BETHEL LIFT STATION STORAGE PROJECT

FINAL DESIGN REPORT

PREPARED FOR:
CLERMONT COUNTY WATER RESOURCES DEPARTMENT
BATAVIA, OHIO

Clermont County
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FINAL

Fishbeck, Thompson, Carr & Huber
engineers • scientists • architects • constructors

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LIST OF ABBREVIATIONS/ACRONYMS

Ac	acres
ADWF	Average Dry Weather Flow
CCTV	Closed Circuit Television
cfs	cubic foot/feet per second
cft	cubic foot/feet
cfy	cubic feet per yard
DWF	Dry Weather Flow
EL	elevation
ft	foot/feet
FTC&H	Fishbeck, Thompson, Carr & Huber, Inc.
GIS	Geographic Information System
gpcd	gallons per capita per day
gpm	gallons per minute
I/I	Inflow/Infiltration
LF	linear foot/feet
MEF	Middle East Fork
MG	million gallons
MH	Manhole
N/A	not applicable
PVC	polyvinyl chloride
QA/QC	Quality Assurance/Quality Control
RDII	Rainfall Derived Inflow and Infiltration
RFP	Request for Proposal
ROW	right-of-way
SSES	Sewer System Evaluation Survey
TAZ	Traffic analysis zone
USACE	U.S. Army Corp of Engineers
VCP	Vitrified Clay Pipe
WaPUG	Wastewater Planning User's Group
WWTP	wastewater treatment plant

1.0 INTRODUCTION

1.1 PROJECT OBJECTIVES

One of the projects out of the Clermont County Water Resources Department Wastewater Master Plan Update (September 2011) and included in the current Five Year Capital Improvement Plan is the addition of 1.5 million gallons of storage at the Bethel Lift Station. The lift station currently has approximately 880,000 gallons of storage. Storage is required at the lift station because capacity restrictions downstream do not permit an increase of pumping rates from the station. The sizing of the additional storage was approximate since there was no model calibration performed upstream of the existing lift station. The scope of this project included:

1. Installation of flow monitoring and rain gauge equipment to obtain data sufficient to calibrate the Department's collection system model.
2. Calibration of the model upstream of the Bethel Lift Station.
3. Determination of additional storage requirements at the lift station.
4. Identify necessary collection system modifications to enable the conveyance of wet weather flows to the lift station.
5. Design of the required storage facilities including construction drawings and technical specifications.

1.2 EXISTING DATA REVIEW

A number of existing data sources were reviewed to gain an understanding of the history of the proposed project and establish a baseline of data that could be used in the evaluation of the lift station and its tributary collection system. These existing data sources included:

- The Clermont County Water Resources Department Wastewater Master Plan Update (September 2011). The Master Plan was reviewed to understand the development of the Bethel Lift Station Storage Project and the parameters that were used for the determination of the initial sizing. It was also used to establish model parameters such as design storms that should be applied to the modeling efforts in this project.
- GIS data for the Clermont County Water Resources wastewater system.
- The County's collection system model in InfoWorks CS. The model was reviewed to prepare for calibration as well as to determine methodology for future flow projections and storage requirements at the Bethel Lift Station.
- Construction drawings of the existing lift station and storage tank. These drawings provided an understanding of the existing lift station equipment and operation as well as parameters that would be needed for model calibration.

- Field visit & interviews with County personnel. Several field visits were made and included discussions with those familiar with the lift station operation equipment. These discussions provided valuable data that established lift station operating parameters that would be included in the hydraulic model.

1.3 TASKS COMPLETED FOR PRELIMINARY DESIGN REPORT

The first task was to establish appropriate flow monitor locations in the collection system that would provide the best data to calibrate the existing hydraulic model. After identifying these locations five flow monitors and one rain gauge were installed to collect three months of flow data. The initial flow monitoring period did not provide sufficient wet weather data, so a second flow monitoring period was needed to obtain data that would allow for proper model calibration. The flow data was then used to calibrate Clermont County's existing InfoWorks model that would be used to recommend collection system improvements as well as to assess the storage requirement at the Bethel lift station for the 5-year, 6-hour design storm.

An area survey was performed for the lift station site that located the existing features and utilities and provided the necessary topographic data to assess location options for the additional storage facilities.

A geotechnical investigation was made to establish subsurface conditions that could affect foundation design as well as establish bedrock elevations. During construction of the existing storage facilities, there were indications that rock elevation could have a significant effect on construction costs that may make an above ground tank more cost effective.

2.0 FLOW MONITORING PROGRAM

A flow monitoring program was conducted to collect flow data to:

- Characterize flows in the collection system during dry weather and wet weather conditions.
- To provide data for calibration of the hydraulic model.
- To identify portions of the collection system that may be subject to excessive infiltration and inflow.

Five (5) Isco 2150 Area Velocity flow monitors were installed in the collection system and one (1) rain gage was installed at the Bethel Lift Station. Flow data and rain gage data were collected from May 18 – July 16, 2012. After reviewing the flow and rainfall data during the study period and performing an initial calibration of the model, it was determined that dry antecedent ground conditions and lack of rainfall during the monitoring period did not produce flows similar to those in the past when Bethel experienced capacity issues or overflows at the Bethel Lift Station. It was decided that the flow monitors should be reinstalled in the spring of 2013 to obtain more representative data and provide a better calibration of the model suitable for sizing additional storage at the lift station.

The flow meters and rain gage were re-installed in the same locations from March 28 – July 2, 2013. A more representative data set was obtained that allowed for better model calibration and a higher level of confidence in model projections.

2.1 OBJECTIVE

The objective of the flow monitoring program was to collect data characterizing the flow through the collection system. This data established base dry weather flows, diurnal flow patterns and documented flow increases in the collection system during various wet weather events. The flow data was then used during calibration of the hydrologic and hydraulic model using InfoWorks. Calibration of the model is discussed later in this report. The purpose of the hydraulic model is to be able to predict flows through the collection system during both dry and wet weather conditions as well as established design storms. In order to accurately predict the flows through the system, the model should be calibrated by as many wet weather events as possible with varying storm volumes, intensities, and durations. The flow data collected from April-July represent both spring and summer flows in the collection system and are satisfactory for the purpose of this project.

2.2 FLOW MONITORING LOCATIONS

The locations for the five flow monitors were strategically selected to characterize flows at various points in the collection system by dividing it into smaller segments. Flow monitor locations were selected to characterize as much of the system as possible and provide enough detail for calibration of the hydraulic model. Other factors taken into account for the locations of the flow meters were site access, safety, and manhole conditions. Figure 2.1 shows the locations of each of the five flow monitors.

2.3 MONITORING RESULTS – 2012

The flow monitoring data and rain gage data were collected and reviewed for QA/QC purposes after each download. The flow monitoring data was reviewed to identify the dry weather flow, diurnal pattern, and a response in flow during wet weather at each meter location. The data collected from each monitor location was reliable and of good quality. Table 2.1 shows a table of average dry weather flow (ADWF) at each of the metered locations.

Table 2.1 - 2012 Average Dry Weather Flows

Meter	ADWF Weekday (gpm)	ADWF Weekend (gpm)
Meter 1 - Grant	141	151
Meter 2 - Circus	26	26
Meter 3 - Bone	42	44
Meter 4 - Burke	55	55
Meter 5 - Water	38	38

The rainfall events during the 2012 monitoring period were typical of summer rainfall with high intensity short duration storms. However, there were no periods of extended rainfall that would saturate the ground and cause long term infiltration. Table 2.2 shows the rainfall distributions and recurrence intervals for storms in 2012.

Table 2.2 - 2012 Rainfall Distribution Table

Storm Rainfall Distributions								
Storm Period	5 min		1 hr		6 hr		24 hr	
	Rainfall (in)	Recurrence Interval	Rainfall (in)	Recurrence Interval	Rainfall (in)	Recurrence Interval	Rainfall (in)	Recurrence Interval
May 21, 2012	0.13	2 month	0.69	3 month	0.91	2 month	0.91	< 2 month
June 1, 2012	0.06	< 2 month	0.24	< 2 month	0.52	< 2 month	0.74	< 2 month
June 17, 2012	0.17	< 2 month	0.38	< 2 month	0.46	< 2 month	0.94	< 2 month
July 18-21, 2012	0.15	2 month	0.34	< 2 month	0.73	< 2 month	0.98	< 2 month
July 23-26, 2012	0.22	6 month	0.38	< 2 month	0.44	< 2 month	0.44	< 2 month
July 26-29, 2012	0.22	6 month	1.09	1 year	1.22	4 month	1.71	4 month
August 10, 2012	0.08	< 2 month	0.39	< 2 month	0.5	< 2 month	0.62	< 2 month

In general, all of the meters showed a response to rainfall similar to inflow where the flow rises and returns to DWF quickly. This is typical of catch basin connections, roof drains, and other direct connections. None of the meters showed responses resembling an infiltration response where flows take much longer to recede to DWF levels. It was decided to reinstall the flow meters in the spring of 2013

when the ground is saturated to view how the system reacts in the spring when the ground is typically saturated due to more frequent rainfall.

2.4 MONITORING RESULTS – 2013

In 2013 the flow meters and rain gage were installed in the same locations as in 2012. Other flow meter sites were investigated on the 18" pipe along Town Run leading to the lift station but were not used due to site accessibility and safety concerns. The flow meters were installed March 28 and removed July 2. Table 2.3 shows the ADWF as it compares to the ADWF observed during 2012.

Table 2.3 - Differences in DWF from 2012 to 2013

Meter	2012 ADWF (gpm)	2013 ADWF (gpm)	Difference (gpm)
Meter 1 - Grant	141	224	83
Meter 2 - Circus	26	40	14
Meter 3 - Bone	42	57	15
Meter 4 - Burke	55	86	31
Meter 5 - Water	38	48	10

Notice the ADWF at each of the meter locations was higher in 2013 than it was in 2012. This difference in ADWF can be described as the "base infiltration" or the amount of infiltration from groundwater during spring months. In addition there were two additional inches of rainfall during the study period in 2013 than there was in 2012. Table 2.4 shows the rainfall distributions for the storms in 2013.

Table 2.4- 2013 Rainfall Distribution Table

Storm Rainfall Distributions								
Storm Period	5 min		1 hr		6 hr		24 hr	
	Rainfall (in)	Recurrence Interval	Rainfall (in)	Recurrence Interval	Rainfall (in)	Recurrence Interval	Rainfall (in)	Recurrence Interval
April 10-15, 2013	0.06	< 2 month	0.33	< 2 month	0.54	< 2 month	0.79	< 2 month
April 18-22, 2013	0.07	< 2 month	0.34	< 2 month	0.52	< 2 month	0.52	< 2 month
April 23-27, 2013	0.09	< 2 month	0.3	< 2 month	0.69	< 2 month	0.69	< 2 month
June 10-15, 2013	0.10	< 2 month	0.3	< 2 month	0.63	< 2 month	0.63	< 2 month
June 25-20, 2013	0.13	< 2 month	0.33	< 2 month	0.9	< 2 month	1.22	< 2 month

The storm events in 2013 did not produce any events with a significant recurrence interval (3-month, 24-hour storm) but the flow results were different than in 2012. The peaking factors were similar between 2012 and 2013 but in 2013 it took much longer for the flow to return to DWF once the storm had passed.

This is typical of the difference between ground that is dry and ground that is saturated causing long term/groundwater infiltration.

3.0 MODEL CALIBRATION

The most important aspect of any modeling work is ensuring the model is up to date and properly calibrated. Calibration is an iterative process that involves adjusting model parameters to achieve a reasonable agreement between observed flow data and modeled flows under dry and wet weather conditions. The Wastewater Planning User's Group (WaPUG) has developed a set of industry guidelines for calibration of hydraulic models to help determine the accuracy of the model calibration. Table 3.1 shows the WaPUG guidelines' criteria and ranges for dry weather flow, wet weather flow, wet weather volume, and the peak depth that the modeler should attempt to adhere to. In addition to the criteria described above, the modeler also should attempt to match the shape of the modeled hydrograph to the observed hydrograph as much as possible.

Table 3.1 - WaPUG Guidelines for Hydraulic Modeling

Criteria	Calibration Standard
Dry Weather Flow Rate	-10% to +10% of measured
Dry Weather Flow Volume	-10% to +10% of measured
Wet Weather Flow Rate	-15% to +25% of measured
Wet Weather Flow Volume	-10% to +20% of measured
Peak Depth	-0.33 feet to +1.67 feet at surcharged locations
	±0.33 feet at non-surcharged locations

3.1 GENERAL

The existing InfoWorks model was provided by Clermont County Water Resources Department. The system wide model for Clermont County was originally developed in HydroWorks but was recently converted to the InfoWorks CS model. The system wide model is separated into several collection systems, each of which is tributary to one of the five WWTPs in Clermont County. Bethel is tributary to the Middle East Fork Treatment Plant (MEF) and is conveyed via 5.5 miles of 10-inch force main from the Bethel lift station.

Originally all wet weather flow entering the sanitary collection system was routed via direct runoff from subcatchments. When the original model was compared against the flow data, the simulated peak flows and wet weather volumes were much higher than the observed flow data. The decision was made to convert all direct runoff to wet weather entering the system through Rainfall Derived Inflow and Infiltration (RDI/I) because of the ease of using RTK parameters during calibration and because it is typically used to simulate the effect of wet weather in a sanitary sewer. The following sections further describe the calibration events and the results.

3.2 CALIBRATION EVENTS

The goal of a calibrated hydraulic model is to be able to simulate the hydrologic and hydraulic response in a collection system for a range of events. With that in mind, the model was calibrated to as many events as possible; five storms in total for 2013 which are listed in Table 2.4. The 2012 flow monitoring and storm events were not used in the model calibration because of the dry antecedent moisture and ground conditions. The results from 2012 were used for verification only.

The only significant anomaly in the flow data during wet weather events was from May 6-12, 2013. The flow monitoring data indicated significant increases in flow rates at each flow monitor, but there was not a significant amount of rain observed at the rain gage during that time period. Secondary sources for rainfall were referenced but no significant rainfall was observed for the time period in May. Therefore, this event was not used as a valid event for calibration.

3.3 CALIBRATION RESULTS

Appendix A shows a summary of the calibration results for each of the meter locations as well as a flow hydrograph comparing the observed data vs. modeled results. In general, the model calibrates well with the collected flow data. Below is a summary of the overall calibration results:

- Meter 1- Volume and peak flow are within acceptable ranges events. All peak depths are within range
- Meter 2 – Two events are in range for total volume, 3 events are within range for peak flow and all events are within range for peak depth
- Meter 3 – Three events are in range for total volume, 4 events are within range for peak flow and all events are within range for peak depth
- Meter 4 – Three events are in range for total volume, all events are within range for peak flow and one event is within range for peak depth
- Meter 5 – Two events are in range for total volume, 4 events are within range for peak flow and all events are within range for peak depth

Ideally all events of varying storm sizes, durations and intensities match the hydraulic model; realistically calibration of a hydraulic model becomes a balancing act. The goal was to calibrate the model to at least 3 of the 5 storms. In the instances where the calibration criteria only matches 2 of the 5 storms, it is important to not sway the model in one direction (over-predicting or under-predicting) in favor of satisfying the calibration criteria. For instance, the total flow volume for Meter 5 is within range for two storms and the other three are just high and just low, swaying the calibration in either direction would result in throwing off the calibration for the other events. It is important to note that the calibration for Meter 1, downstream of the other four meters, matches very well and is a good indicator the model is properly predicting flows from the collection system to the Bethel lift station.

Once the model was calibrated, storms causing recorded overflows at the Bethel lift station were simulated as a way of verifying the model calibration. Table 3.2 shows the list of overflow events provided by the County.

Table 3.2 – Bethel Lift Station Sanitary Sewer Overflows

Date	Duration	Time	Flow Estimate (gallons)
05/02/2010	5 HOURS	3:33PM to 11:28PM	125,000
02/25/2011	14 HOURS	7:58AM to 10:05PM	250,000
03/10/2011	8 HOURS	12:50AM to 8:10PM	145,000
04/12/2011	14.5 HOURS	8:25AM to 11:00PM	260,000
04/20/2011	15 HOURS	8:37AM to 11:47PM	375,000
04/23/2011	4 HOURS	8:00PM to 12:00AM	100,000
05/03/2011	13.5 HOURS	9:33AM to 10:00PM	250,000
11/28/2011	25 HOURS	11/28 2:40PM to 11/29 3:30PM	800,000
12/05/2011	16 HOURS	12/5 9:00PM to 12/6 1:15PM	500,000

The volumes of the overflows were determined by an instantaneous level measurement in the Parshall flume and estimating the duration of the overflow. The rainfall was obtained from weatherunderground.com for nearby Batavia because rainfall information was not available for Bethel. The rainfall was checked against other weather stations as well as other sources to verify rainfall amounts were similar between sources and weather stations. The calibrated model did not indicate an overflow for any of these events. Almost all of these events occurred in 2011, particularly in April. 2011 established a new record for rainfall in the area, exceeding the old record by over 10-inches and April was the wettest month of the year. There are a number of possible causes for these overflow events including extreme high groundwater conditions, high water levels in streams and water entering manholes or other outfalls that would not occur under less than record rainfall conditions. The decision was made not to attempt to adjust the model calibration to match conditions experienced only in the wettest year on record.

4.0 GEOTECHNICAL FINDINGS

A geotechnical investigation was performed by Thelen Associates, Inc. September 4, 2012. The full geotechnical report can be found in Appendix B. The following summarizes their findings.

4.1 OBJECTIVES

The original scope of this project was to design a 1.5 MG storage tank located on a property to the southwest of the Bethel lift station. The purpose of the geotechnical investigation was to determine the general subsurface profile in the area of the proposed tank and to locate the elevation of rock in the area. Rock was encountered during the construction of the Bethel lift station and is a concern for the construction of the proposed storage tank.

4.2 BORING LOCATIONS

To determine the subsurface profiles at the site, six (6) test borings were performed. The locations of the test borings were determined by FTCH and the locations and elevations of the boring locations were staked by G. J. Berding Surveying, Inc. Appendix B contains the geotechnical report which shows the locations of the soil borings.

4.3 GEOTECHNICAL DATA AND RESULTS

Split-spoon samples were obtained at each of the boring locations using the standard drive sample method and rock samples were obtained by rock coring with a diamond tipped core barrel. Surface materials at the site were comprised of mainly stiff to very stiff silty clay and some shale. All of the borings were extended a minimum of 2 samples into bedrock, interbedded shale and limestone. The depths to the surface of the bedrock varied between 4.5 feet and 9.5 feet. In addition, trace water was encountered in the test borings between 7 and 11 feet.

4.4 BORING LOGS

Results from the sample borings can be found in the full geotechnical report in Appendix B.

5.0 ALTERNATIVES DEVELOPMENT AND ANALYSIS

The goals of this project were to determine how much, if any, additional storage is needed at the Bethel Lift Station and to identify capacity issues within the collection system. Following the methodology in the Clermont County Wastewater Master Plan, the collection system was reviewed for current flows and 2030 flows using the 5-year, 6-hour storm. Methodology for determining 2030 flows is discussed later in this document.

5.1 PROJECTING FUTURE FLOWS

The methodology in the Master Plan called for projecting future wastewater production based on developable land within Traffic Analysis Zones (TAZs). For this project, the developable areas were based on the 2030 future condition model provided by Clermont County. Figure 5.1 shows the developable areas. The methodology for determining the amount of additional flow for 2030 is outlined in the Wastewater Master Plan Update where for pre-defined areas:

- RDII factors used a 0.5% R1 factor for inflow and 0.5% R3 factor for infiltration. No factor for R2 was used.
- Groundwater was allocated at 30 gal/capita/day (gpcd).
- Additional wastewater was 100 gpcd

The results of this analysis are discussed later in this document. In addition to reviewing how the system today would react to a design event, these 2030 flows were reviewed to ensure any work to the collection system not only fixes today's problems but is built with future growth in mind. Designing collection system improvements based on these flows ensures the collection system is sustainable.

5.2 STORAGE

The original scope for this project was centered around the design of a 1.5 MG storage tank. This was based on model runs utilizing the existing un-calibrated hydraulic model for 2030 flows as well as the occurrence of a number of overflow events at the lift station, mostly in 2011. After calibration of the model based on the flow monitoring data, a number of simulations were made to determine what additional storage would be required to meet the County's design standards. The 5-year, 6-hour design storm was simulated utilizing the Wastewater Master Plan methodology for determining 2030 flows (Section 4.4 Future Wastewater Production, Wastewater Master Plan Update). Assuming the tank is cleaned out and the full capacity of the storage tank at the lift station is available, model results indicated that no additional storage will be required.

Because the model calibration falls within the accepted parameters established by WaPUG and the calibration curves (Appendix A) show a good match between modeled and actual flows, the only factor that may point toward additional storage at the lift station are the recorded overflow events in 2011. As discussed in Section 3.3 - Calibration Results, the year 2011 was a record setting year for rainfall in the area. It is not in the best interest of the County to construct additional storage at this time based solely on these events. Therefore, it is recommended that additional storage not be constructed at this time and the County's efforts be directed at collection system improvements such as increasing wet weather flow capacity to reduce system back-ups and identifying and correcting excessive infiltration and inflow that can reduce wet weather flows at the lift station.

5.3 COLLECTION SYSTEM

Using the calibrated model, the collection system tributary to the lift station was assessed to identify capacity issues and determine improvements that would allow wet weather flow to be transported to the lift station and reduce or eliminate system back-ups. There are several known capacity issues in the collection system, most notably the stretch at Plane Street through Burke Park to the intersection of Plane Street and Rich Street. Flow data from the meter in Burke Park showed peaking factors between 6 and 9 which suggests an inflow response. Additionally there are 5 miles of pipe tributary to Burke Park which were installed in 1941 indicating there may be some potential inflow sources as well as leaks associated with older pipes. Table 5.1 summarizes the pipe replacement that would be required to provide sufficient collection system capacity to prevent significant surcharging for the 5-year, 6-hour design storm for both current and future (2030) wet weather flows. Locations with less than three feet of surcharging were not deemed a problem area as stated in the Master Plan. Figures 5.2 and 5.3 show the pipe replacements required to transport wet weather flows for 2013 and 2030 flows respectively.

It should be noted that pipe replacements upstream of flow meter locations are based on the standard model parameters, but additional flow monitoring would be needed to verify specific pipes that should be upsized and verify pipe sizes.

Table 5.1 – Bethel Pipe Replacements

Scenario	24" Pipe (LF)	18" Pipe (LF)	15" Pipe (LF)	12" Pipe (LF)	10" Pipe (LF)	8" Pipe (LF)	Total (LF)
2013 Upsized Pipes	3,755	600	2,615	6,775	1,305	2,570	17,620
2030 Upsized Pipes	3,755	2,715	2,260	9,135	2,445	1,425	21,735

5.4 INFLOW REMOVAL

The previous section outlined collection system upgrades that would be required assuming no I/I is removed from the system. The flow monitoring indicated that there is significant I/I in portions of the collection system that, if some removals are made, can reduce the extent of pipe replacement. Since there has been no comprehensive testing performed to identify I/I sources, several scenarios were examined to determine the potential savings that could be realized by assuming various removal potentials. Therefore, the collection system was evaluated by simulating 25%, 50% and 75% removals of inflow.

For the alternatives to focus on effects of I/I removal, areas were targeted where flow peaking factors (flow meter data) indicated a significant inflow response. The hydrologic model was created using a calibration method known as RTK, which uses a combination of three triangular unit hydrographs to approximate the storm response. Figure 5.4 shows an example of a component of the RTK hydrograph and Figure 5.5 shows an example of a total RTK hydrograph.

Figure 5.4 – Component of RTK hydrograph

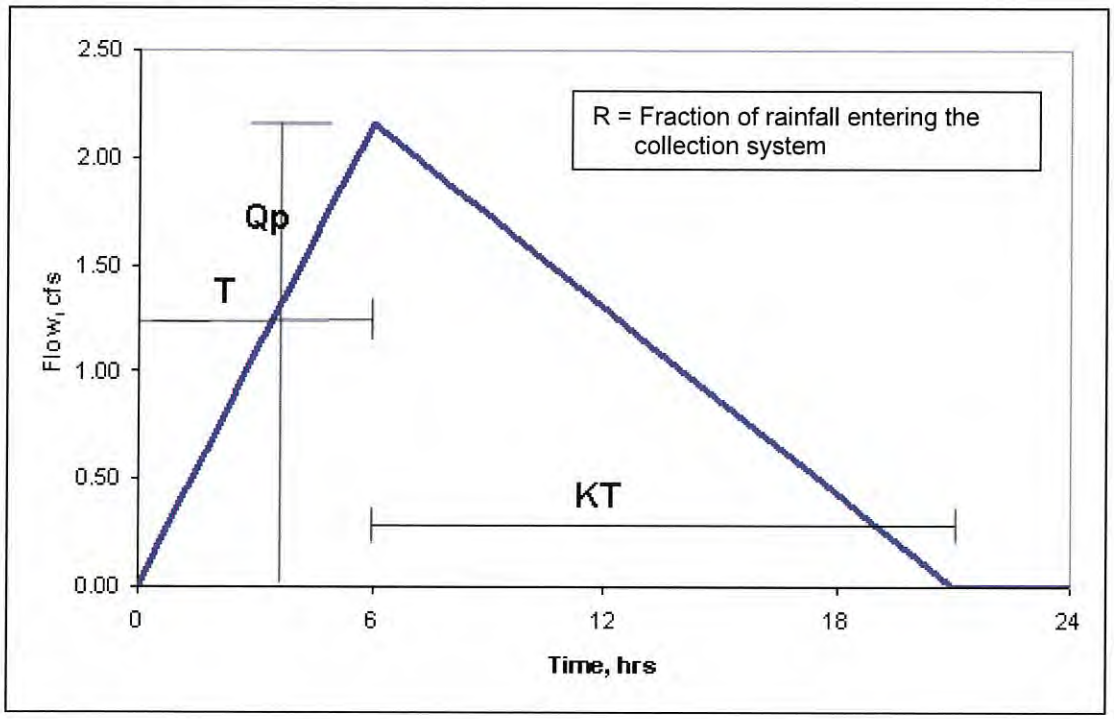
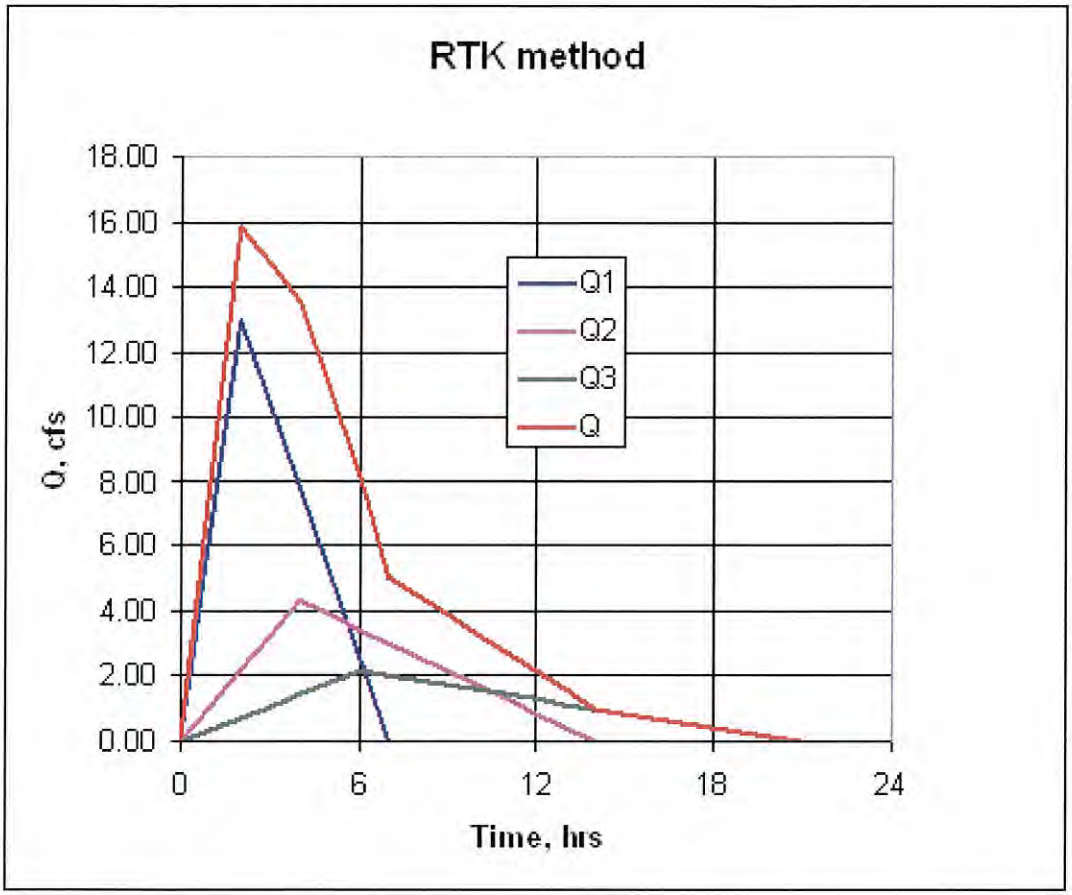


Figure 5.5 – Total RTK hydrograph



The R parameter represents the fraction of rainfall volume entering the sewer system as RDII. R1 is the portion of the hydrograph attributed to inflow, while R2 and R3 represent short term and extended infiltration into the system. The T parameter is the time to the peak for each of the unit hydrographs and the K parameter is the ratio of the time to recession and the time to peak. The total RDII is equal to the sum of inflow and infiltration, i.e. Q1, Q2 and Q3 and is used to estimate the storm response at each calibrated flow monitoring location.

