	
Check valve operation:	2	2	not applicable
Flow meter installation:			
Flow meter output:			
Piping corrosion:	4	2	coating is deteriorated. Some corrosion visible.
Electrical Equipment Observations:			
Control panel:	3	3	alternator has failed multiple times, circuit board replaced, display failed
Power service:	-	3	480V 3 phase, typically runs at 504 to 505 V
Site lighting:	-	4	One yard light plus plug-in
Generator available:	2	3	Failed in 2013, circuit board out at time of inspection
Fuel tank size:	2	2	100-gal (more than 24 hrs)
Fuel tank containment:	4	4	Spill would result in overflow to surface water
Level control:	3	2	4-float system
VFD:			not applicable
Hoists:			
Condition:			None at lift station. Hoists on service trucks.
Capacity:			
Load test:			
Operator Comments/Observations:			
Power surges contribute to generator and control panel board failures			
Other Historical Information:			
Generator run 1/wk			
Pump maintenance contractor 1/yr			



Condition Rating:

- 1 - New or nearly new
- 2 - Well-maintained and less than expected service life
- 3 - Functional and nearing or past end of service life - Add to future CIP
- 4 - Visible or noted degradation of materials or function - Add to current CIP
- 5 - Severe degradation of materials or function - Failure possible

Function Rating:

- 1 - Superior function - Includes features not typically expected
- 2 - Functions as intended - Typical maintenance requirements
- 3 - Functions as intended - Maintenance and operational requirements exceed typical
- 4 - Function is impaired or inadequate - Not reliable for current service
- 5 - Not functional

Attachment B

32nd Street Lift Station Condition Assessment Worksheet

Lift Station Condition Assessment Notes

Facility Name:	Sedalia 32nd Street Lift Station		
Date:	8-Oct-21		
By:	Bryan Oakley - Barr		
Others Onsite:	Dru Bloess, Dave Gerken, Bob Summers - City of Sedalia; Andrea Collier - Barr		
Year Built:	1994		
	Condition	Function	Comments
Site Design Notes:			
Access:	2	2	
Turf/Landscaping:	2	2	
Expansion Area:	2	2	
Structural Observations:			
General:	2	2	Concrete in good condition
Top slab:	3	-	
Interior concrete:	4	-	Does not include slam-proof operation.
Hatch:	4	4	
Vent:	-	-	No vent
Valve Vault:	3	2	
Pumps:			
General:	3	3	ABS 1001
Horsepower:	-	-	33.5
Capacity relative to rating:	-	-	390 gpm @117 ft TDH from pump curve. 389 gpm per City. Design point is left of peak efficiency.
Replacement parts available and in stock:	-	-	Yes. Regular maintenance performed by pump vendor.
Over-temp protection:	-	-	Yes
Max starts per hour:	-	2	4/hr max allowable, 3 calculated at critical flow
Pump-down time:	-	2	5 min at zero flow. 2-5 minutes typically recommended
Reliable alternation:	3	3	
Pipes and Valves:			
Valve vault accessible:	2	3	Visible rust. Not regularly exercised.
Actuator on shut-off valve:	3	4	
Check valve operation:	2	2	not applicable
Flow meter installation:			
Flow meter output:			not applicable
Piping corrosion:	4	2	coating is deteriorated. Some corrosion visible.
Electrical Equipment Observations:			
Control panel:	3	3	Mounted on plywood board
Power service:	-	3	480V 3 phase
Site lighting:	-	4	One yard light plus plug-in
Generator available:	2	3	Failed in 2013, circuit board out at time of inspection
Fuel tank size:	2	2	100-gal (24 hrs under normal operation. Less for wet weather)
Fuel tank containment:	3	3	
Level control:	3	2	4-float system
VFD:			not applicable
Hoists:			
Condition:			None at lift station. Hoists on service trucks.
Capacity:			
Load test:			
Operator Comments/Observations:			
High I/I. Peak is more than 15 times average dry weather. Pump failure during wet weather event caused one pump to run continuously for approximately 38 hours			
Other Historical Information:			
Generator run 1/wk Pump maintenance contractor 1/yr			





Condition Rating:

- 1 - New or nearly new
- 2 - Well-maintained and less than expected service life
- 3 - Functional and nearing or past end of service life - Add to future CIP
- 4 - Visible or noted degradation of materials or function - Add to current CIP
- 5 - Severe degradation of materials or function - Failure possible

Function Rating:

- 1 - Superior function - Includes features not typically expected
- 2 - Functions as intended - Typical maintenance requirements
- 3 - Functions as intended - Maintenance and operational requirements exceed typical
- 4 - Function is impaired or inadequate - Not reliable for current service
- 5 - Not functional



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