To predict the flows in the collection system based on inflow removals, the R1 parameter was reduced by 75%, 50%, and 25% to simulate the system response assuming different effective rates of inflow removal. For example, assuming an effective inflow removal rate of 75%, the remaining 25% was left in the model as it is unlikely all the inflow can be removed. Three different removal rates were analyzed because of the current unknown factors associated with types of inflow and possibility and ease of removal. Figures 5.6 – 5.11 show the targeted areas for inflow removal for both the current condition (2013) and for the future (2030) design year. Even with inflow removal, some areas of the collection system will still be under capacity due to shallow pipe slopes. The amount of proposed pipe needed to provide sufficient capacity is listed in Table 5.2 and the locations of these pipes are shown in Figures 5.6 – 5.11.

Table 5.2 – Bethel Pipe Replacements with Inflow Removals

Scenario	24" Pipe (LF)	18" Pipe (LF)	15" Pipe (LF)	12" Pipe (LF)	10" Pipe (LF)	8" Pipe (LF)	Total (LF)
2013 Upsized Pipes	3,755	600	2,615	6,775	1,305	2,570	17,620
2013 25% Inflow Removal	3,755		2,715	3,145	3,045	2,570	15,230
2013 50% Inflow Removal	3,755		2,715	2,445	2,670	2,570	14,155
2013 75% Inflow Removal			600	3,195	2,670	2,570	9,035
2030 Upsized Pipes	3,755	2,715	2,260	9,135	2,445	1,425	21,735
2030 25% Inflow Removal	3,755	2,715	2,260	8,765	2,815	1,425	21,735
2030 50% Inflow Removal	3,755	1,595	3,380	8,060	2,820	1,425	21,035
2030 75% Inflow Removal	3,755		4,975	8,060	1,140	1,425	19,355

Table 5.2 lists the required pipe replacements that would be necessary for 2013 flows and 2030 flows assuming various inflow removal rates (25%, 50% and 75%). While replacement pipe lengths are reduced with additional inflow removals for 2013 flows, they are similar for all removals for 2030 flows unless 75% inflow removal can be realized. Any pipes that are replaced should be designed to handle 2030 flow rates, therefore, it is unlikely that inflow removals will have a significant effect on replacement pipe sizes. Inflow removals can still affect lift station operations and the need for future storage.

Table 5.3 shows preliminary cost estimates for the pipe replacements for various alternatives.

Table 5.3 – Cost Estimates for Pipe Replacement

Scenario	Total length of pipe (LF)	Cost Estimate ($)
2013 Upsized Pipes	17,620	3,336,000
2013 25% Inflow Removal	15,230	2,801,000
2013 50% Inflow Removal	14,155	2,562,000
2013 75% Inflow Removal	9,035	1,374,000
2030 Upsized Pipes	21,735	4,228,000
2030 25% Inflow Removal	21,735	4,189,000
2030 50% Inflow Removal	21,035	4,028,000
2030 75% Inflow Removal	19,355	3,563,000

5.5 ALTERNATIVE SELECTION

Any pipe replacements should be sized to handle 2030 flows in the system. At this time, it is unknown what inflow removal rates may be achievable. Historically, inflow removals of 75% are rare in a sanitary sewer system. However, given the age of the system and the number of creek crossings, potential removals could be significant in some areas. Additional sewer system investigation can provide a better understanding of inflow sources and the potential for eliminating or reducing these flows.

Initially it is recommended to replace piping from Plane St. and Rich St. through Burke Park to Osborne St. The pipe from Plane and Rich St to Water and Main is a 12" and from Water and Main to Osborne should be replaced with an 18" pipe. Some of the pipe through this area was installed in 1941 and crosses the creek 5 times. Replacing these pipe sections is recommended for the following reasons:

1. Much of the pipe was installed in 1941 and has reached its expected useful life.
2. Replacing this section can provide some insight into flow reductions that can be expected from replacing the older pipes in the collection system.
3. It is a common pipe replacement to all alternatives.
4. It will solve flooding issues at Plane and Rich St. and in Burke Park.

Figure 5.12 shows the locations of the proposed pipe replacements.

6.0 RECOMMENDATIONS AND CONCLUSIONS

As stated previously, the original scope for this project was centered around the design of a 1.5 MG storage tank. This was based on model runs utilizing the existing un-calibrated hydraulic model for 2030 flows as well as the occurrence of a number of overflow events at the lift station, mostly in 2011. After calibration of the model based on the flow monitoring data, a number of simulations were made to determine what additional storage would be required to meet the County's design standards. Assuming the tank is cleaned out the full capacity of the storage tank at the lift station is available, model results from the 5-year, 6-hour design storm indicated that no additional storage will be required. Therefore, it is recommended that additional storage not be constructed at this time and the County's efforts be directed at collection system improvements such as increasing wet weather flow capacity to reduce system back-ups and identifying and correcting excessive infiltration and inflow that can reduce wet weather flows at the lift station. The following section explains in further detail the proposed maintenance and improvements to the collection system.

6.1 LIFT STATION OPERATION AND MAINTENANCE

Lift station operation and maintenance can affect both hydraulic levels in the 18-inch sewer upstream of the storage tank as well as limiting capacity in the tank thus increasing potential for overflows. One issue that can affect the storage capacity at the lift station is the availability of the full tank capacity during a storm event. There is often anywhere from 2.5 to 3.0 feet of water in the tank which reduces the tank's design storage capacity by as much as 15%. Cleaning of the tank and pumping out this residual water would provide additional insurance against overflows during storms greater than the 5-year, 6-hour event and particularly when development occurs and the flows approach those projected for the year 2030.

One manhole upstream of the storage tank (MH 12958) shows some visible evidence of past surcharging. The rim elevation of this manhole is approximately one foot lower than the overflow elevation for the tank. Therefore, flooding at this manhole can occur before the tank overflows. Surcharging in this line is unavoidable, but the flooding at the manhole can be prevented by either raising the manhole or installation of a solid bolt down lid. The bolt down lid would be the least expensive option since flooding at the manhole should occur only occasionally.

6.2 EXISTING 18-INCH SEWER

The most downstream flow meter location was at MH 2540 which is just east of Route 125. It is possible that there could be excessive I/I entering the pipe between MH 2540 and the lift station that would not be accounted for in the model calibration since it is downstream of the first flow meter. There is not a significant sanitary contribution in this sewer, but several factors could affect I/I in the pipe. Although this pipe is only 30 years old and is PVC, it crosses under Town Run 5 times, is tied into existing manholes which have observed infiltration and runs in/near wooded areas which may have cracked the pipes.

Clermont County performed manhole inspections for this stretch of 18" sewer and observed some infiltration as well as evidence of water topping and entering the manhole.

It is recommended the County repair any known infiltration areas and damaged cone sections where water may be able to enter. In manholes where there is evidence of water topping the manhole and entering the collection system, watertight lids should be installed. According to Neenah Foundry Company test results on their manhole lids, a 1-inch diameter vent hole will allow 4.4 gpm to enter the manhole with only 1-inch of head on it. This can add up to significant flows if there are multiple vent holes and higher hydraulic heads. Table 6.1 summarizes the manhole inspection along the 18-inch pipe.

Table 6.1 – 18" Sewer Manhole Inspections

MANHOLE ID	EVIDENCE OF SURCHARGE	EVIDENCE OF WATER TOPPING MH	NUMBER OF HOLES	OBSERVED INFLOW	CONE CONDITION	BARREL/CHANNEL CONDITION
2537	No	Yes	3-5	None	Good	Good
2530	No	Yes	3-5	None	Good	Good
2529	No	Yes	3-5	Low	Low Infiltration	Good
2528	No	No	3-5	None	Good	Good
2527	Yes	No	3-5	None	Good	Good
2526	Yes	No	None	None	Good	Low Infiltration
2525	No	No	None	None	Good	Roots with low infiltration
2524	Yes	No	None	Low	Good	Good
2523	Yes	No	pick -one	None	Roots and infiltration	Roots and low infiltration
2522	Yes	No	None	None	Low Infiltration	Dirty/Debris
2521	Yes	No	None	None	Good	Silt/Debris
2520	Yes	No	3-5	None	Cracked	Good
12958	Yes	No	None	None	Good	Good
12957	Yes	No	None	None	Good	Good

In addition to repairing these manholes and replacing vented lids with solid lids, internal inspection using CCTV is recommended for this stretch of pipe. As stated before, although this pipe is made of PVC and is only 30 years old it does run through a heavily vegetated area with the chance for roots to crack the pipe. The pipe goes under Town Run 5 times between MH 2540 and the lift station which if there is a crack in the pipe at one of these crossings, it could allow a significant amount of water into the pipe and increasing the likelihood of overflows at the lift station. The County's CCTV inspection equipment will likely not be capable of performing the inspection due to water levels in the pipe and may have to contract this work.

6.3 PIPE REPLACEMENT

The pipe replacements recommended in Sections 5.4 and 5.5 represent all pipe replacements associated with capacity issues in the InfoWorks model during the 5-year, 6-hour design storm. Some of these pipes are located upstream of flow meters and these capacity issues cannot be fully verified. Each of the flow meter areas were calibrated using the same parameters and were treated as one large basin, therefore it is difficult to tell how much of the flow is coming from one area of the basin versus another. For instance, modeling results indicate the replacement of the 6" pipe on Route 133 from Legion Lane to Kennedy Ford Road. This area (replaced in 1995) has similar parameters to the area on Circus Street and Osborne Street (original pipe in 1941) because they are both tributary to Meter 2 – Circus. Because of this, the presence of and location of capacity issues upstream of flow meter locations cannot be fully verified using the model. The County's knowledge of the collection system along with additional flow monitoring may be useful in determining future pipe replacements within the collection system.

Figure 5.12 shows the locations of the pipe replacements recommended at this time. The 18" pipe running from Burke Park to Osborne Street is being recommended to convey future wet weather flows as well as to prevent flooding in Burke Park. Although some of this pipe (Plane St. to Osborne St.) was lined in 1993, the 10" PVC was installed at such a shallow slope than it cannot convey wet weather flows upstream, which causes flow to back up and flood in Burke Park. Additionally, the pipe from Plane St. and Rich St. to Water St. and Main St. will replace original (1941) 8" VCP with a 12" pipe to convey future wet weather flows and eliminate flooding at the intersection of Plane St. and Rich St. It is anticipated this pipe will also reduce the amount of I/I in the system due to the replacement of the original pipe and eliminate the possibility for water to enter the system at the five creek crossings in this stretch of pipe. The recommendation for pipe replacement and manhole replacement from East St. to Plane St is also recommended by a 2008 SSES study performed by EMH&T where CCTV confirmed the presence of high infiltration.

6.4 ADDITIONAL FIELD INVESTIGATIONS

At this time it is recommended to do a detailed SSES study in the center of the village bounded by East St, West St, Plane St and Kennedy Ford Rd. Figure 6.1 shows the proposed study area. The pipes downstream of MH 2616 (Meter 2 – Circus) and MH 2581 (Meter 3 – Bone) are recommended for pipe replacement based on current flow projections for the design storm but the need can be eliminated with some inflow removal. This area was chosen as a potential area for I/I removal because flow monitoring data suggests an inflow response and because the sewers in the area have reached their expected useful life (installed 1941). This work will not overlap with the SSES study performed by EMH&T in 2008. The study should consist of manhole inspections, smoke and dye testing, and CCTV work. It is recommended Clermont County do the majority of the CCTV work to save cost.

6.5 COST ESTIMATE AND SEQUENCING

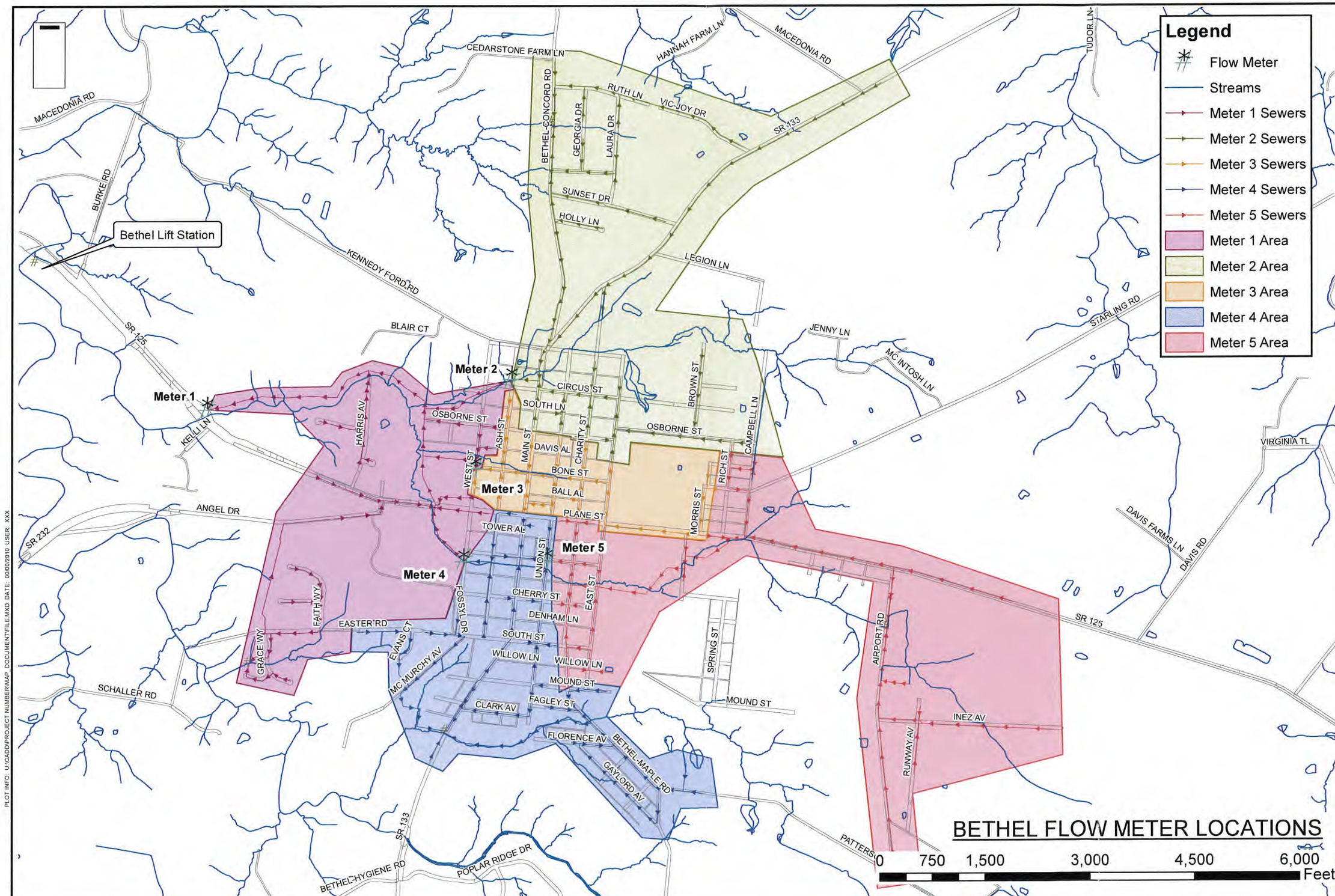
It is recommended the County perform the tasks mentioned above in the following sequence:

- Storage Tank - Clean and pump remaining water from the tank at the Bethel lift station and verify proper operation of all pumps and gates.
- 18-inch Sewer – Perform internal CCTV inspection and replace lids with solid covers. Perform manhole rehabilitation as necessary.
- Potential High Inflow Areas - Perform field investigations in areas highlighted in red on Figure 6.1
- Install new pipe from Plane St. and Rich St. to Osborne St.

The above sequencing addresses the most immediate issues first as well as those that are easiest to implement. The I/I investigation work may identify improvements that could be made that would reduce pipe replacement sizes in the system.

At this time, costs have not been associated with cleaning the Bethel lift station storage tank, field investigations including CCTV work, manhole inspections, smoke and dye testing since the County may wish to perform some or all of his work with their own staff. Cost of inflow removal could vary significantly depending on what the field investigations show. Without further information the cost associated with inflow removal cannot be accurately predicted. The construction cost for the proposed sewer replacement is approximately $1.22 million with total project costs including contingencies, design, and inspection estimated at approximately $1.65 million.

Item	Unit	Quantity	Unit Price	Cost
Mobilization/Demobilization	Lump Sum	1	$ 150,000	$ 150,000
18" Sewer	Lin Ft	2,718	$ 79	$ 214,722
15" Sewer	Lin Ft	495	$ 81	$ 40,145
12" Sewer	Lin Ft	4,057	$ 73	$ 294,133
Remove or Abandon Manhole	Each	28	$ 375	$ 10,500
Sanitary Manhole	Each	28	$ 2,120	$ 59,360
Full Depth Pavement- Remove and Replace	Sq Yd	2,168	$ 70	$ 151,760
4" Topsoil, Including Seed, Fertilize and Mulch	Sq Yd	4,294	$ 3	$ 12,883
Soil Erosion and Sedimentation Control	Lump Sum	1	$ 50,000	$ 50,000
Silt Fence	Lin Ft	4,831	$ 3	$ 12,078
Clear, Grub, Remove trees and Stumps within Public and Temporary Easement	Lump Sum	1	$ 20,000	$ 20,000
TV Inspection	Lin Ft	7,270	$ 3	$ 18,175
Maintain Sanitary Flow	Lump Sum	1	$ 75,000	$ 75,000
Preconstruction Survey	Lump Sum	1	$ 20,000	$ 20,000
Creek Crossings	Each	7	$ 2,299	$ 16,092
Connect Existing Sanitary Lateral Taps	Each	32	$ 400	$ 12,800
Connect Existing Sewer	Each	20	$ 250	$ 5,000
Dewatering	Lump Sum	1	$ 54,900	$ 54,900
Utility Crossings	Each	9	$ 1,325	$ 11,925
Total				**$ 1,229,471**



BETHEL FLOW METER LOCATIONS

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Bethel Lift Station Storage

PROJECT NO. G120381

FIGURE NO. 2.1

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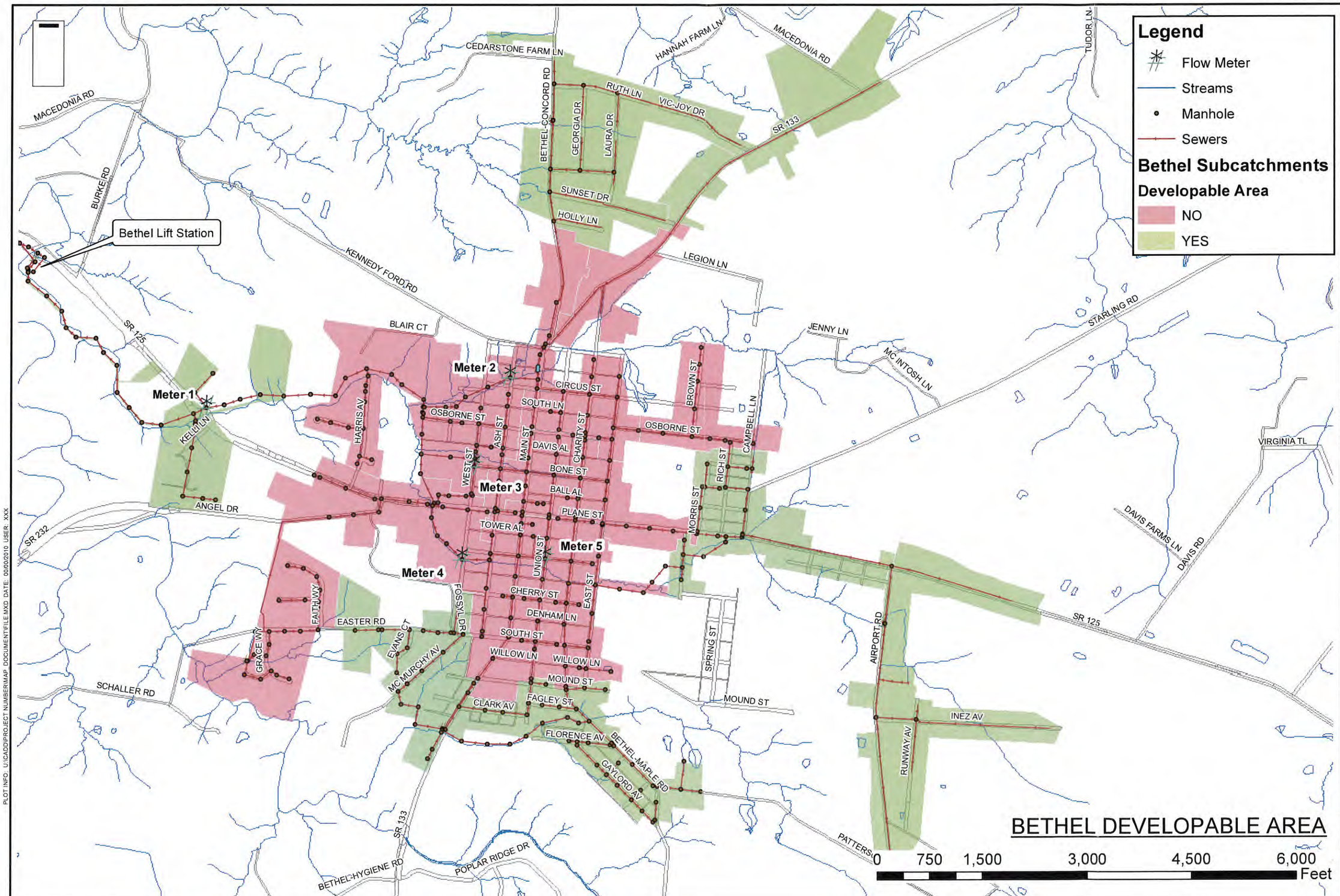

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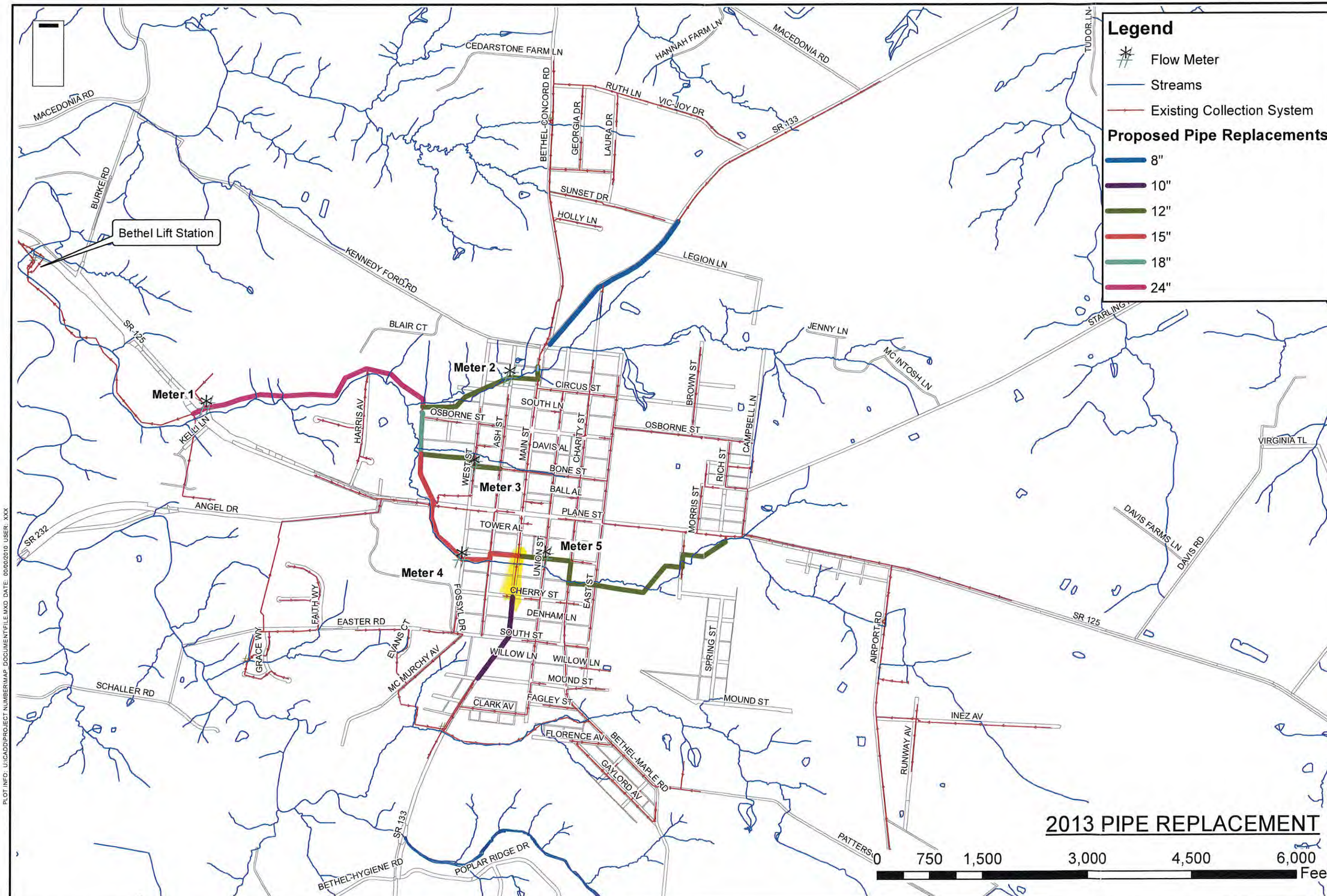

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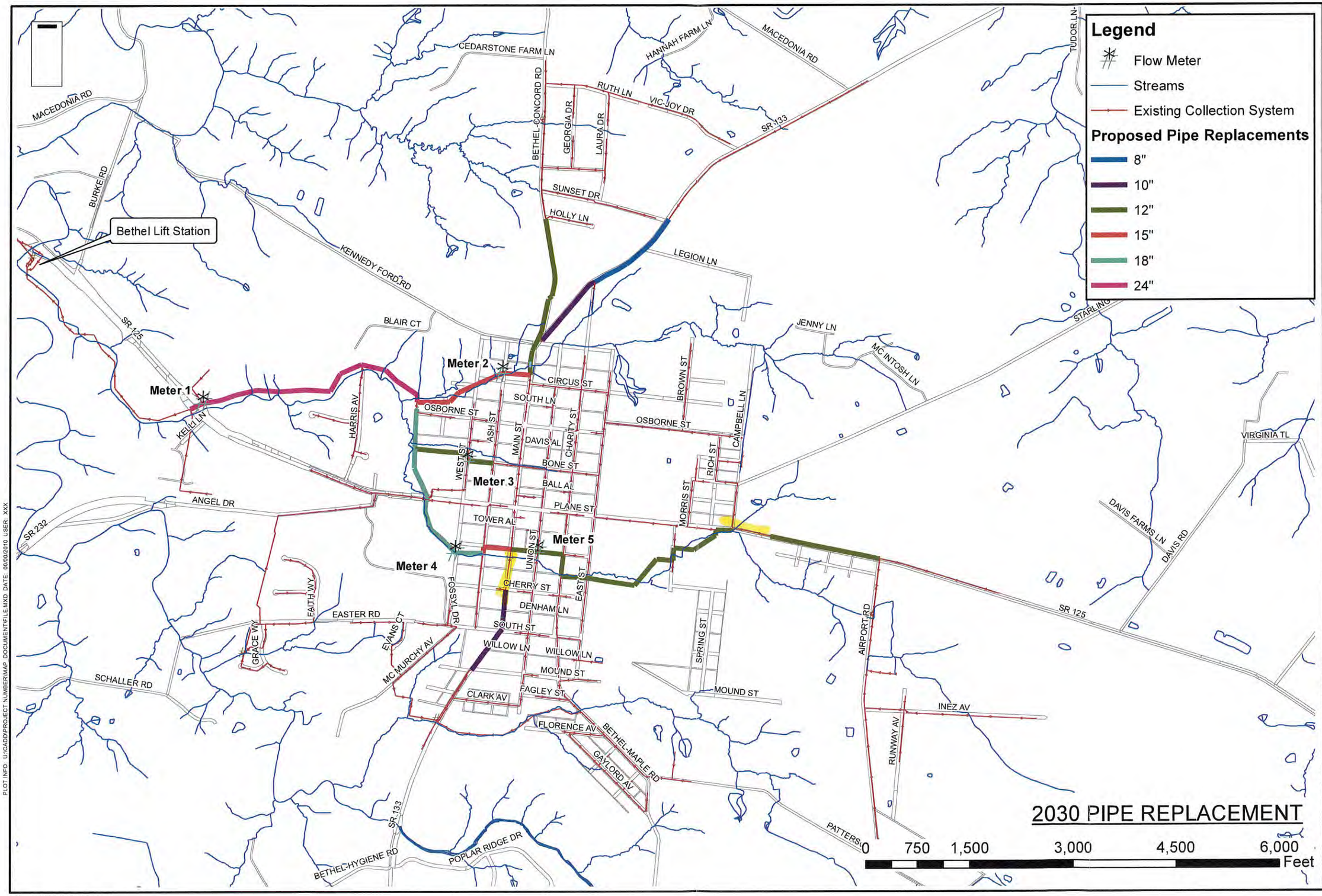

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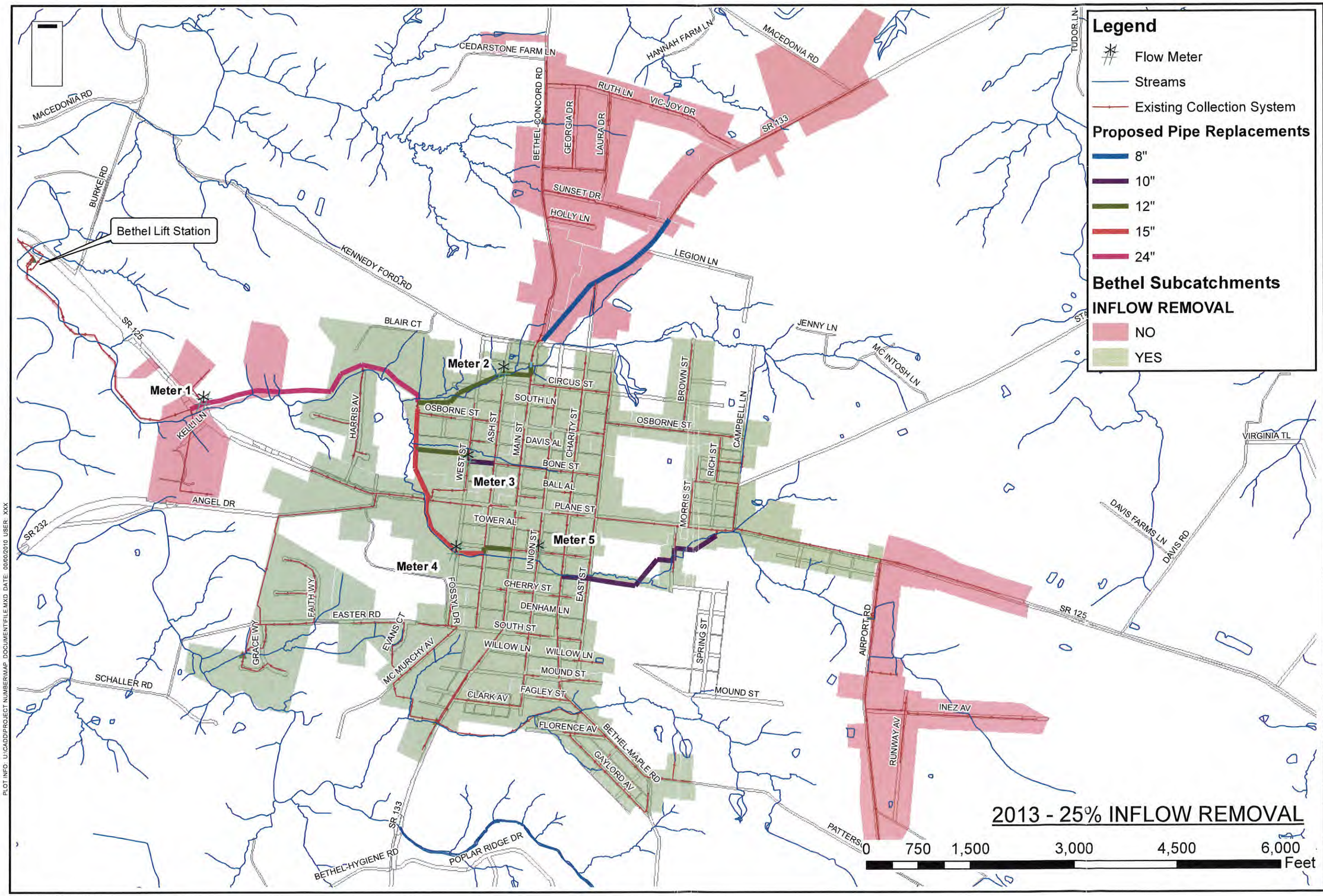

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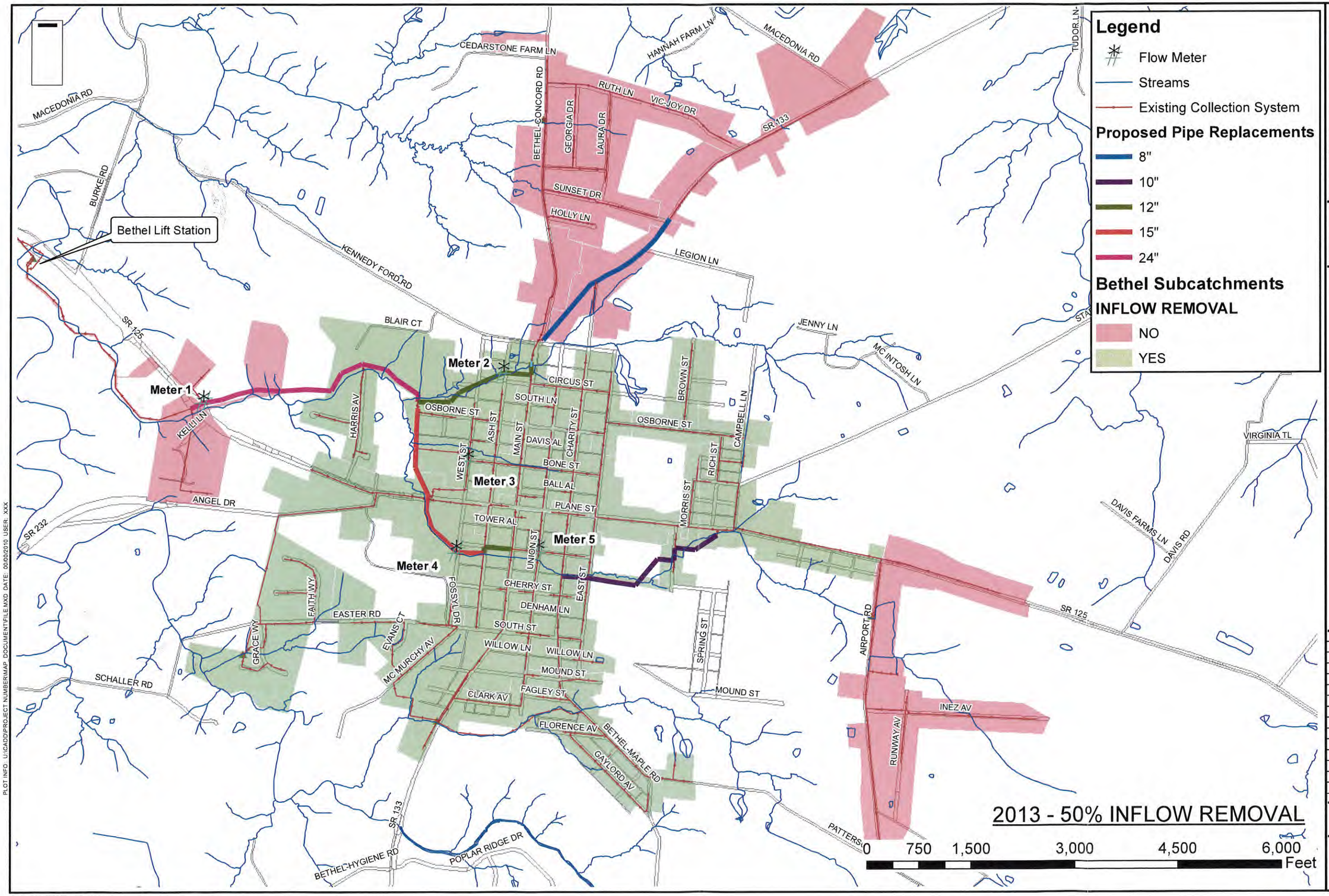

2013 - 50% INFLOW REMOVAL

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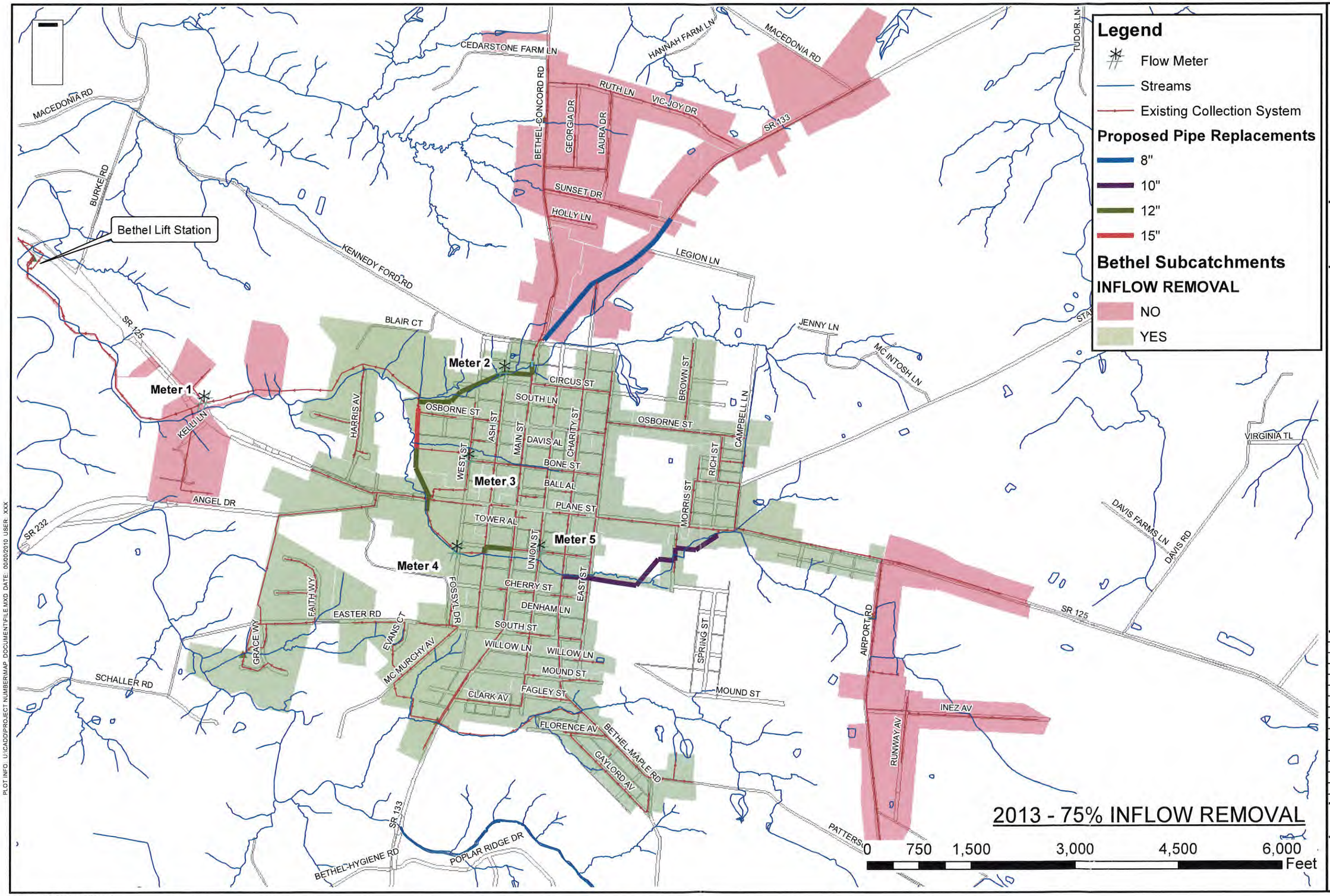

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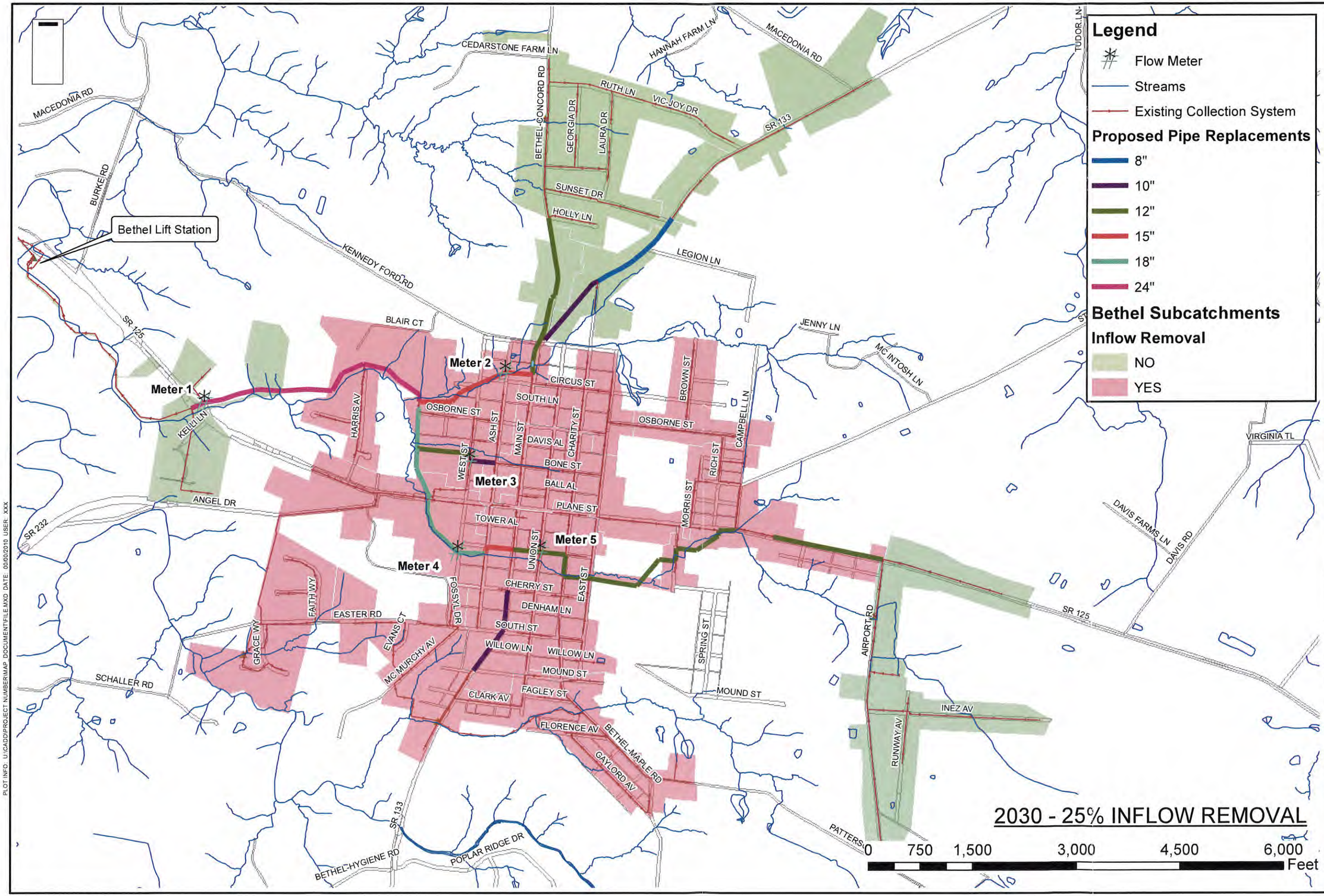

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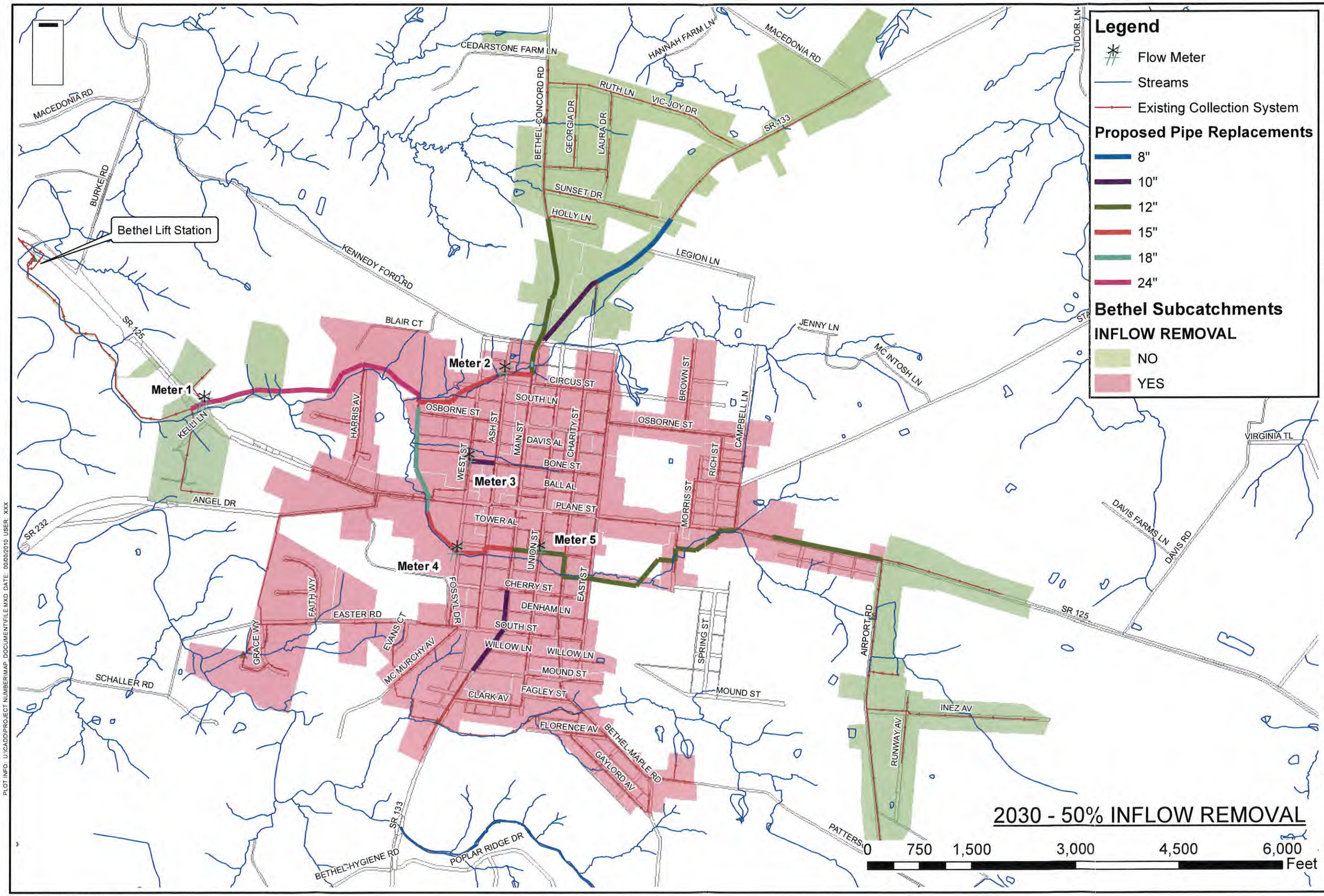

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FIGURE NO.
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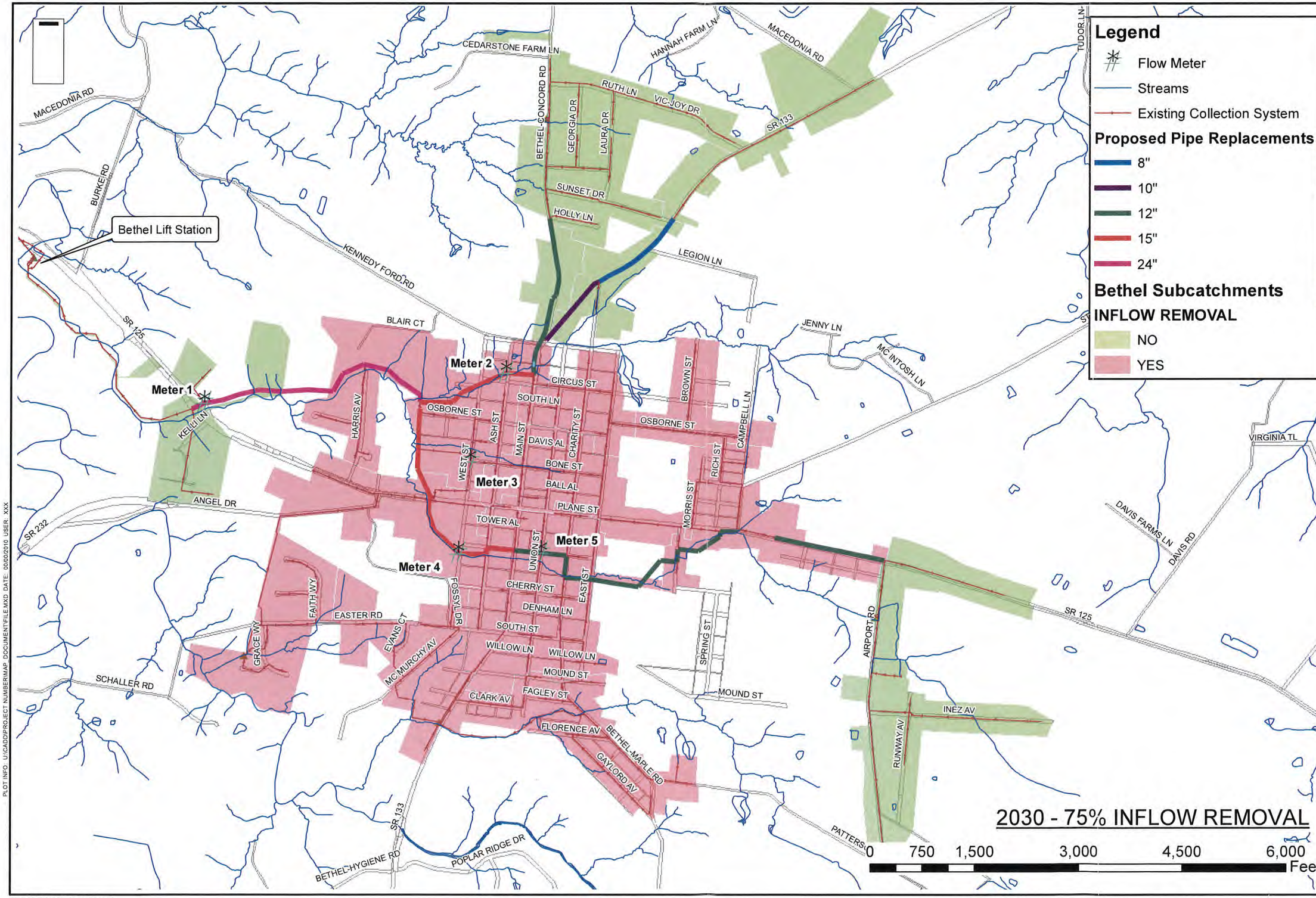

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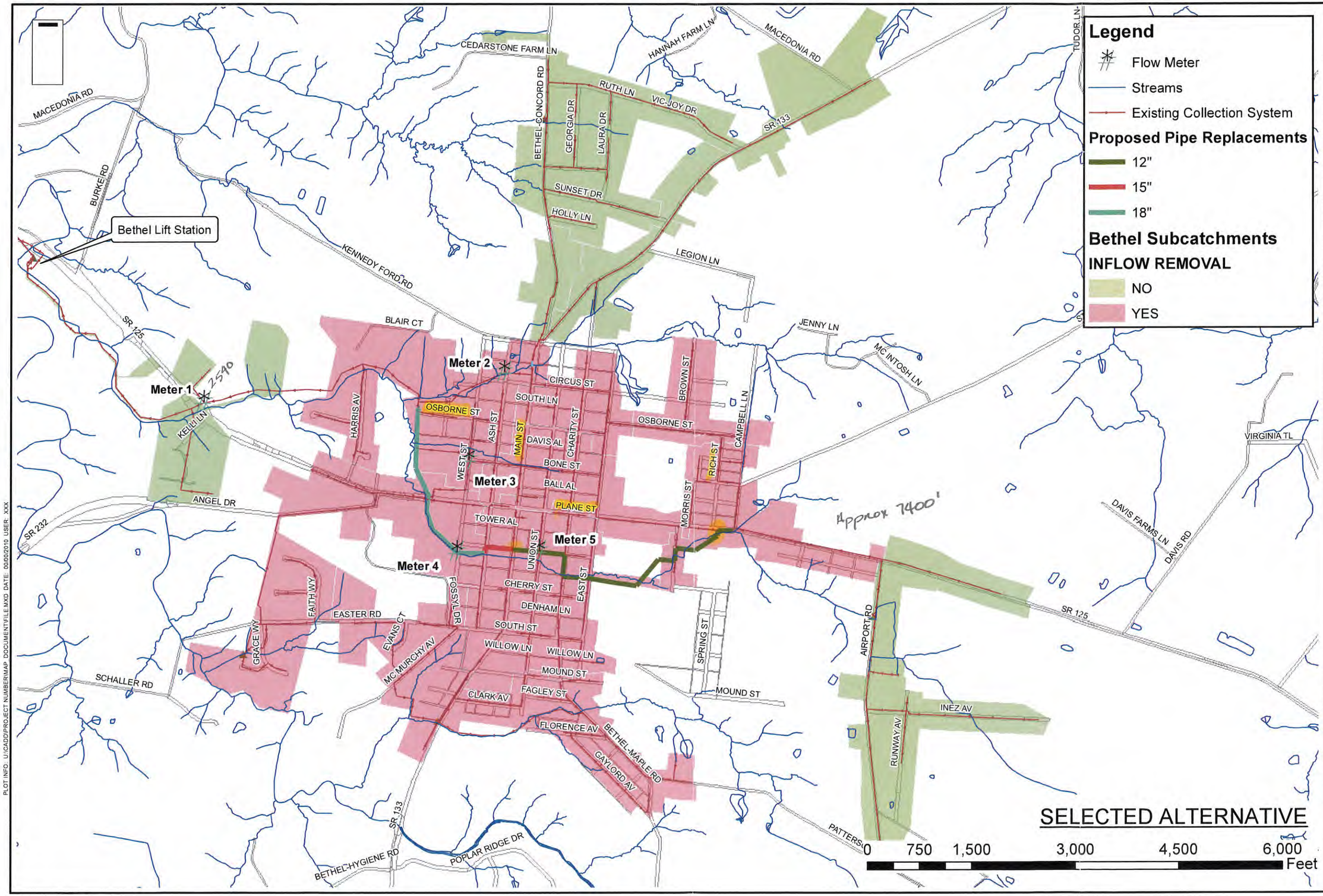

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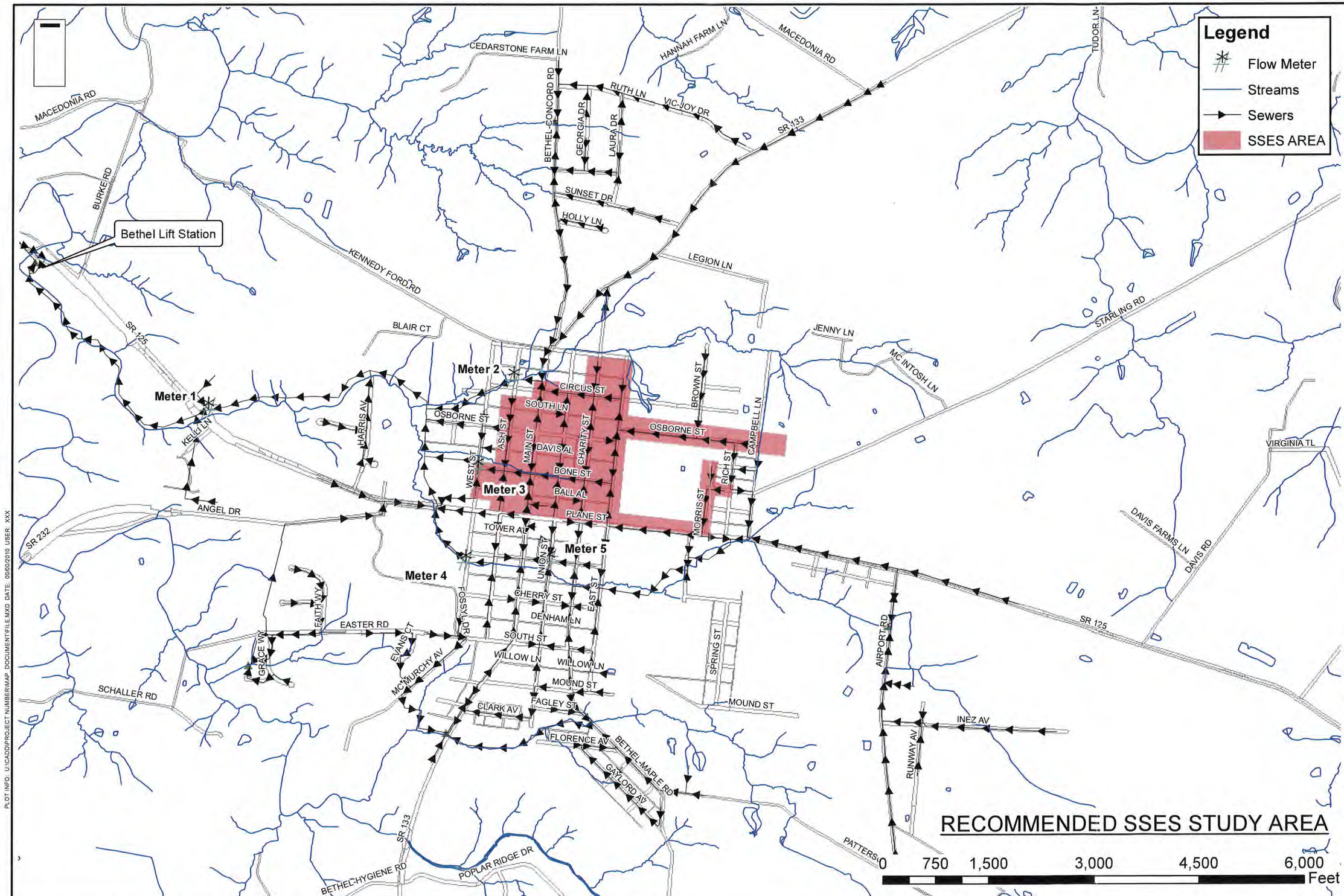

RECOMMENDED SSES STUDY AREA

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Bethel Lift Station Storage

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FIGURE NO. 6.1

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Appendix A

Bethel Calibration Results

In-System Flow Meter Calibration Results

Meter Location Summary

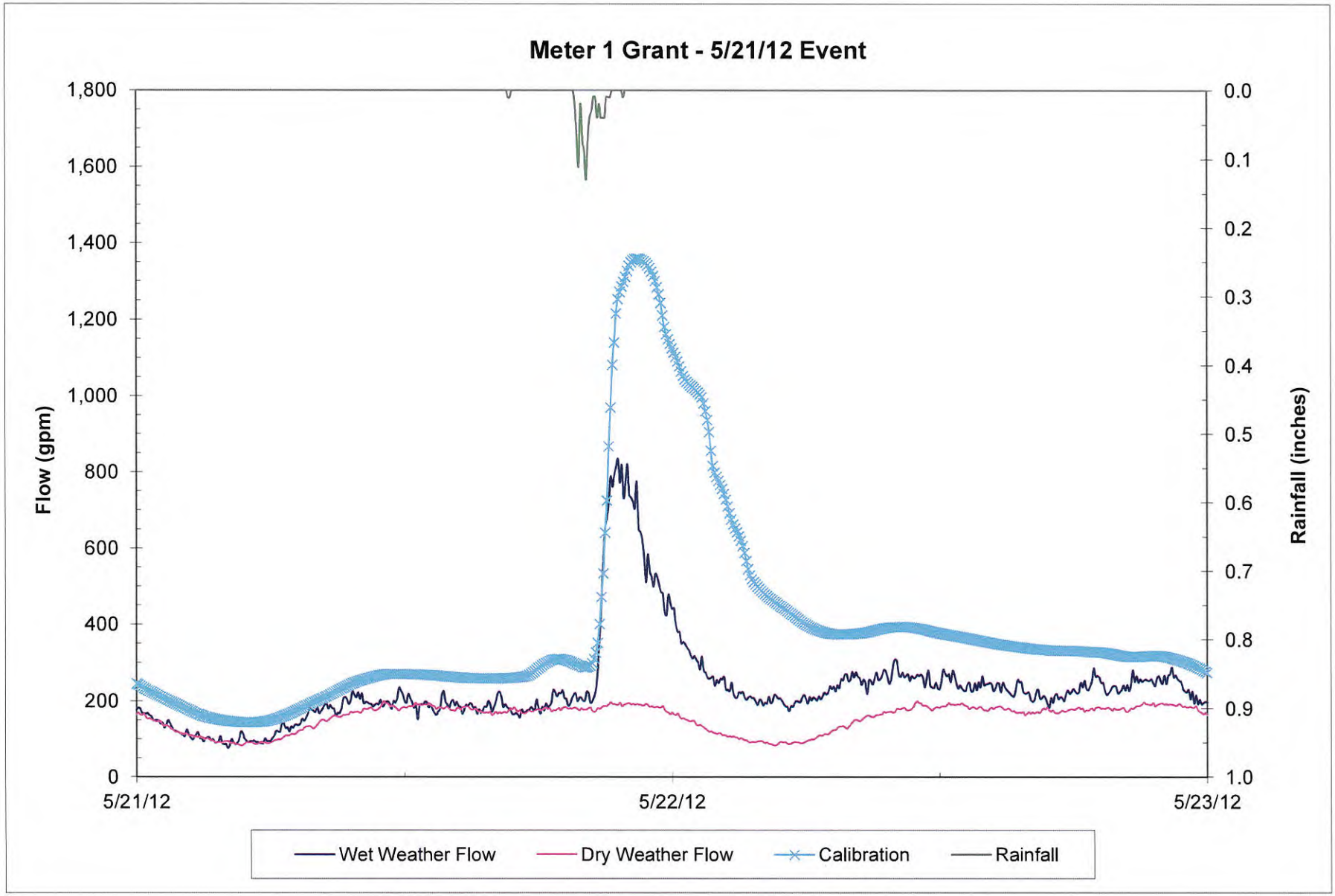
Site ID	Site 1
Location Summary	Grant Career Center
Manhole Structure ID	2541
Model Link ID	2541.1
Pipe Dimension (in)	18
Pipe Shape	Circular

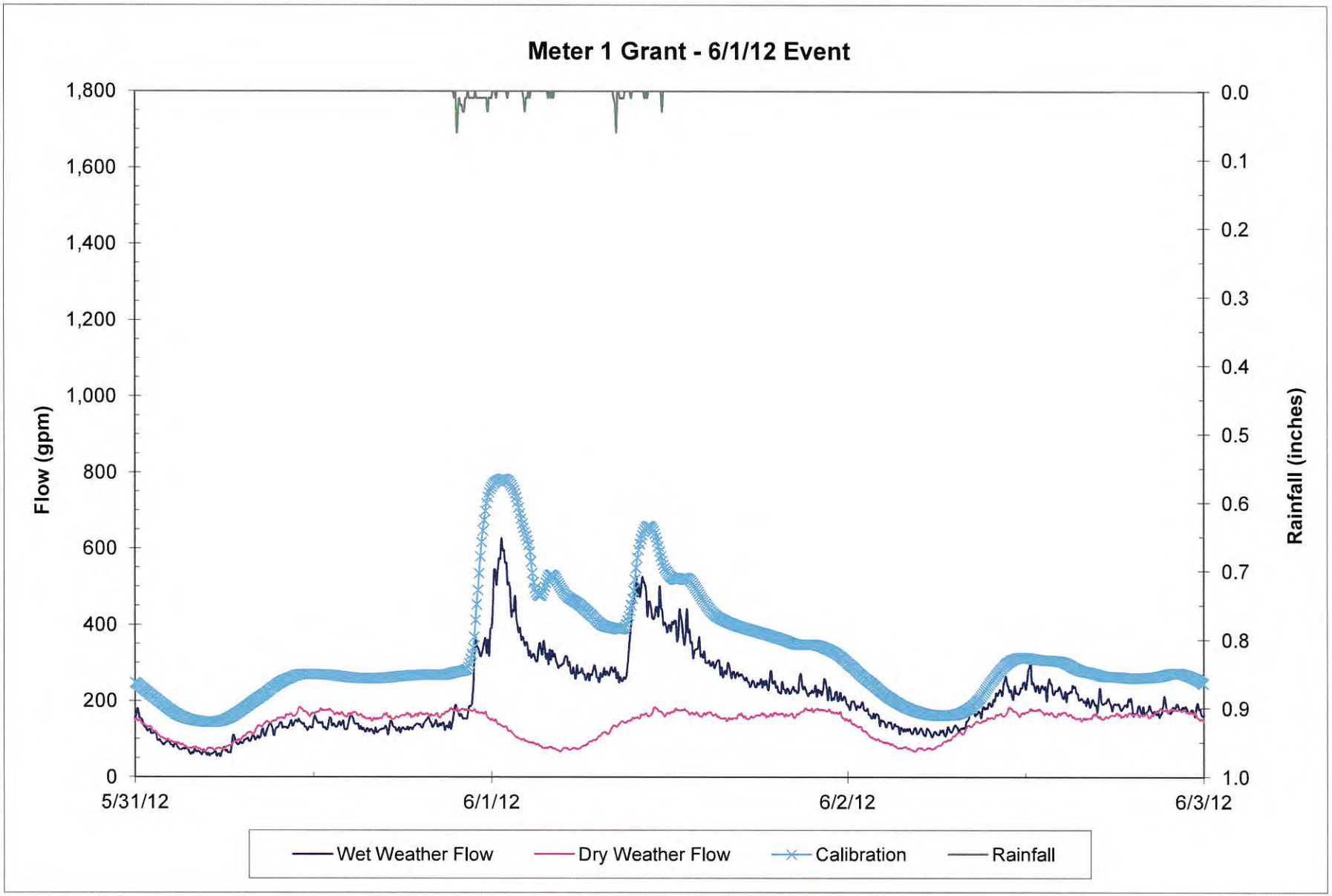
Tabular Summary of Meter-to-Model Volume, Peak Flow, and Depth Comparisons for Calibration

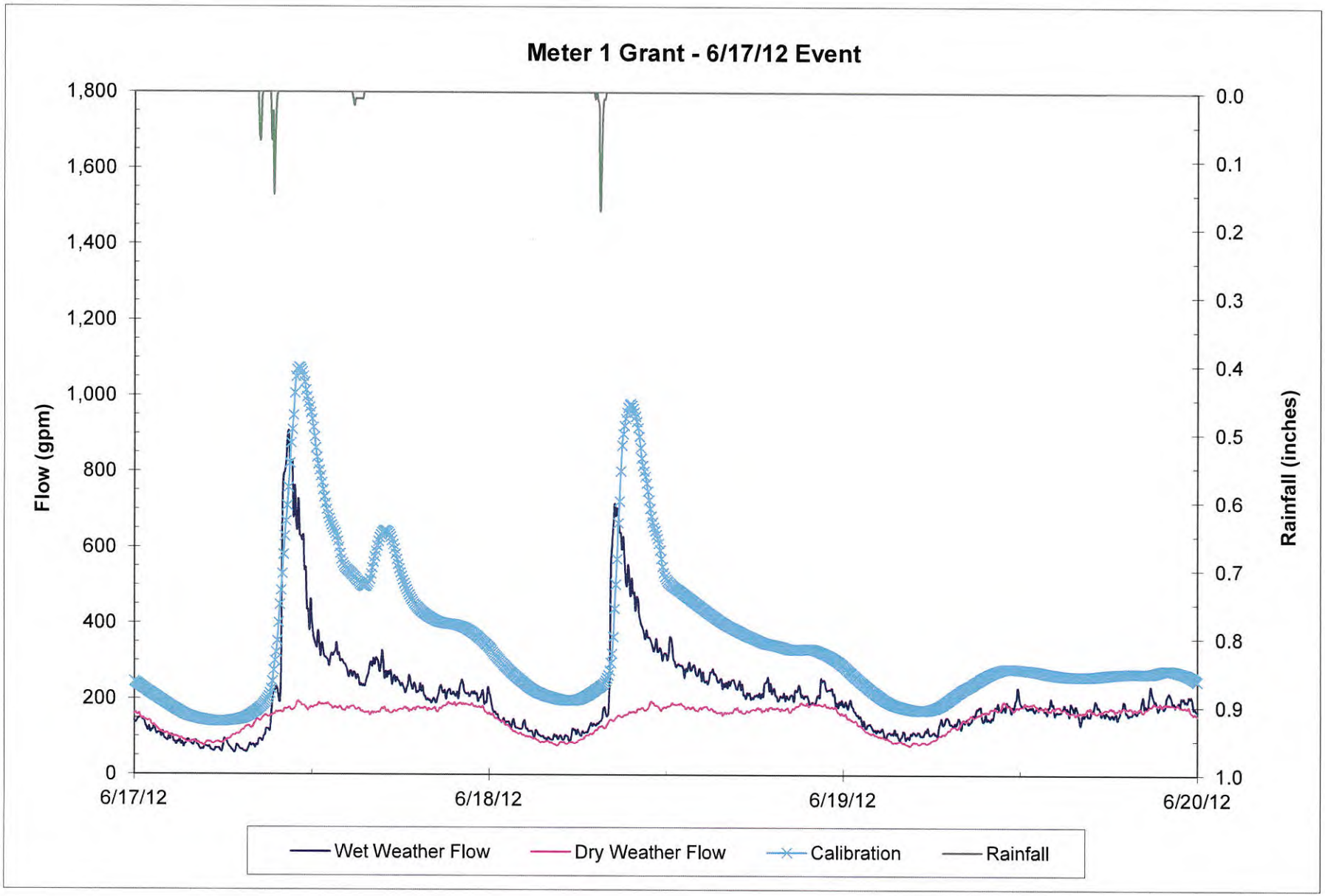
Calibration Periods	Data Comments	Volume (MG)			Peak Flow (gpm)			Peak Depth (ft)		
		Meter	Model	Percent Difference (%)	Meter	Model	Percent Difference (%)	Meter	Model	Difference (ft)
April 11, 2013		1.93	2.10	9%	982	1113	13%	0.68	0.76	0.08
April 19, 2013		1.76	1.63	-7%	1078	976	-9%	0.71	0.72	0.01
April 24, 2013		1.78	1.70	-4%	1062	1064	0%	0.71	0.75	0.04
June 10, 2013		1.53	1.81	18%	938	872	-7%	0.66	0.66	0.01
June 26, 2013		1.39	1.99	43%	968	1211	25%	0.89	0.81	-0.08
Total No. of Available Periods		5			5			5		
Total No. of Periods Meeting Wet-Weather Calibration Criteria		4			4			5		

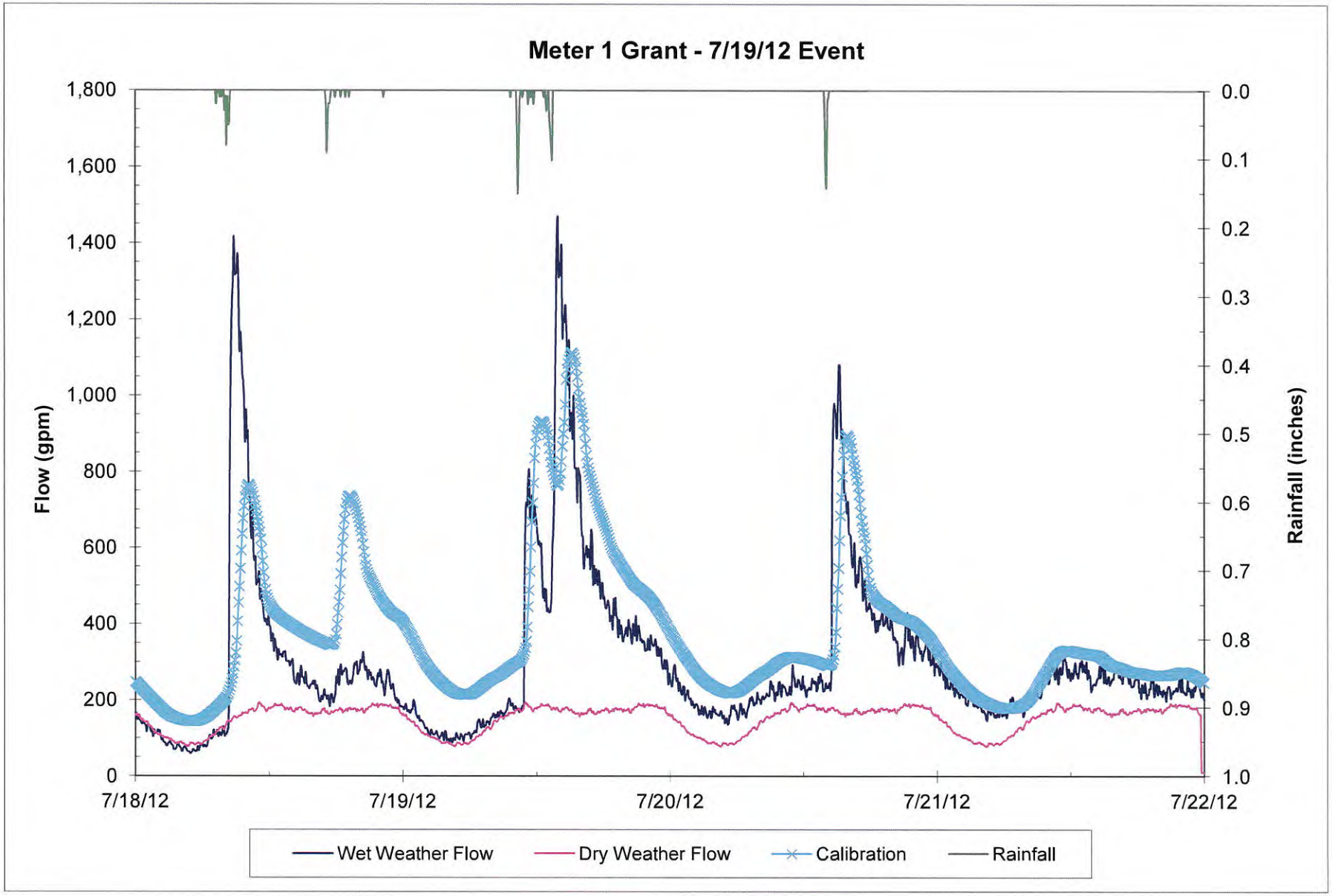
Wet-Weather Calibration Criteria

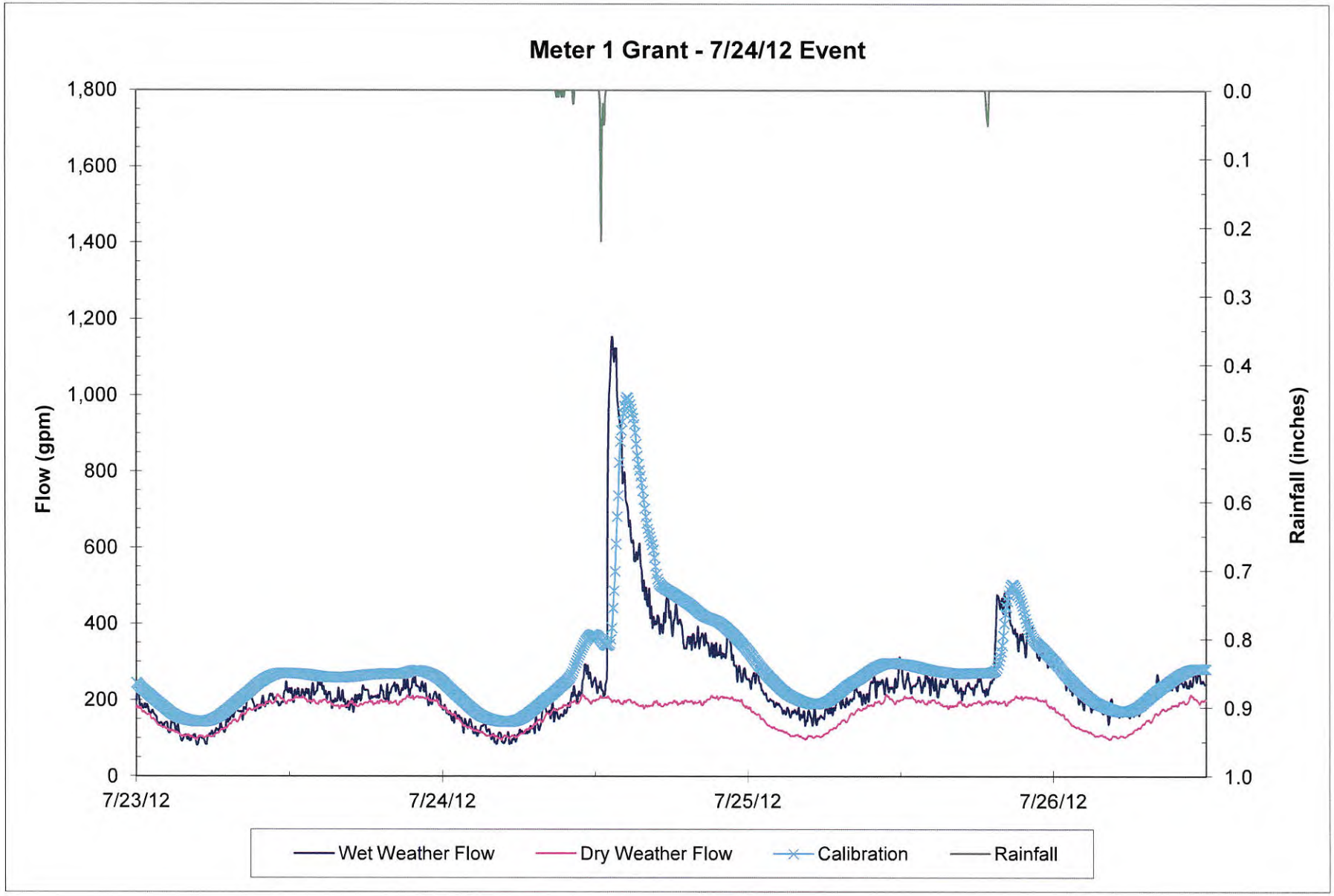
Criteria	Calibration Standard
Peak Flow Rate	-15% to +25% of Observed
Wet Weather Volume	-10% to +20% of Observed
Peak Depth	±0.33 ft at non-surcharged locations
	-0.33 feet to +1.67 feet at surcharged locations

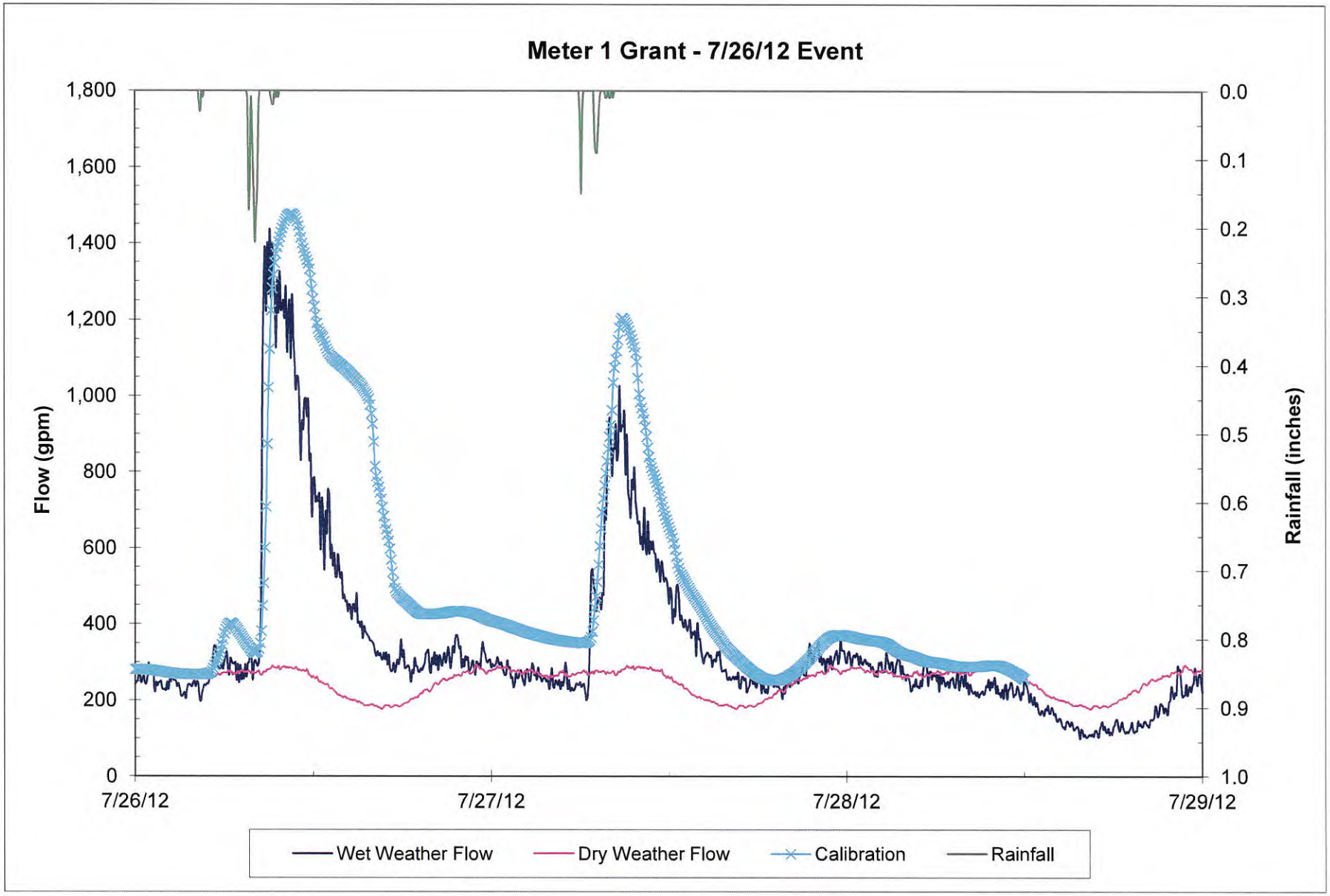

Meter 1 Grant - 5/21/12 Event
Flow (gpm)
1,800
1,600
1,400
1,200
1,000
800
600
400
200
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
5/21/12
5/22/12
5/23/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall

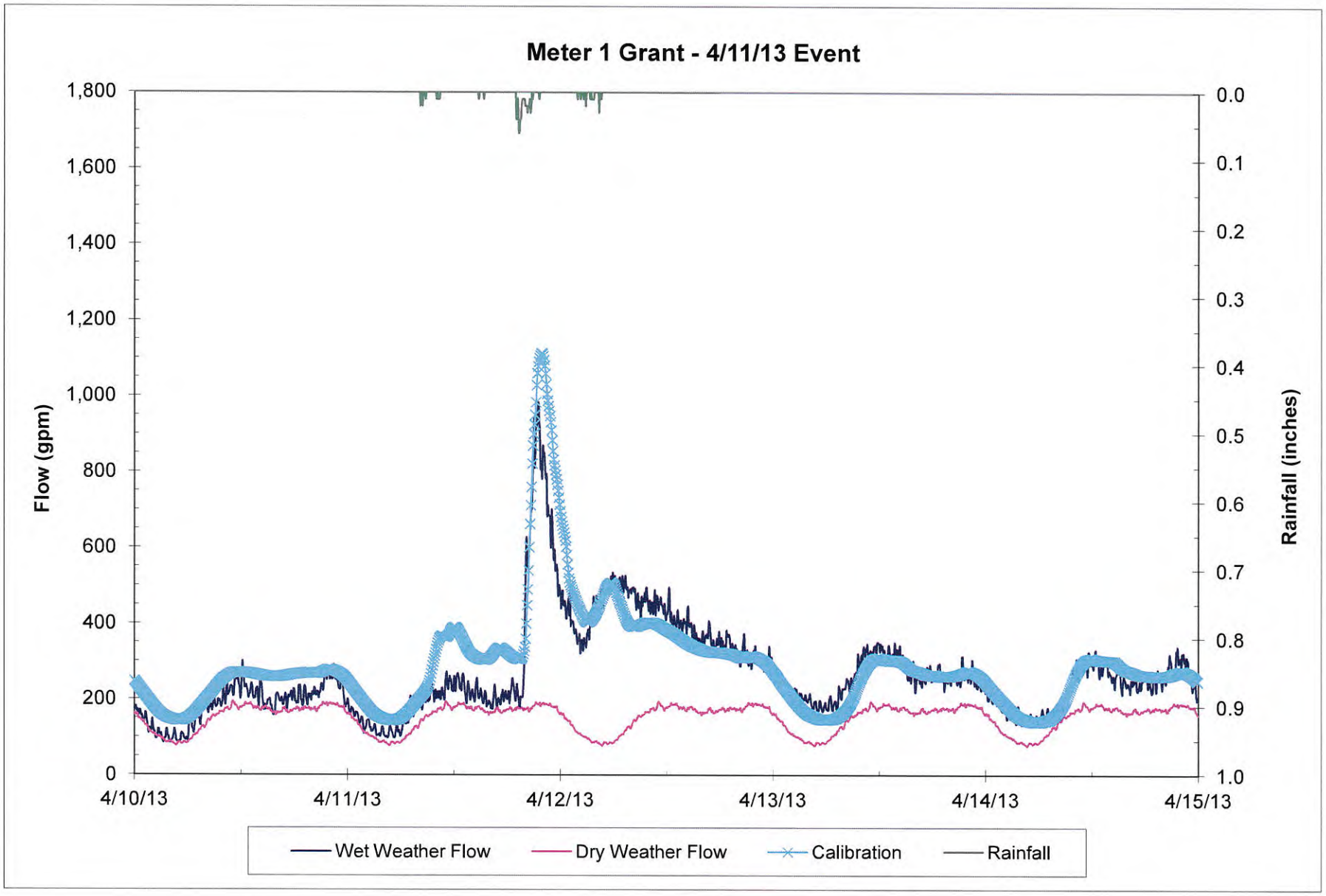

Meter 1 Grant - 6/1/12 Event
Flow (gpm)
1,800
1,600
1,400
1,200
1,000
800
600
400
200
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
5/31/12
6/1/12
6/2/12
6/3/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall

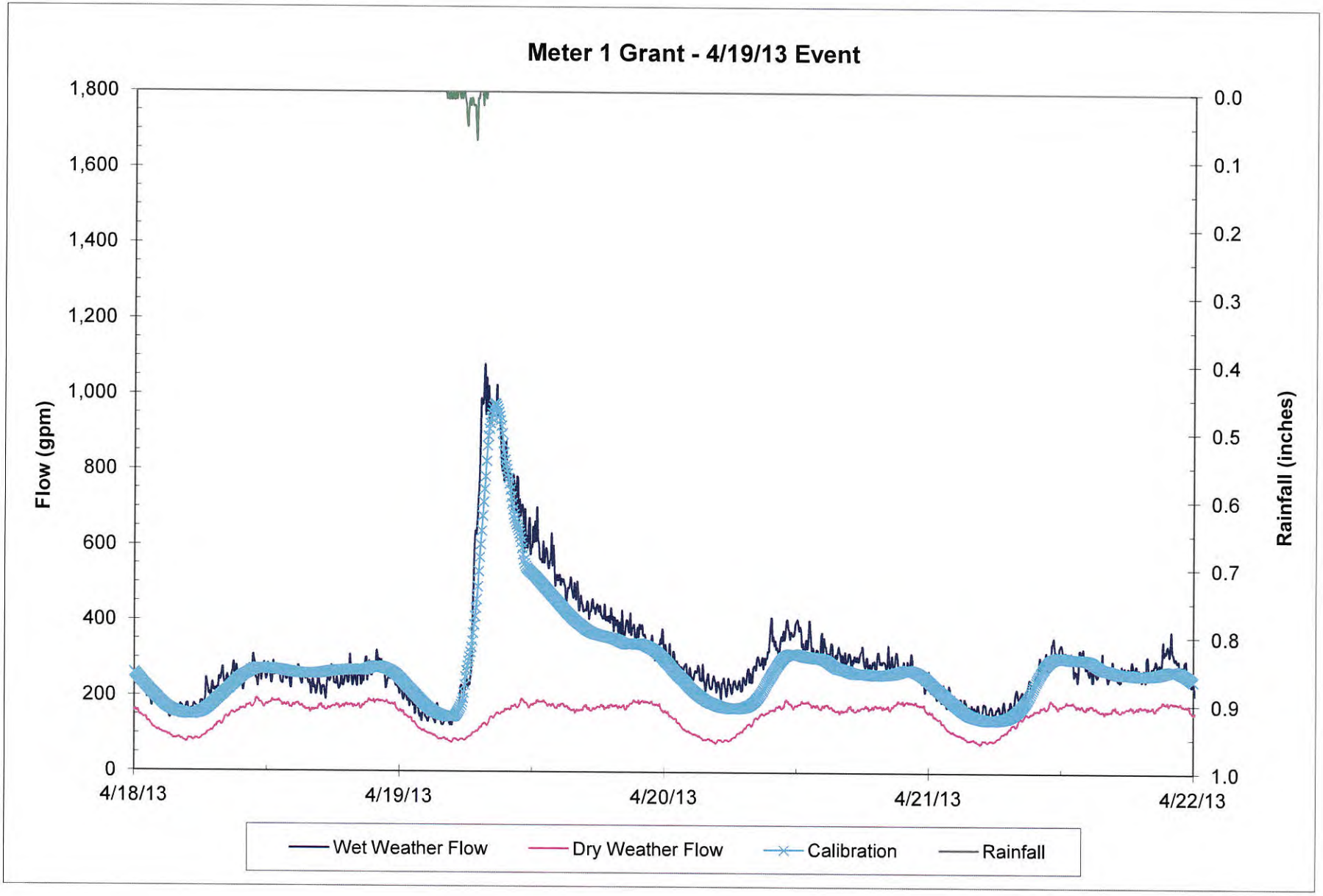

Meter 1 Grant - 6/17/12 Event
Flow (gpm)
1,800
1,600
1,400
1,200
1,000
800
600
400
200
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
6/17/12
6/18/12
6/19/12
6/20/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall

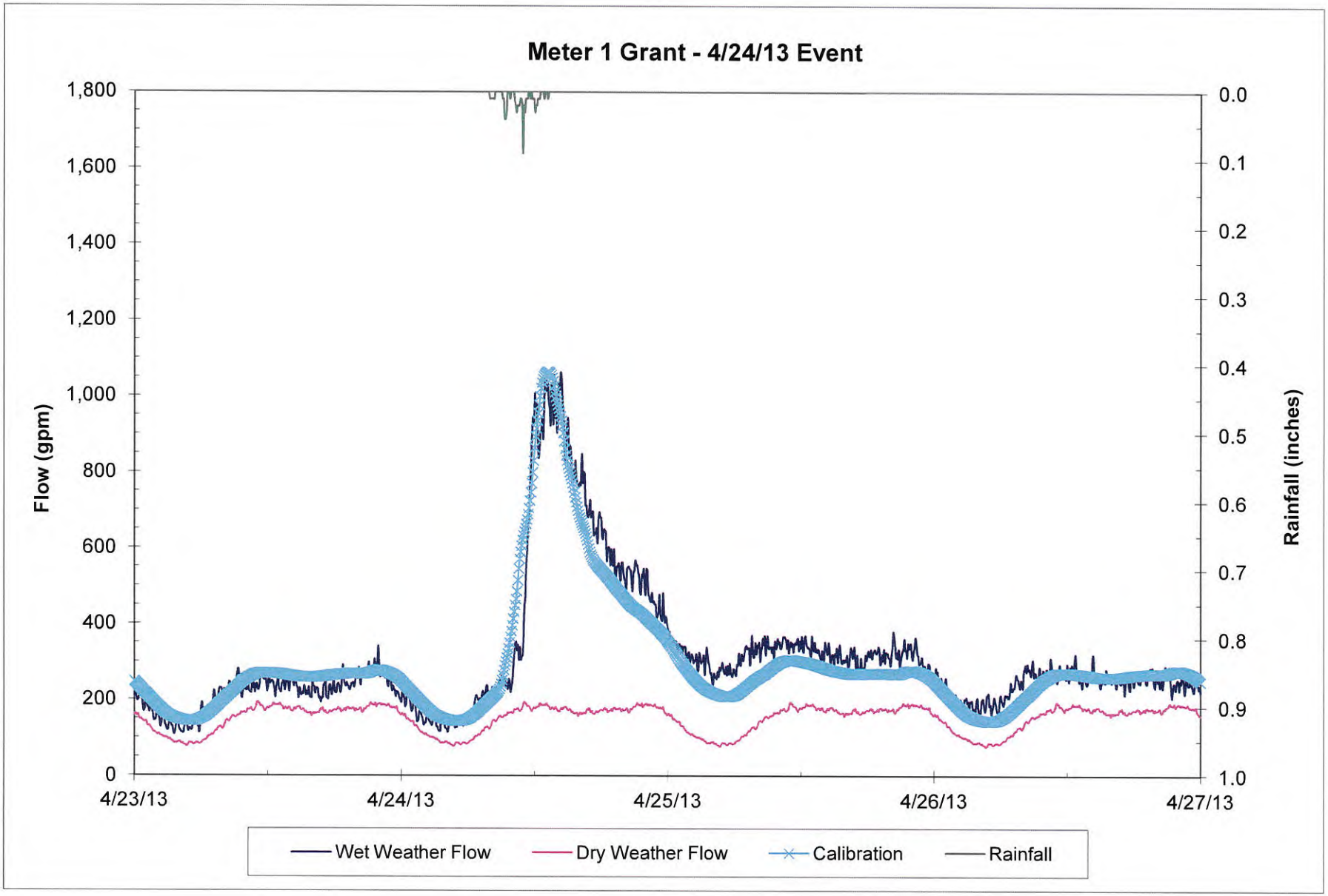

Meter 1 Grant - 7/19/12 Event
Flow (gpm)
1,800
1,600
1,400
1,200
1,000
800
600
400
200
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
7/18/12
7/19/12
7/20/12
7/21/12
7/22/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall

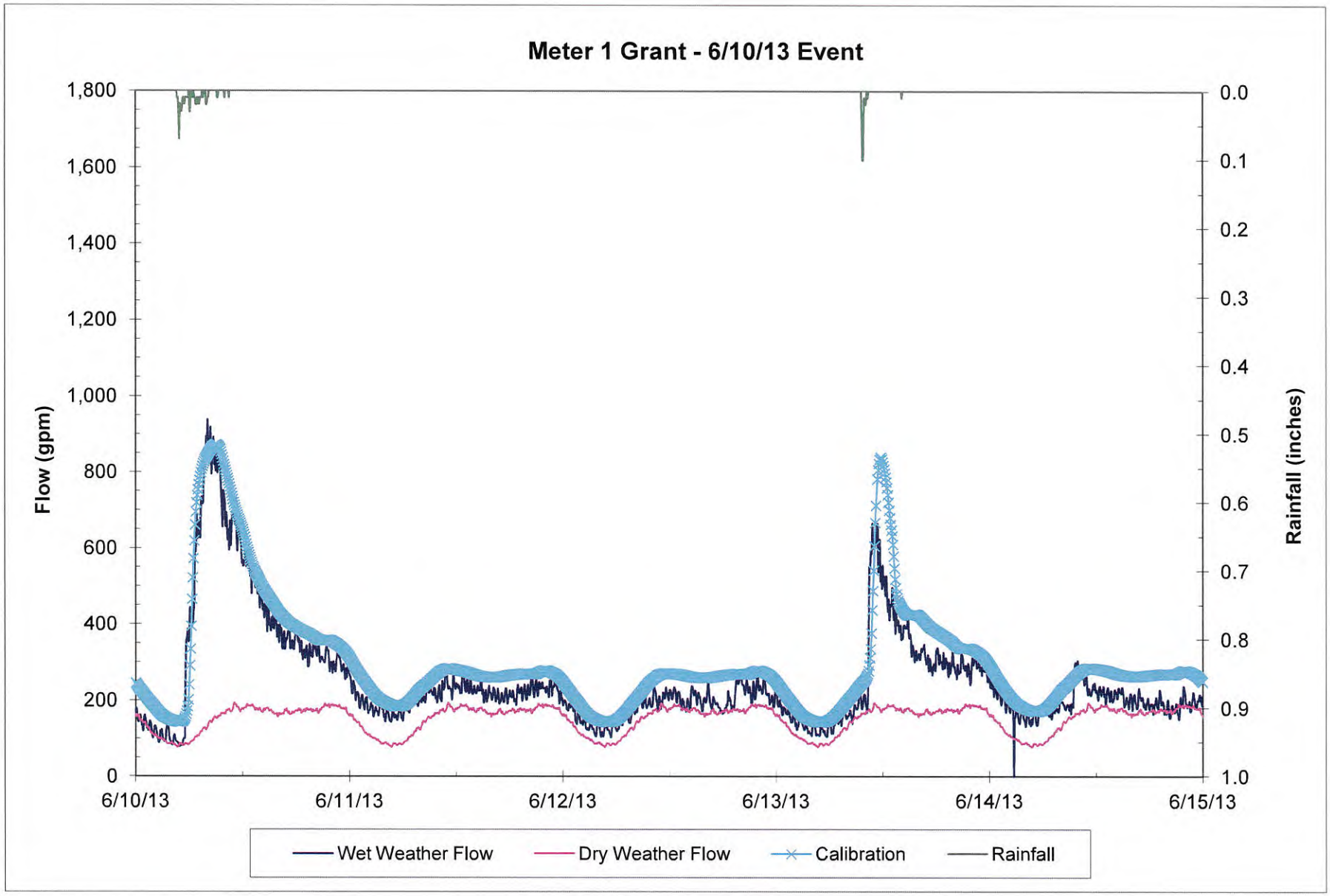

Meter 1 Grant - 7/24/12 Event
Flow (gpm)
1,800
1,600
1,400
1,200
1,000
800
600
400
200
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
7/23/12
7/24/12
7/25/12
7/26/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall

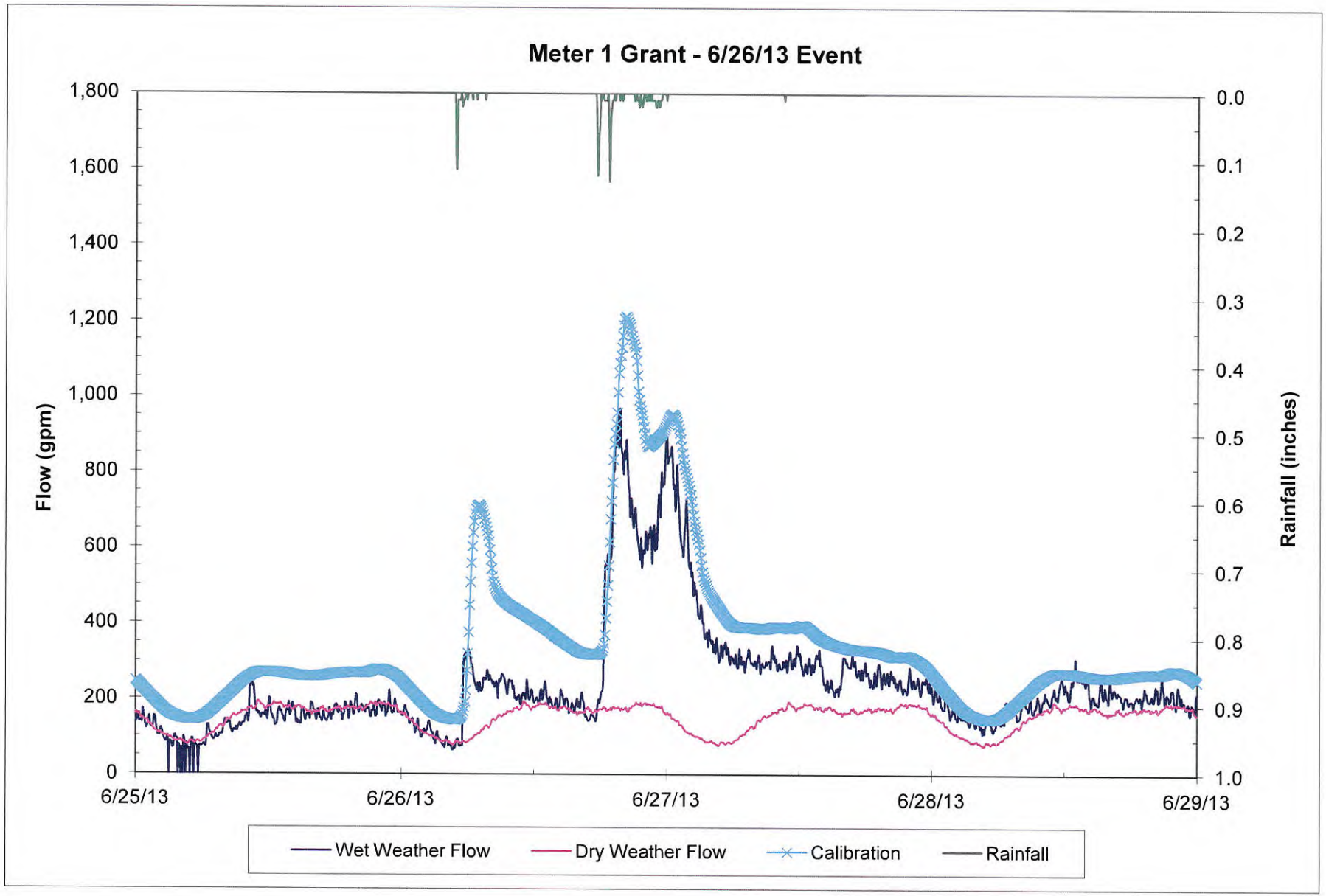

Meter 1 Grant - 7/26/12 Event
Flow (gpm)
1,800
1,600
1,400
1,200
1,000
800
600
400
200
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
7/26/12
7/27/12
7/28/12
7/29/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 1 Grant - 4/11/13 Event
Flow (gpm)
Rainfall (inches)
1,800
1,600
1,400
1,200
1,000
800
600
400
200
0
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
4/10/13
4/11/13
4/12/13
4/13/13
4/14/13
4/15/13
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 1 Grant - 4/19/13 Event
Flow (gpm)
1,800
1,600
1,400
1,200
1,000
800
600
400
200
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
4/18/13
4/19/13
4/20/13
4/21/13
4/22/13
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 1 Grant - 4/24/13 Event
Flow (gpm)
1,800
1,600
1,400
1,200
1,000
800
600
400
200
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
4/23/13
4/24/13
4/25/13
4/26/13
4/27/13
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall

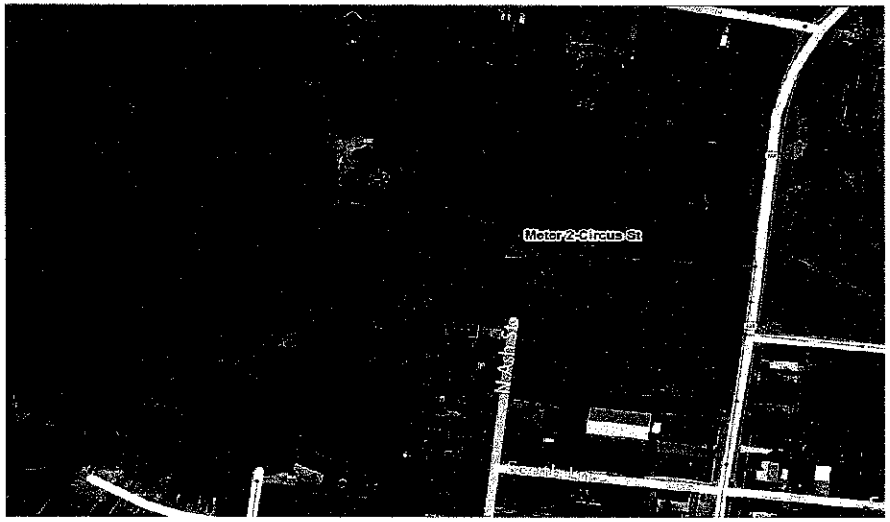

Meter 1 Grant - 6/10/13 Event
Flow (gpm)
Rainfall (inches)
1,800
1,600
1,400
1,200
1,000
800
600
400
200
0
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
6/10/13
6/11/13
6/12/13
6/13/13
6/14/13
6/15/13
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 1 Grant - 6/26/13 Event
Flow (gpm)
1,800
1,600
1,400
1,200
1,000
800
600
400
200
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
6/25/13
6/26/13
6/27/13
6/28/13
6/29/13
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall

In-System Flow Meter Calibration Results

Meter Location Summary

Site ID	Site 2
Location Summary	Circus St - In woods from Access off Main St.
Manhole Structure ID	2574
Model Link ID	2574.1
Pipe Dimension (in)	10
Pipe Shape	Circular

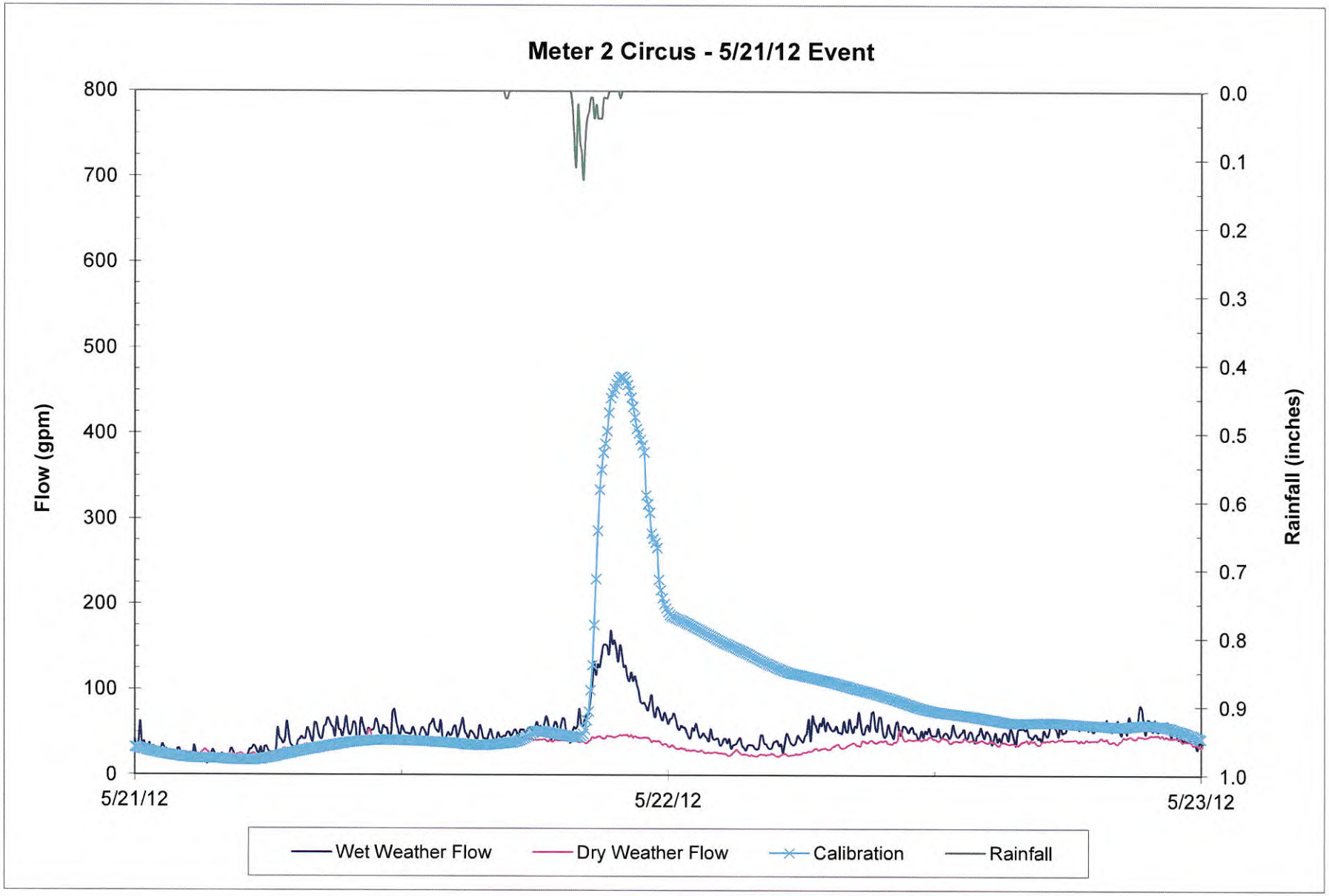


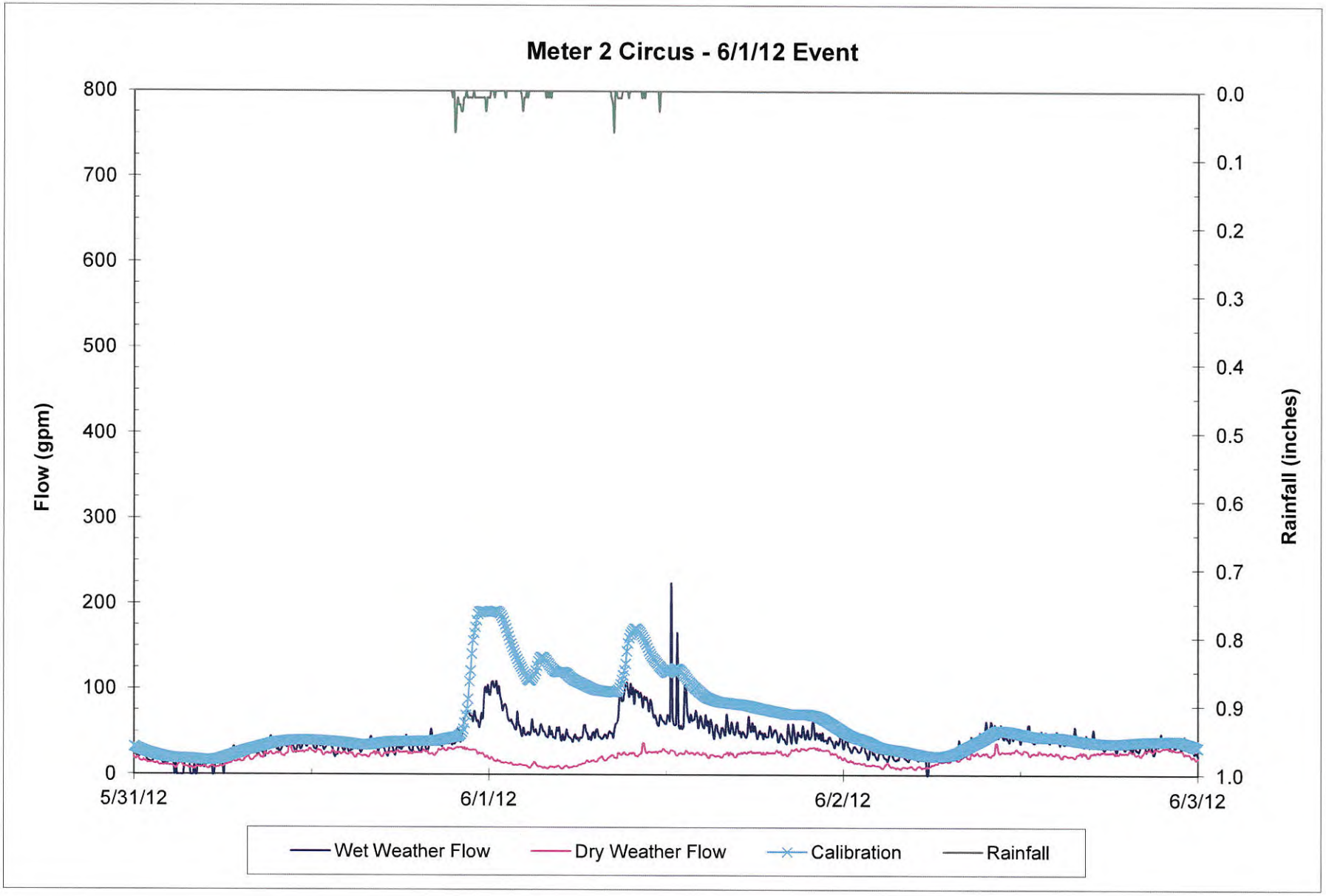
Tabular Summary of Meter-to-Model Volume, Peak Flow, and Depth Comparisons for Calibration

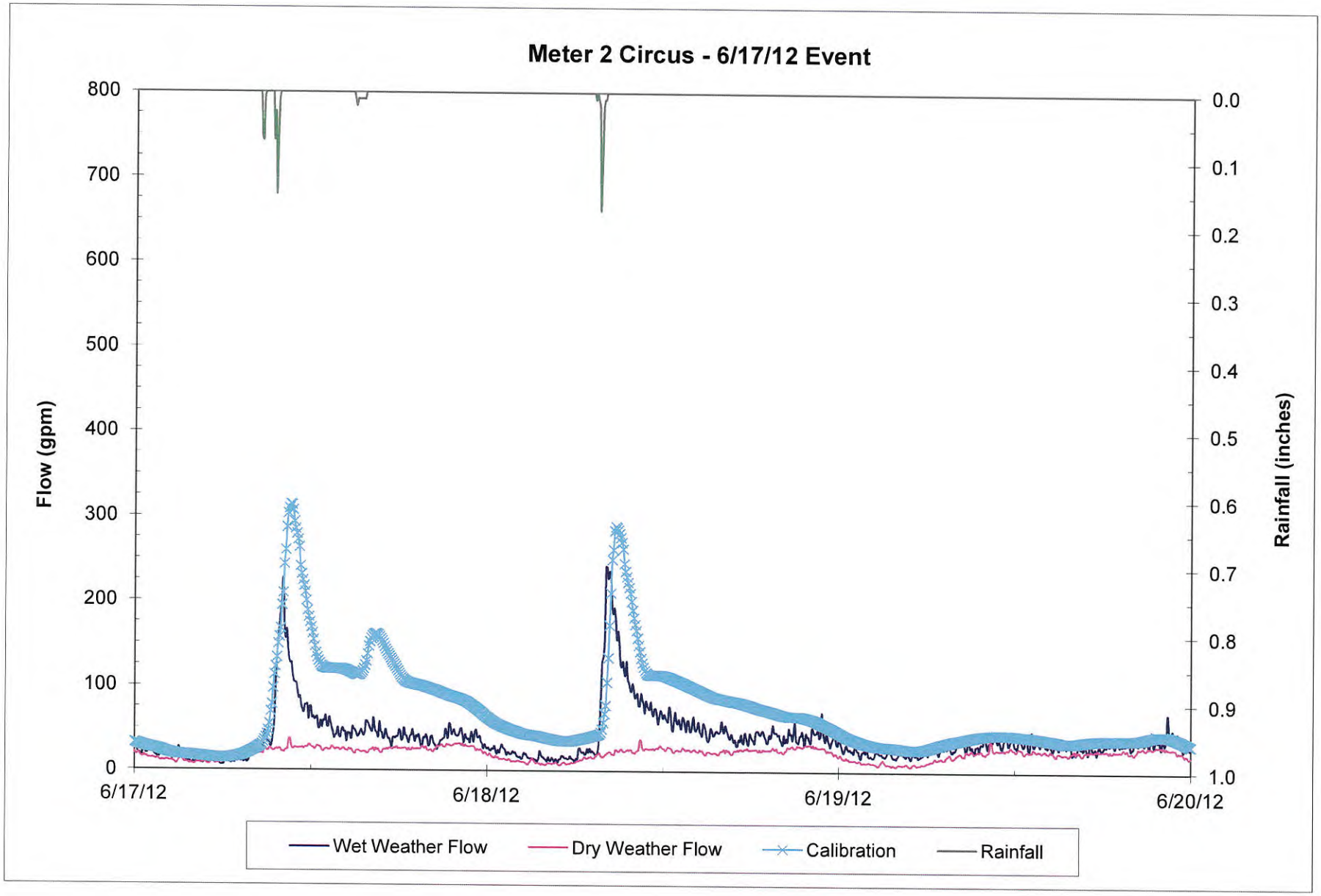
Calibration Periods	Data Comments	Volume (MG)			Peak Flow (gpm)			Peak Depth (ft)		
		Meter	Model	Percent Difference (%)	Meter	Model	Percent Difference (%)	Meter	Model	Difference (ft)
April 11, 2013		0.41	0.37	-9%	272	312	15%	0.53	0.54	0.01
April 19, 2013		0.39	0.28	-28%	350	274	-22%	0.59	0.50	-0.09
April 24, 2013		0.42	0.31	-26%	307	281	-9%	0.57	0.51	-0.06
June 10, 2013		0.34	0.34	2%	232	228	-2%	0.49	0.45	-0.05
June 26, 2013		0.33	0.40	22%	275	354	29%	0.55	0.60	0.05
Total No. of Available Periods		5			5			5		
Total No. of Periods Meeting Wet-Weather Calibration Criteria		2			3			5		

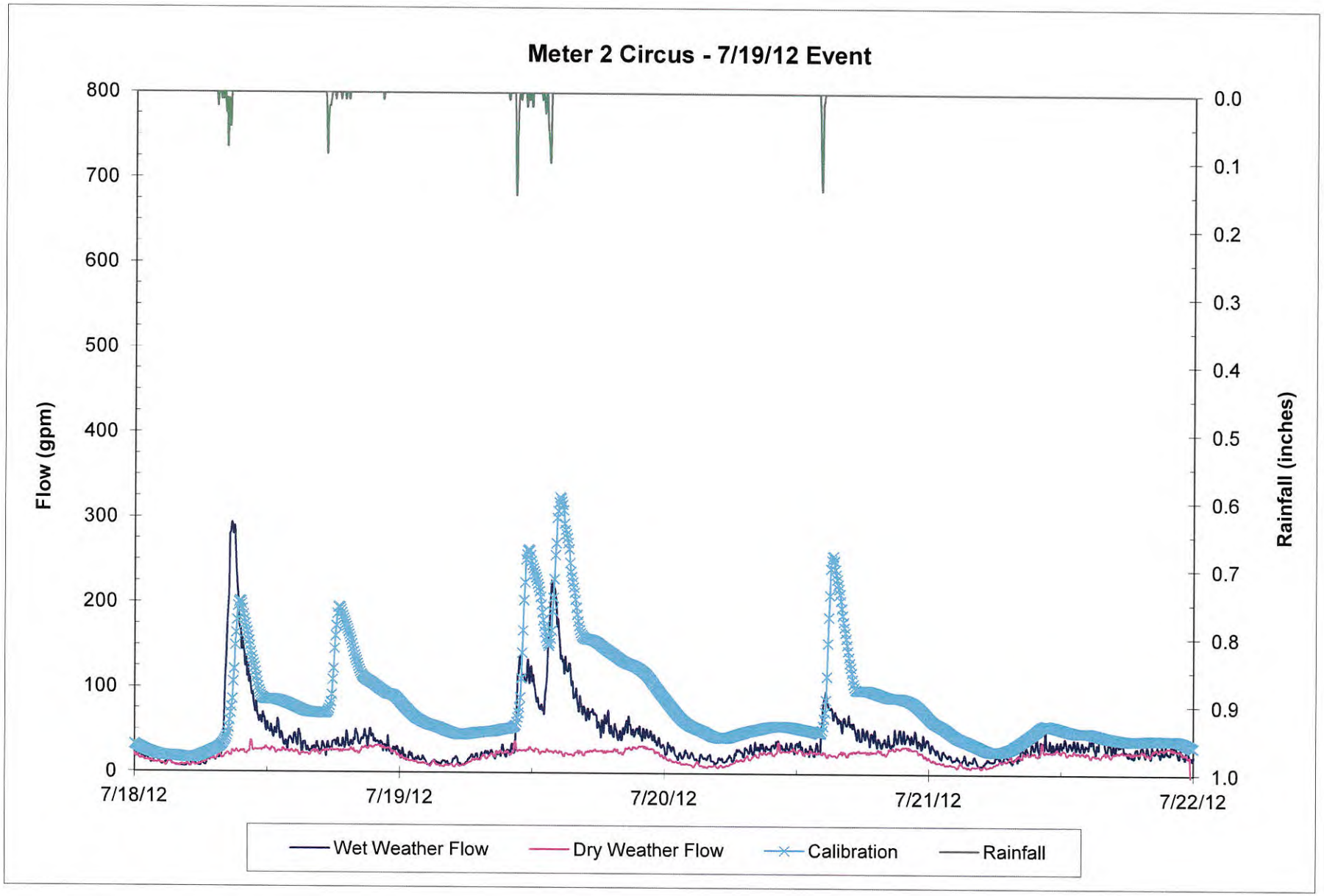
Wet-Weather Calibration Criteria

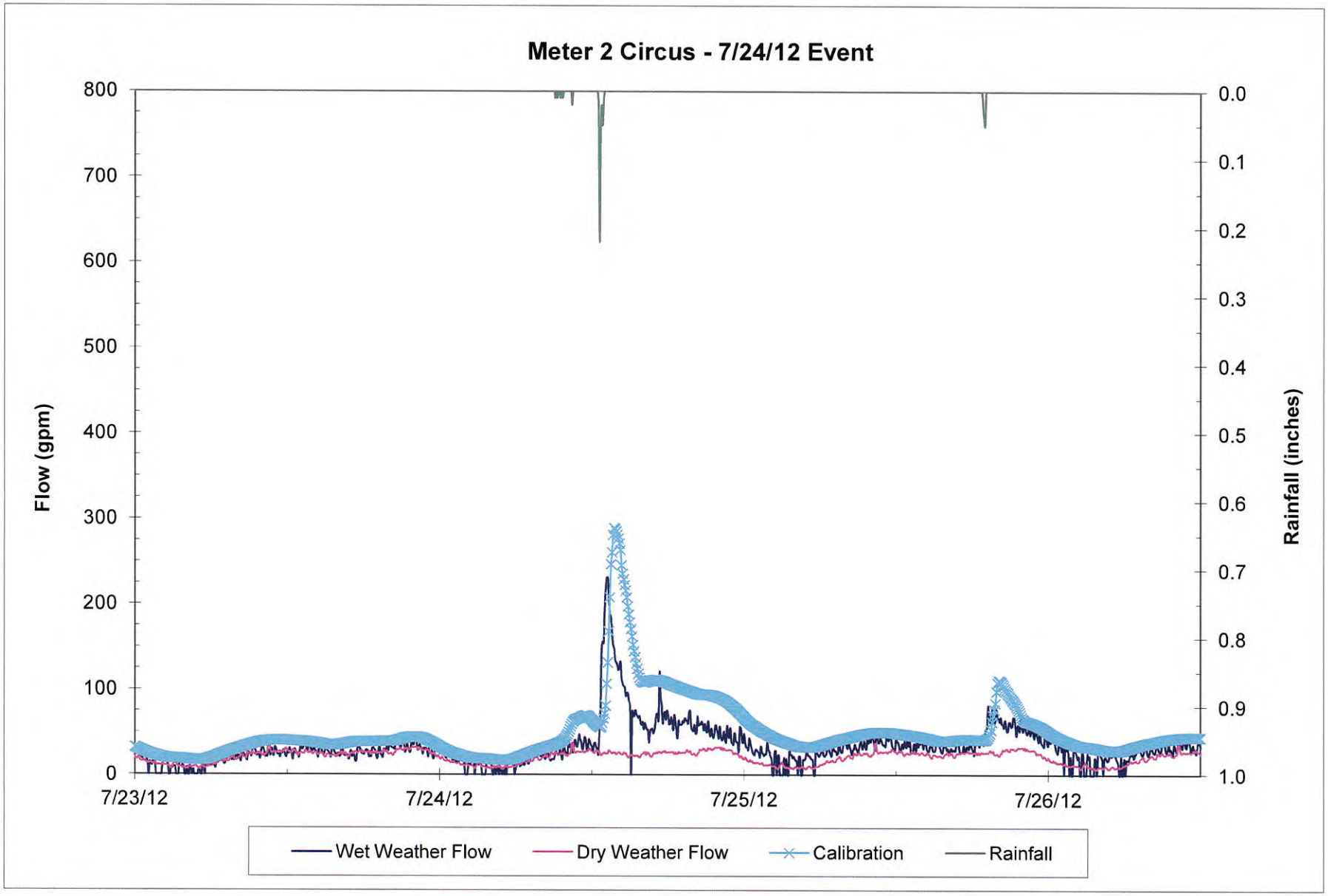
Criteria	Calibration Standard
Peak Flow Rate	-15% to +25% of Observed
Wet Weather Volume	-10% to +20% of Observed
Peak Depth	±0.33 ft at non-surcharged locations
	-0.33 feet to +1.67 feet at surcharged locations

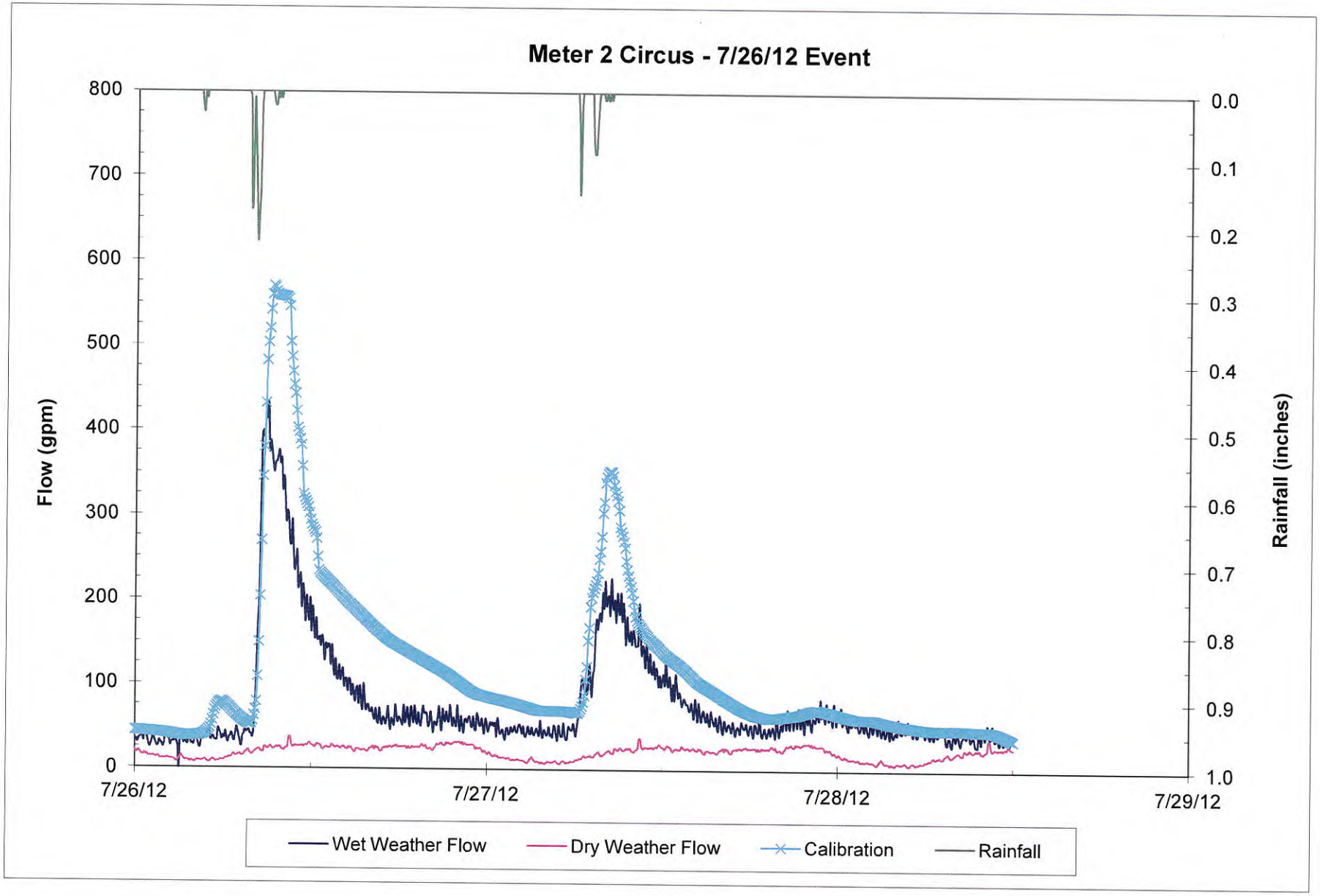

Meter 2 Circus - 5/21/12 Event
Flow (gpm)
800
700
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
5/21/12
5/22/12
5/23/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall

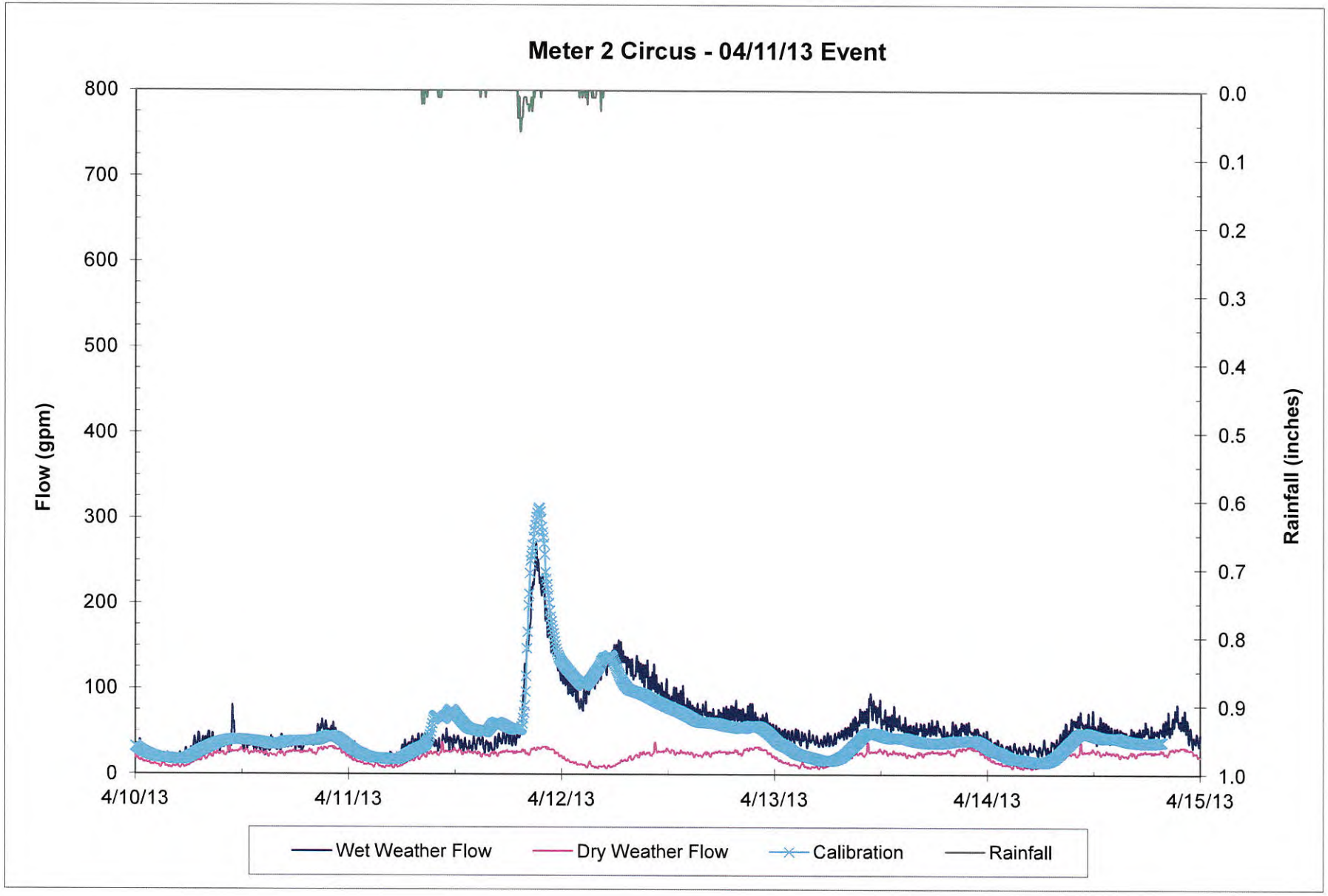

Meter 2 Circus - 6/1/12 Event
Flow (gpm)
800
700
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
5/31/12
6/1/12
6/2/12
6/3/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall

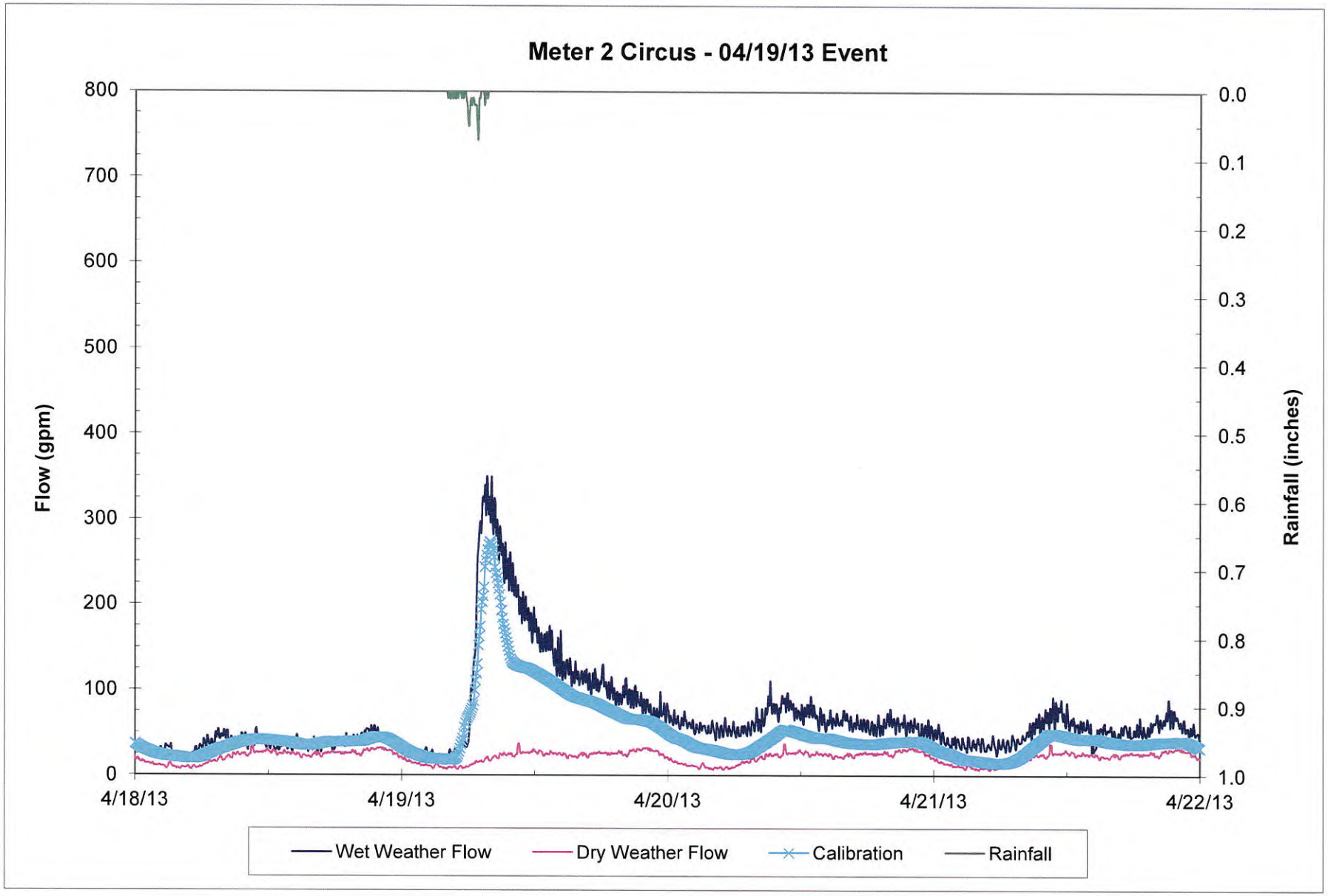

Meter 2 Circus - 6/17/12 Event
Flow (gpm)
800
700
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
6/17/12
6/18/12
6/19/12
6/20/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall

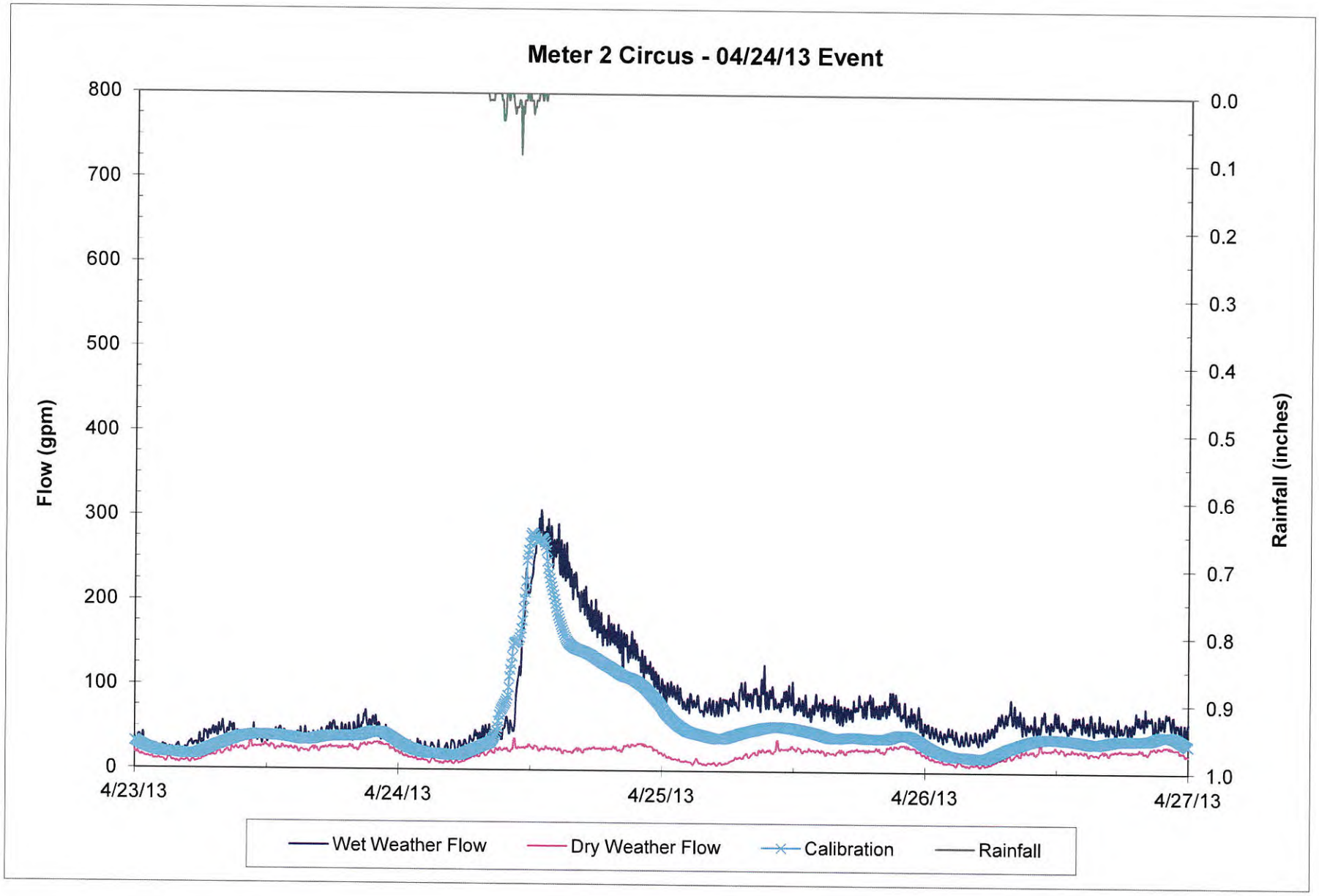

Meter 2 Circus - 7/19/12 Event
Flow (gpm)
800
700
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
7/18/12
7/19/12
7/20/12
7/21/12
7/22/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall

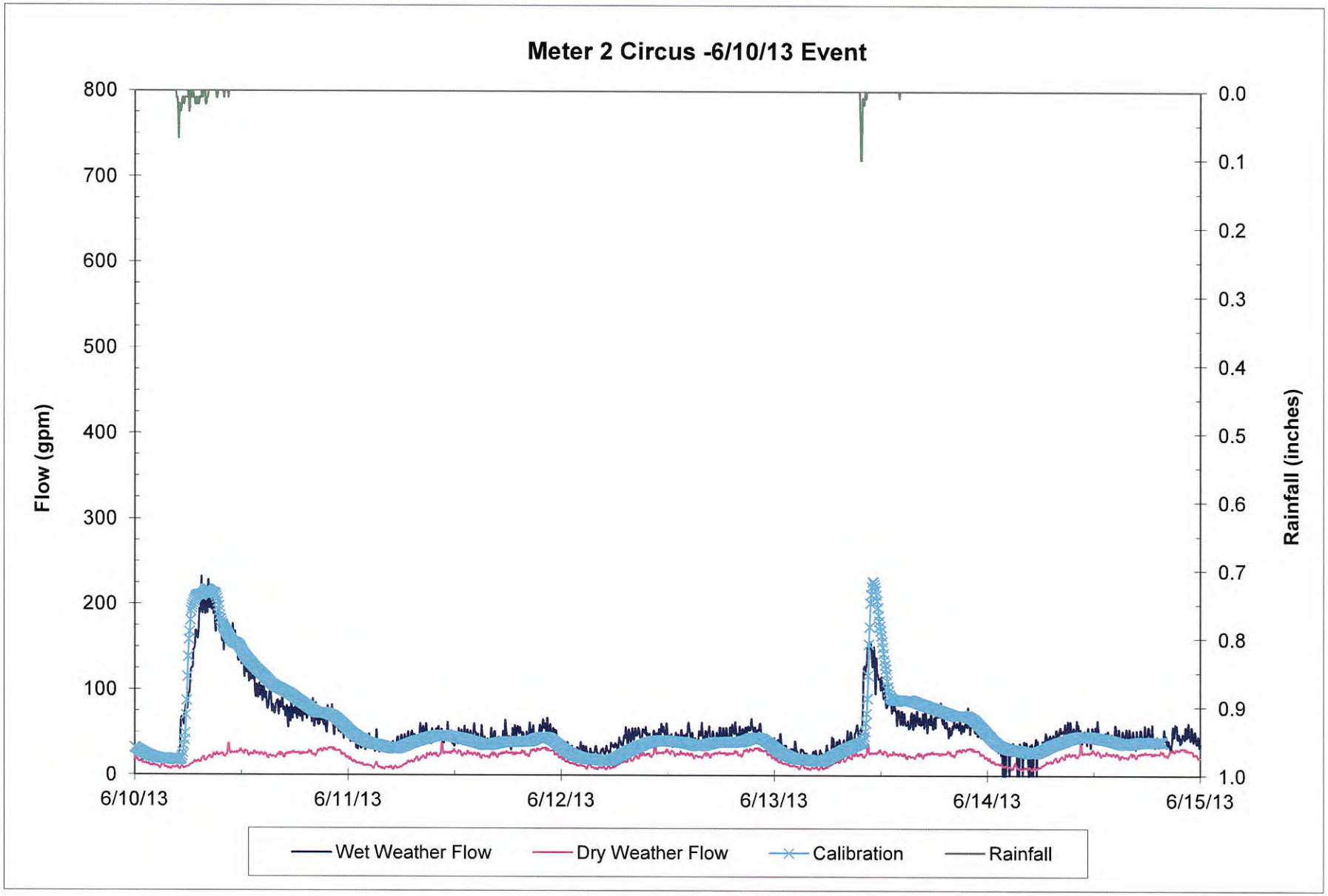

Meter 2 Circus - 7/24/12 Event
Flow (gpm)
800
700
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
7/23/12
7/24/12
7/25/12
7/26/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall

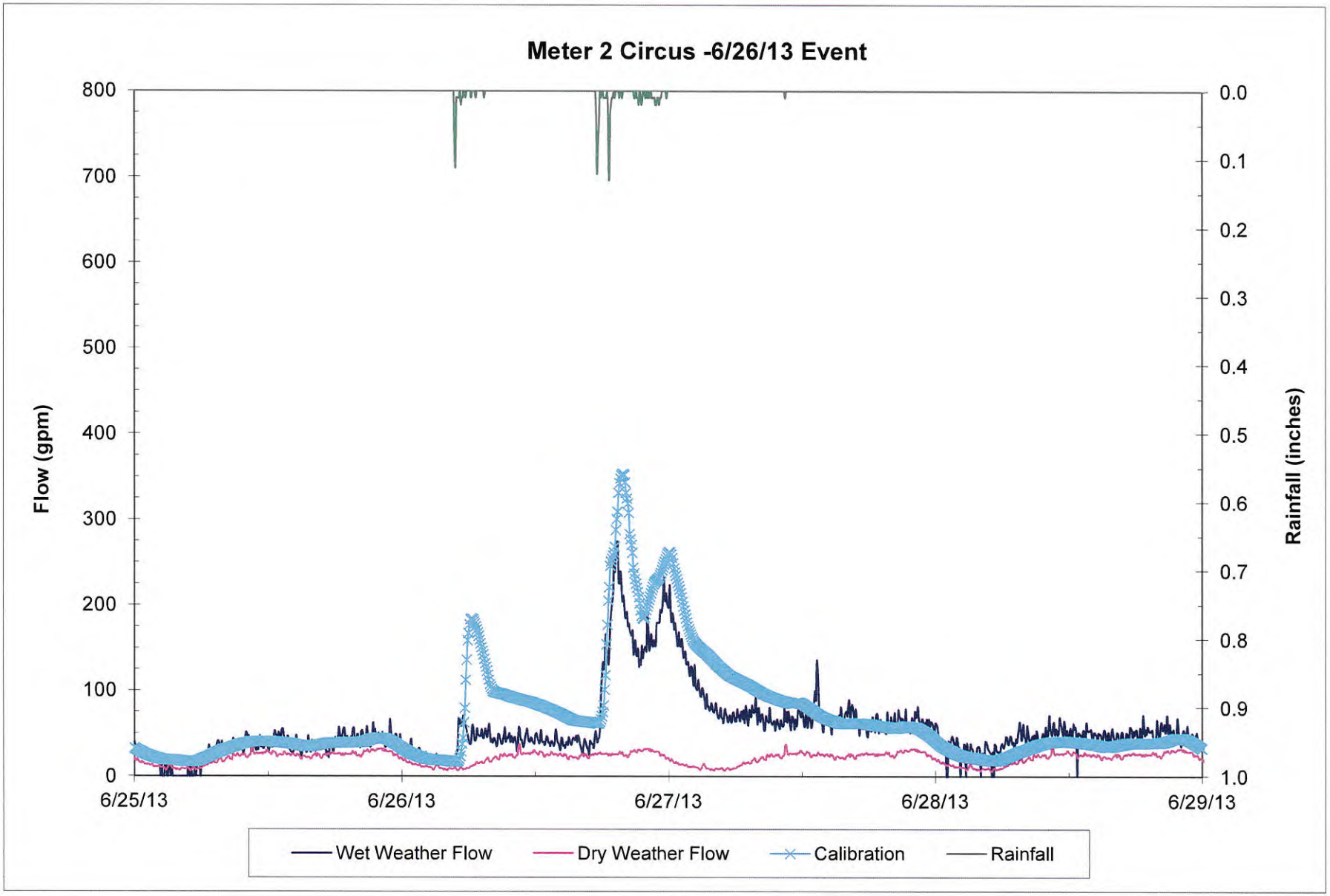

Meter 2 Circus - 7/26/12 Event
Flow (gpm)
800
700
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
7/26/12
7/27/12
7/28/12
7/29/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 2 Circus - 04/11/13 Event
Flow (gpm)
800
700
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
4/10/13
4/11/13
4/12/13
4/13/13
4/14/13
4/15/13
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 2 Circus - 04/19/13 Event
Flow (gpm)
800
700
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
4/18/13
4/19/13
4/20/13
4/21/13
4/22/13
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 2 Circus - 04/24/13 Event
Flow (gpm)
800
700
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
4/23/13
4/24/13
4/25/13
4/26/13
4/27/13
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 2 Circus -6/10/13 Event
Flow (gpm)
800
700
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
6/10/13
6/11/13
6/12/13
6/13/13
6/14/13
6/15/13
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 2 Circus -6/26/13 Event
Flow (gpm)
Rainfall (inches)
800
700
600
500
400
300
200
100
0
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
6/25/13
6/26/13
6/27/13
6/28/13
6/29/13
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall

In-System Flow Meter Calibration Results

Meter Location Summary

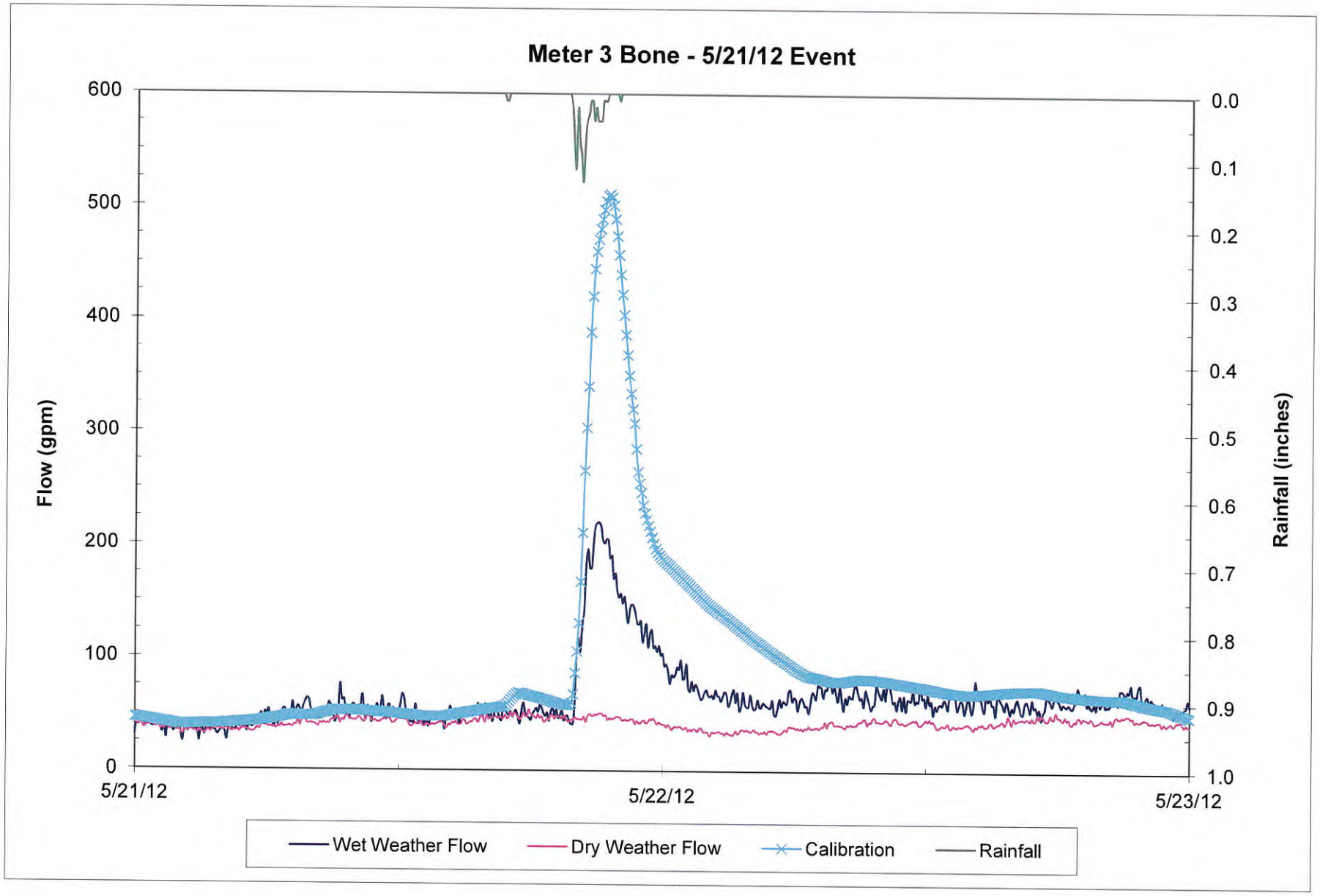
Site ID	Site 3
Location Summary	Bone St and West St
Manhole Structure ID	2655
Model Link ID	2655.1
Pipe Dimension (in)	8
Pipe Shape	Circular

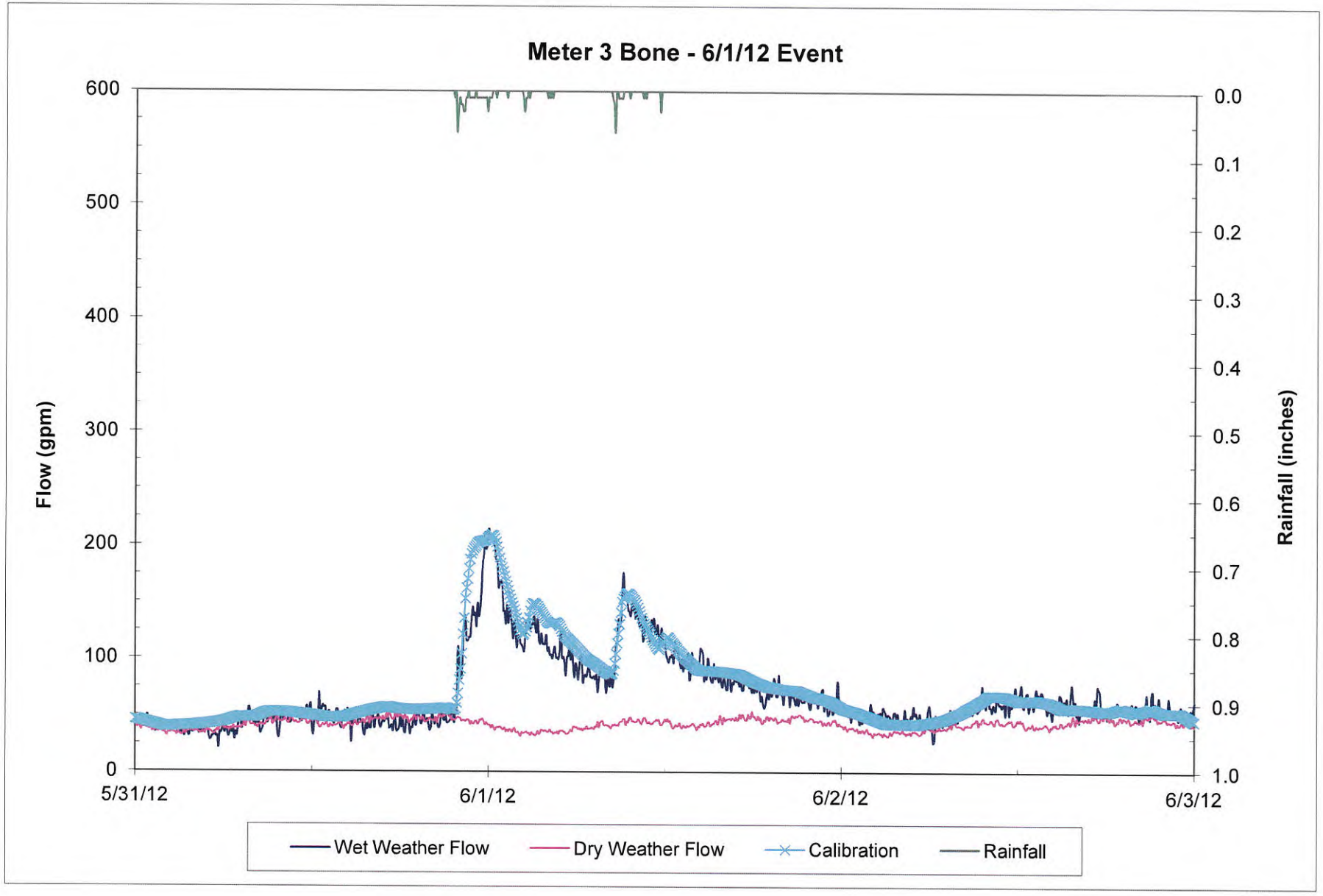
Tabular Summary of Meter-to-Model Volume, Peak Flow, and Depth Comparisons for Calibration

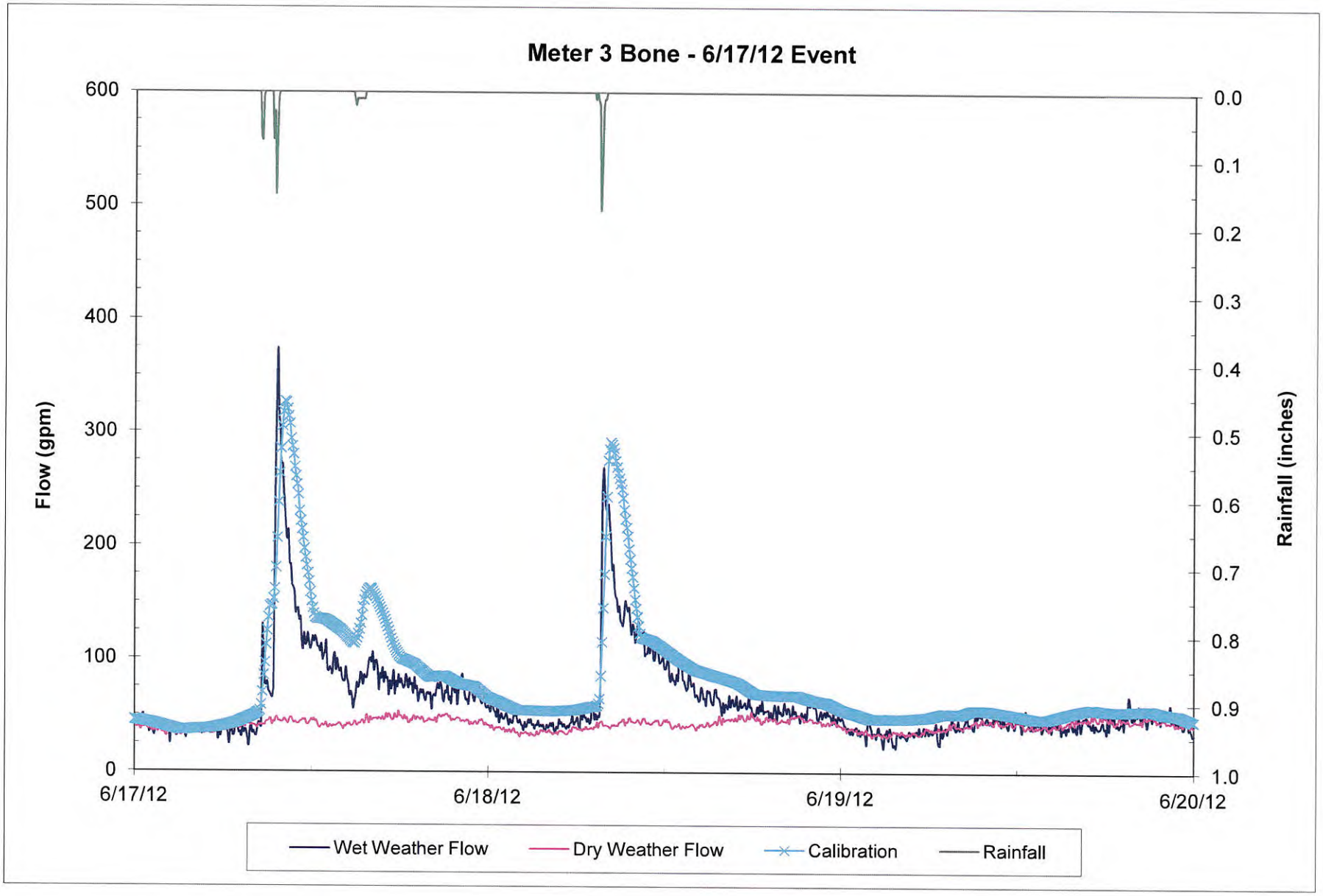
Calibration Periods	Data Comments	Volume (MG)			Peak Flow (gpm)			Peak Depth (ft)		
		Meter	Model	Percent Difference (%)	Meter	Model	Percent Difference (%)	Meter	Model	Difference (ft)
April 11, 2013		0.42	0.47	12%	258	317	23%	0.38	0.40	0.02
April 19, 2013		0.36	0.36	2%	263	288	9%	0.40	0.38	-0.02
April 24, 2013		0.35	0.38	10%	232	286	23%	0.35	0.38	0.03
June 10, 2013		0.30	0.41	36%	214	232	9%	0.33	0.34	0.01
June 26, 2013		0.28	0.45	62%	285	373	31%	0.42	0.44	0.02
Total No. of Available Periods		5			5			5		
Total No. of Periods Meeting Wet-Weather Calibration Criteria		3			4			5		

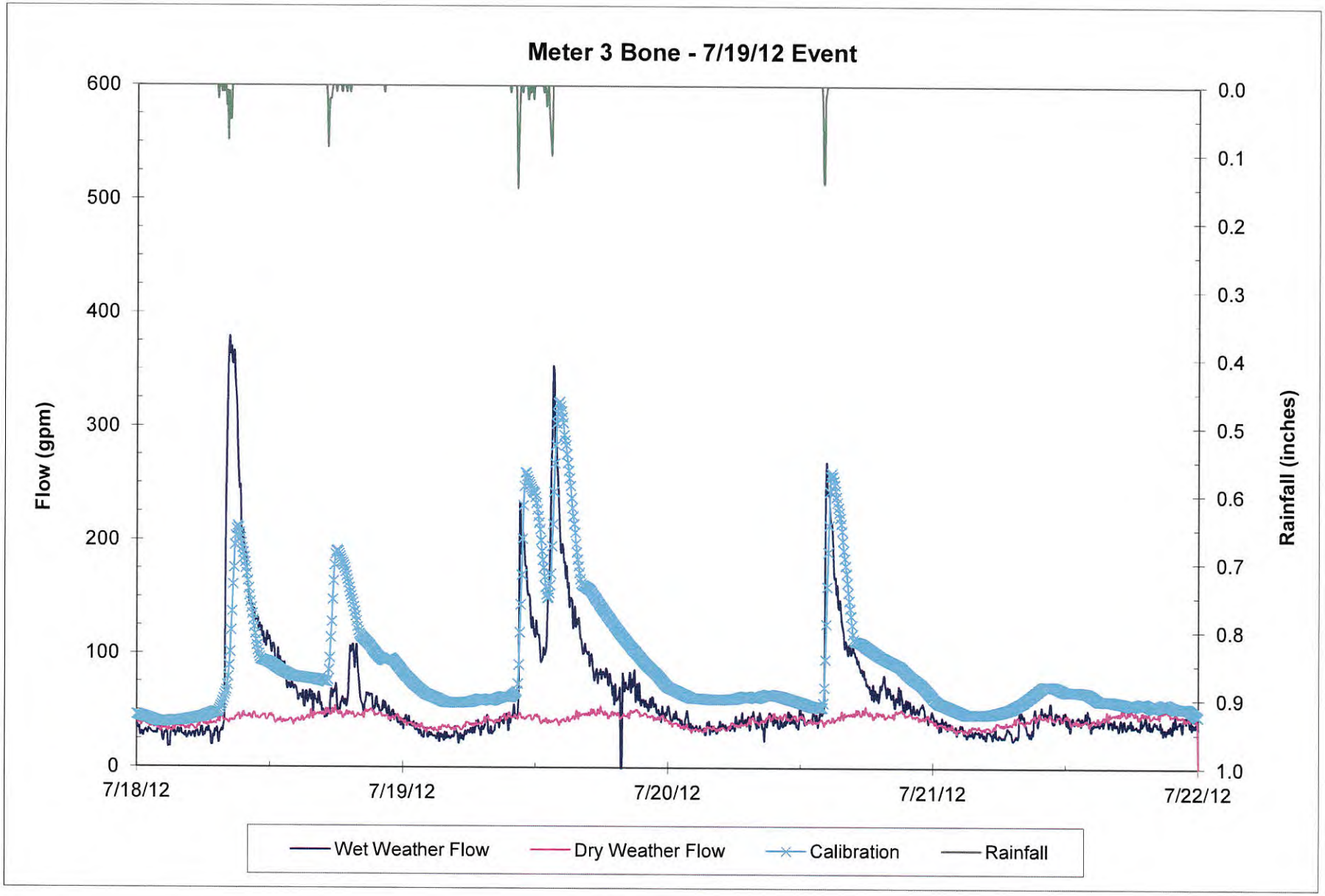
Wet-Weather Calibration Criteria

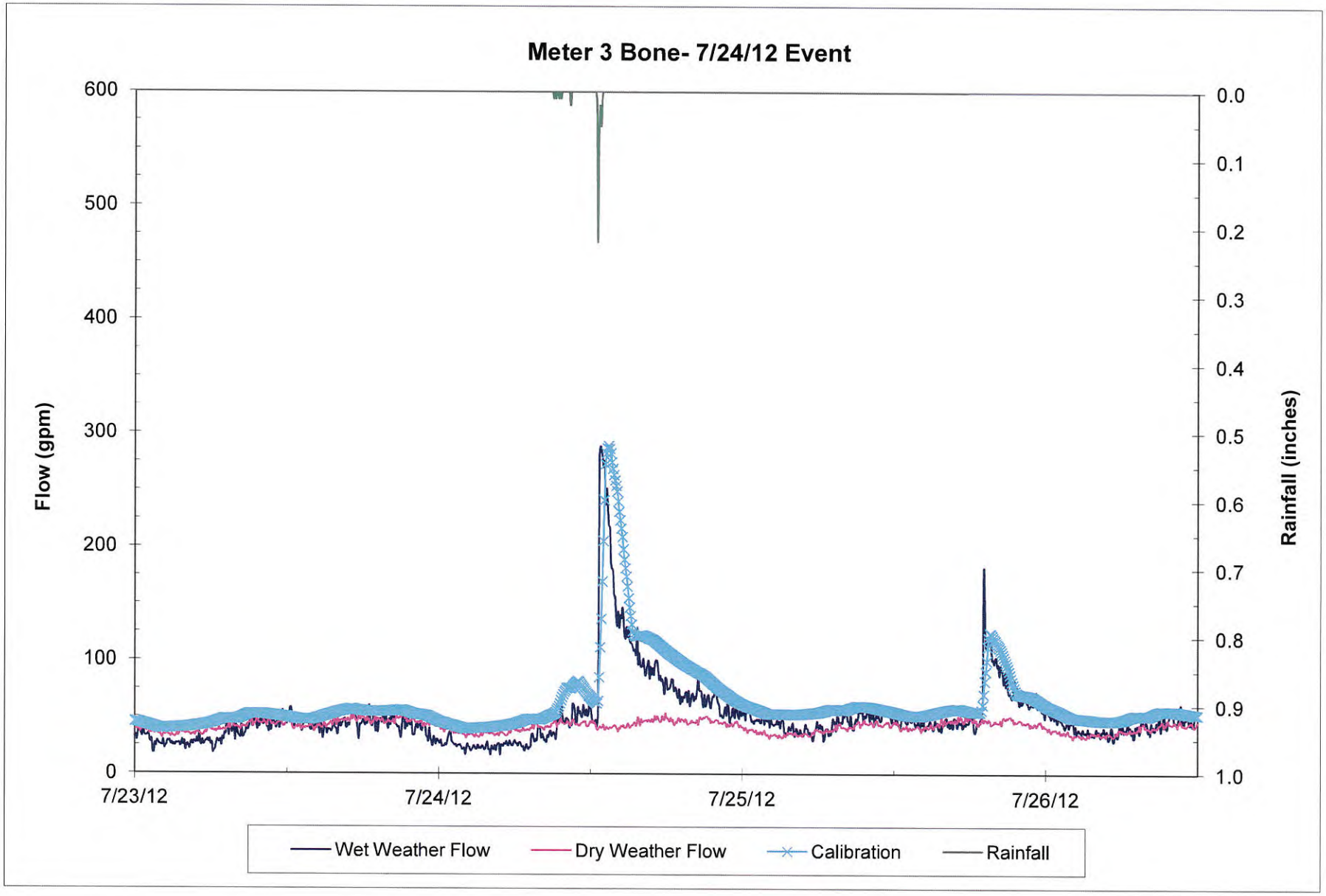
Criteria	Calibration Standard
Peak Flow Rate	-15% to +25% of Observed
Wet Weather Volume	-10% to +20% of Observed
Peak Depth	±0.33 ft at non-surcharged locations
	-0.33 feet to +1.67 feet at surcharged locations

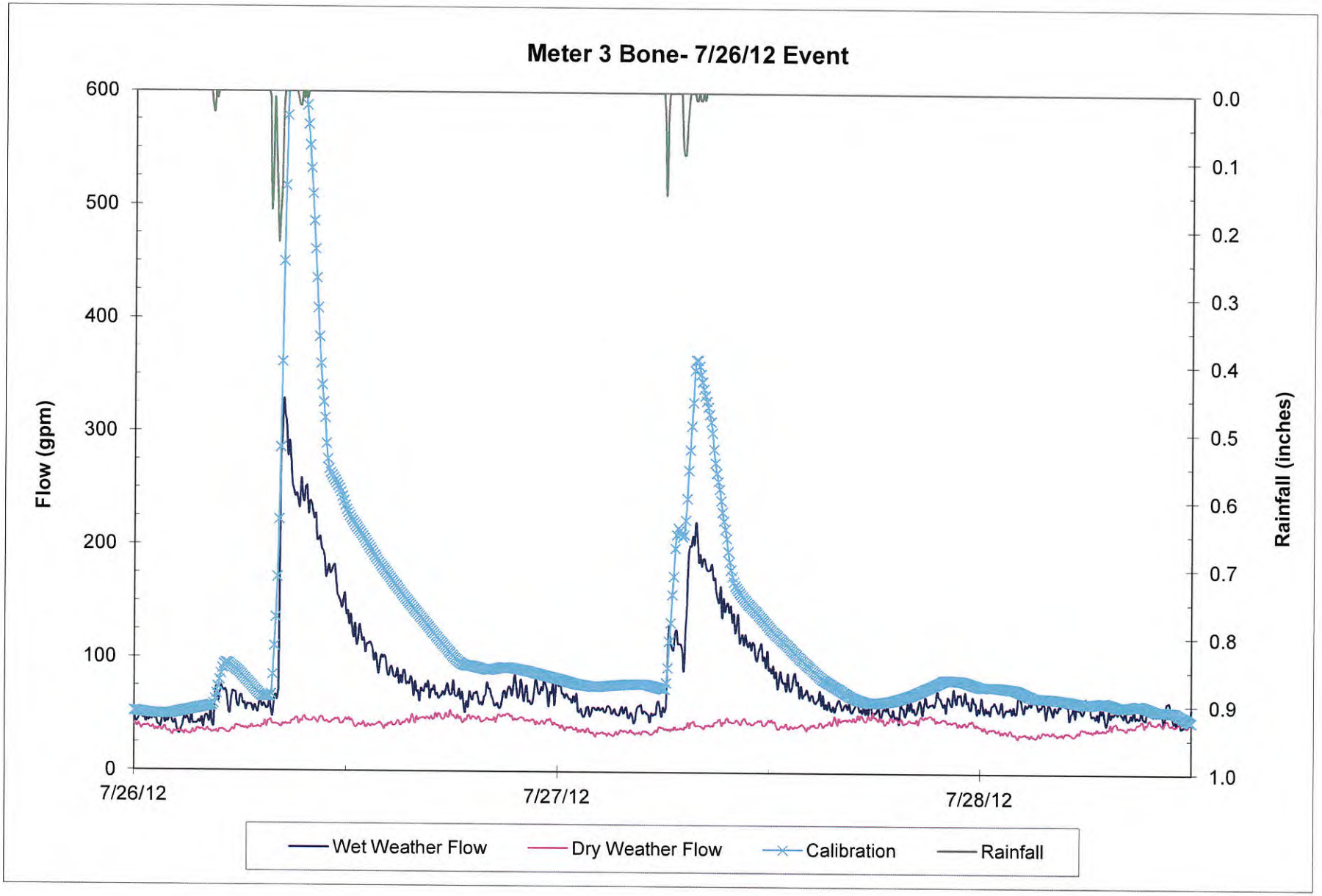

Meter 3 Bone - 5/21/12 Event
Flow (gpm)
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
5/21/12
5/22/12
5/23/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 3 Bone - 6/1/12 Event
Flow (gpm)
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
5/31/12
6/1/12
6/2/12
6/3/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 3 Bone - 6/17/12 Event
Flow (gpm)
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
6/17/12
6/18/12
6/19/12
6/20/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 3 Bone - 7/19/12 Event
Flow (gpm)
Rainfall (inches)
600
500
400
300
200
100
0
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
7/18/12
7/19/12
7/20/12
7/21/12
7/22/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 3 Bone- 7/24/12 Event
Flow (gpm)
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
7/23/12
7/24/12
7/25/12
7/26/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall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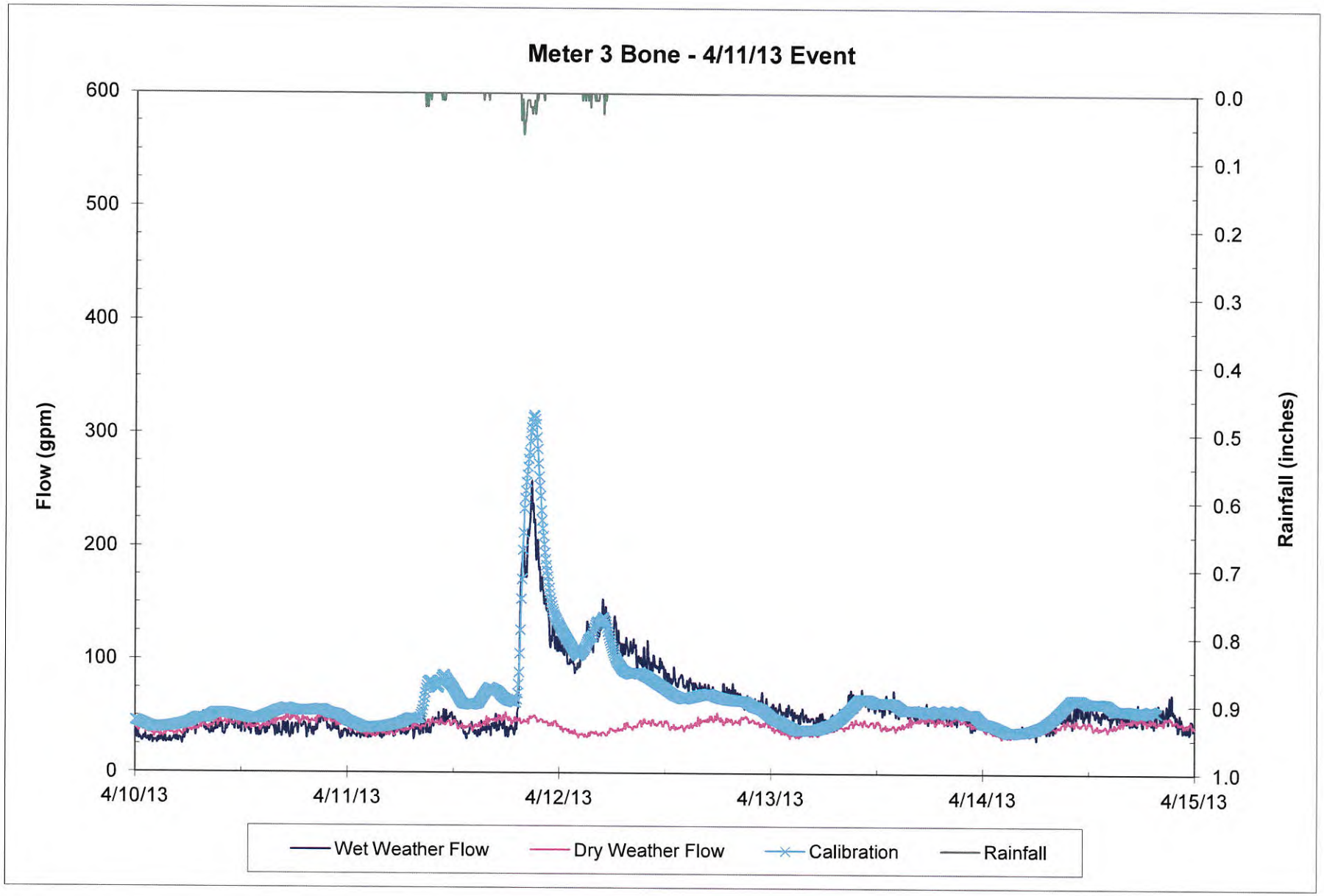
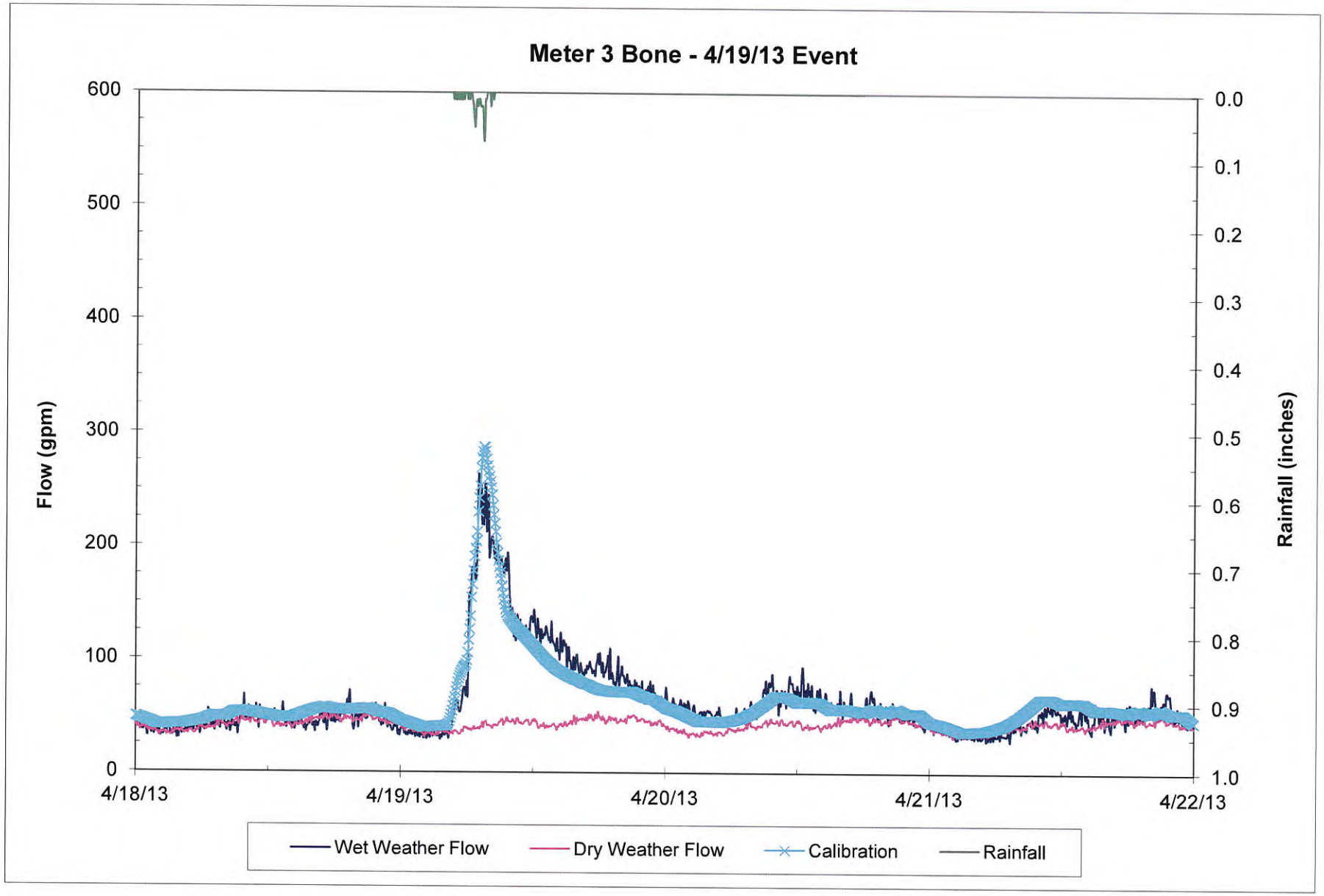
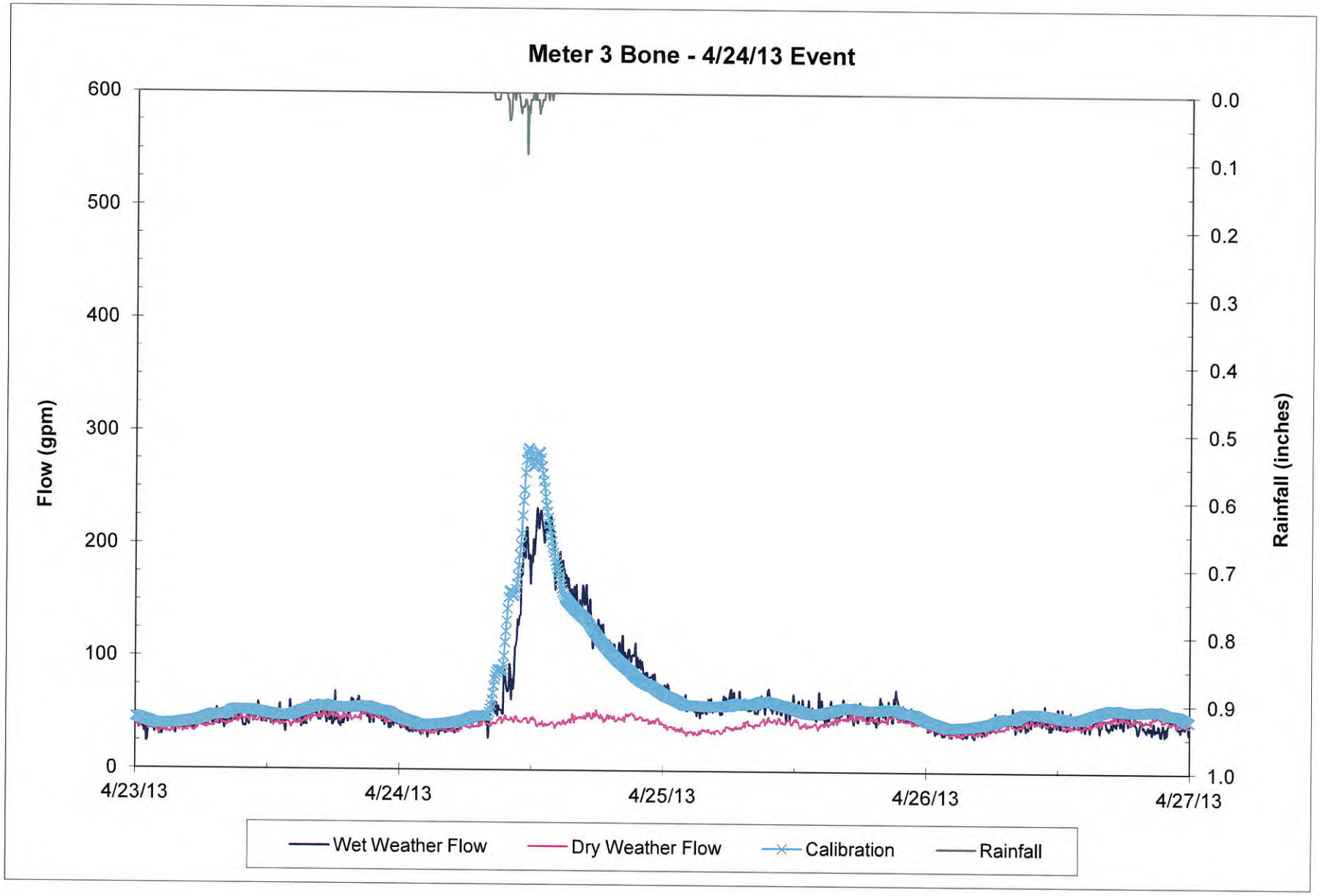
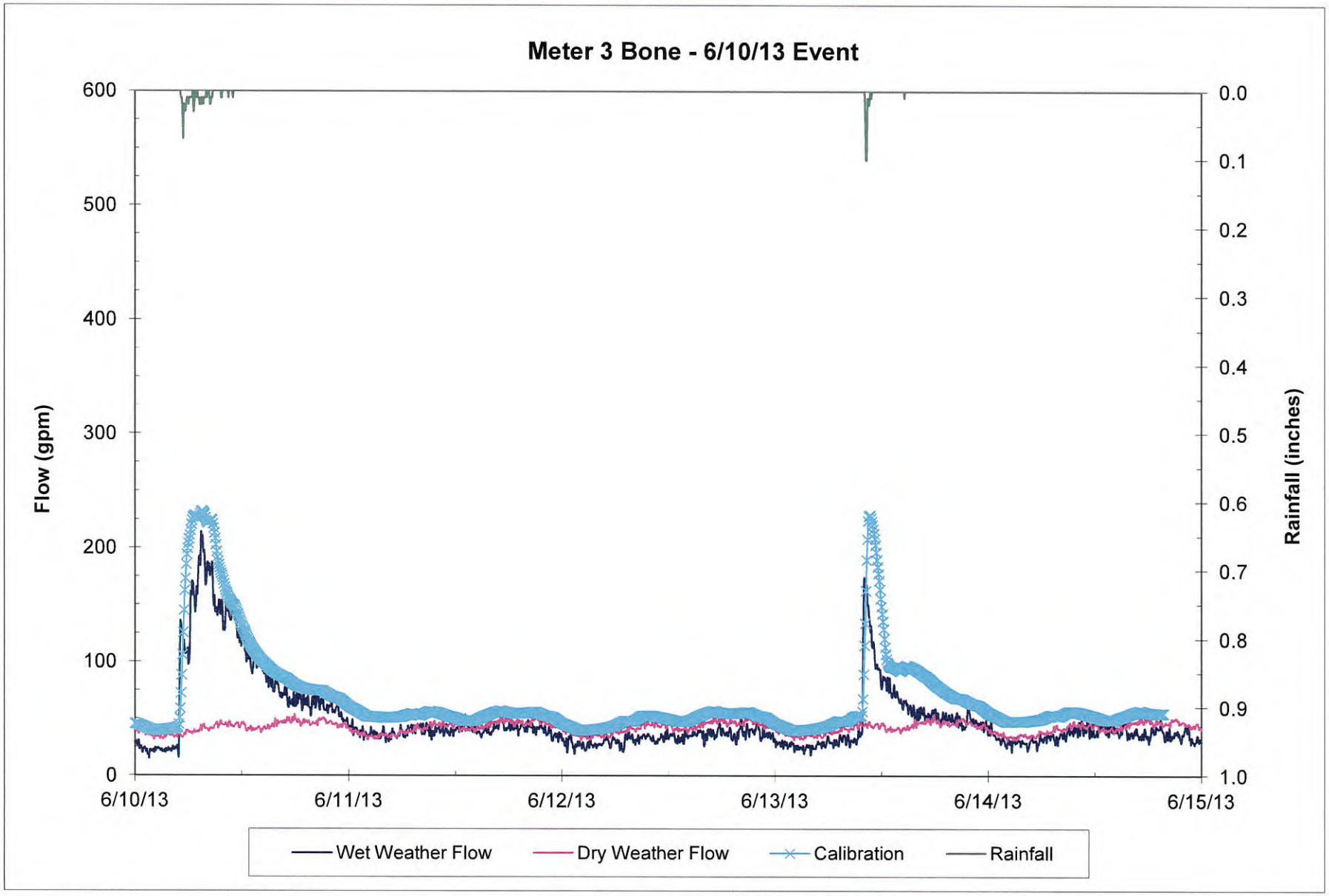
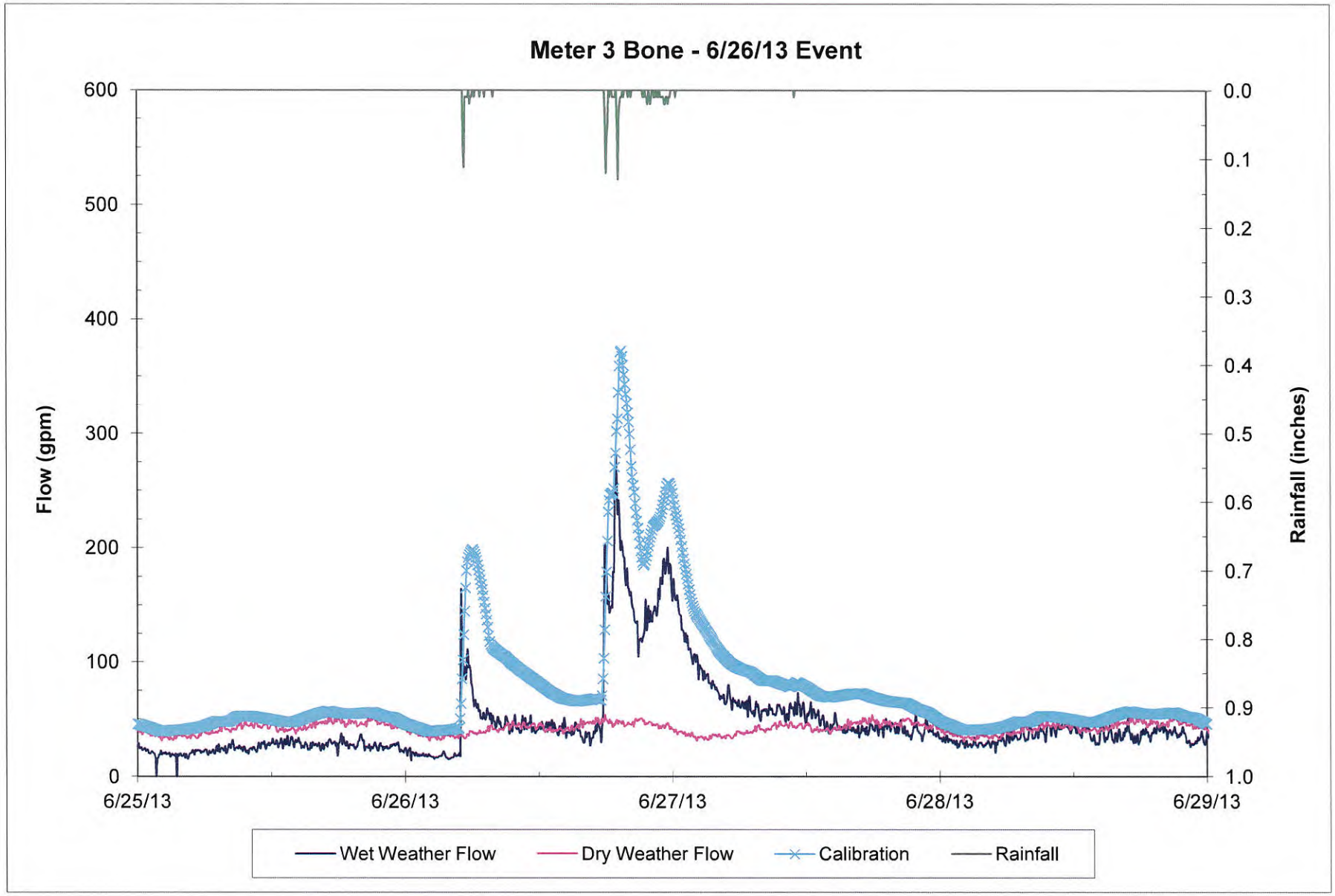
Flow (gpm)
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
7/26/12
7/27/12
7/28/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 3 Bone - 4/11/13 Event
Flow (gpm)
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
4/10/13
4/11/13
4/12/13
4/13/13
4/14/13
4/15/13
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 3 Bone - 4/19/13 Event
Flow (gpm)
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
4/18/13
4/19/13
4/20/13
4/21/13
4/22/13
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 3 Bone - 4/24/13 Event
Flow (gpm)
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
4/23/13
4/24/13
4/25/13
4/26/13
4/27/13
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 3 Bone - 6/10/13 Event
Flow (gpm)
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
6/10/13
6/11/13
6/12/13
6/13/13
6/14/13
6/15/13
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 3 Bone - 6/26/13 Event
Flow (gpm)
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
6/25/13
6/26/13
6/27/13
6/28/13
6/29/13
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall

In-System Flow Meter Calibration Results

Meter Location Summary

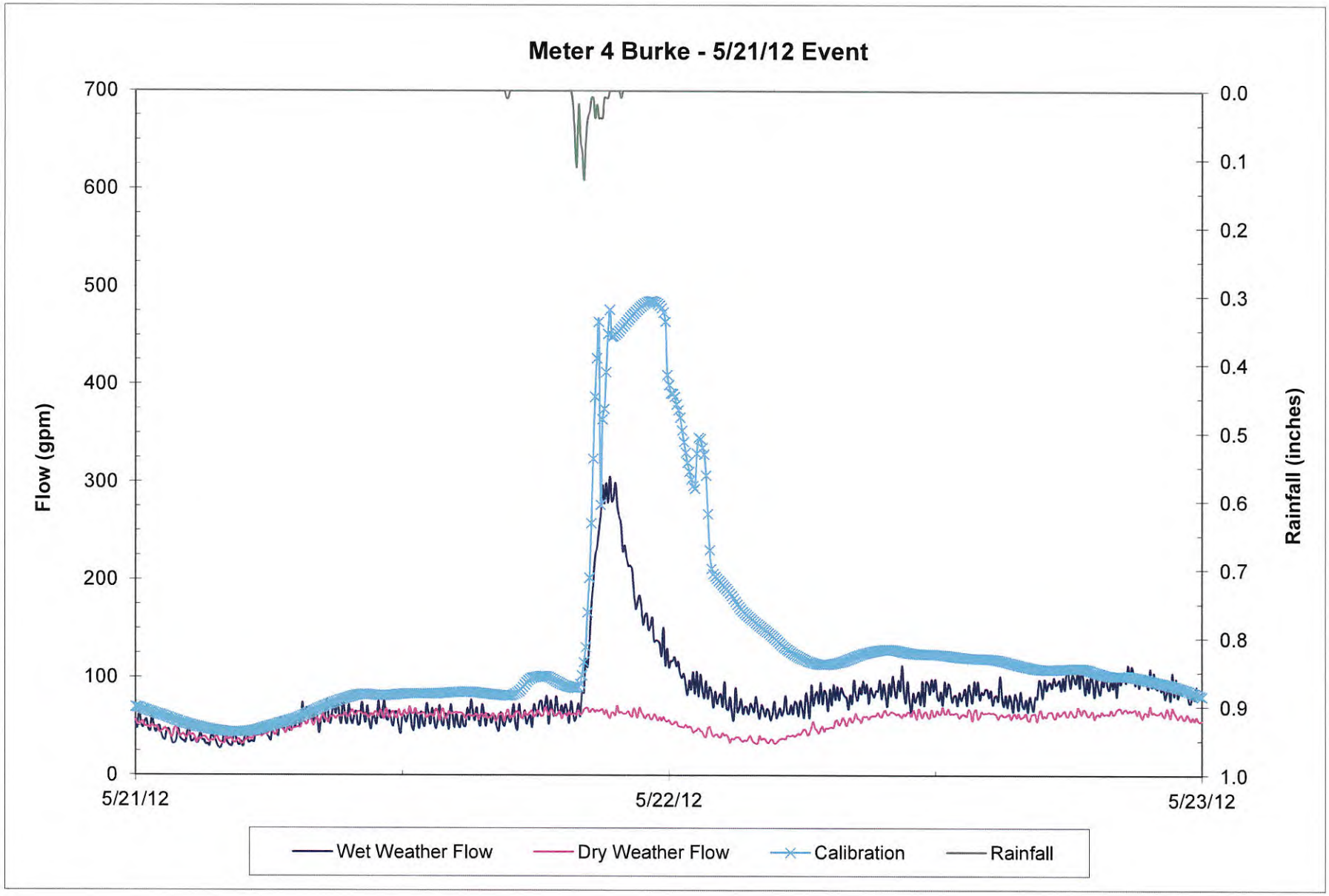
Site ID	Site 4
Location Summary	Burke Park - next to bridge
Manhole Structure ID	2593
Model Link ID	2593.1
Pipe Dimension (in)	10
Pipe Shape	Circular

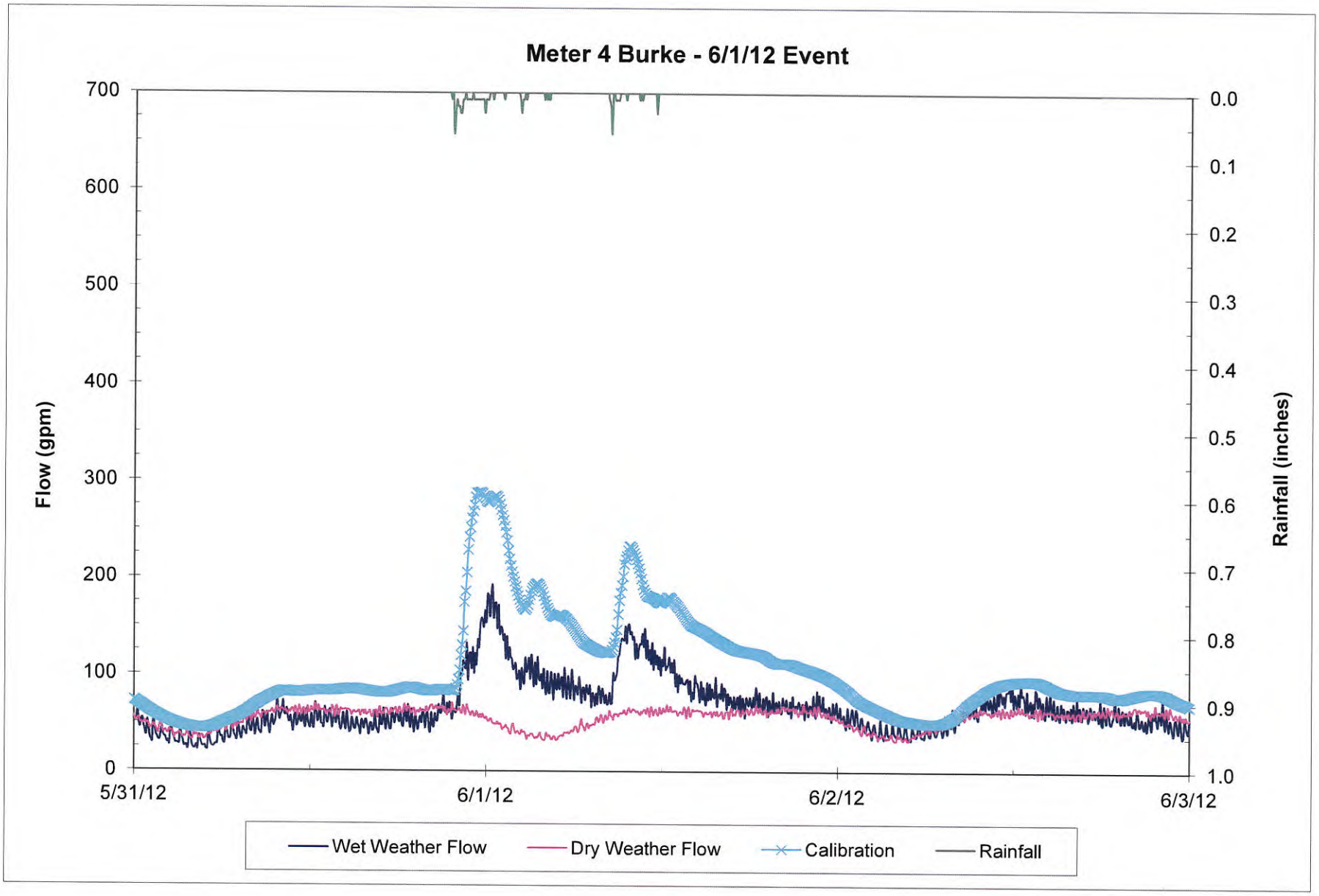
Tabular Summary of Meter-to-Model Volume, Peak Flow, and Depth Comparisons for Calibration

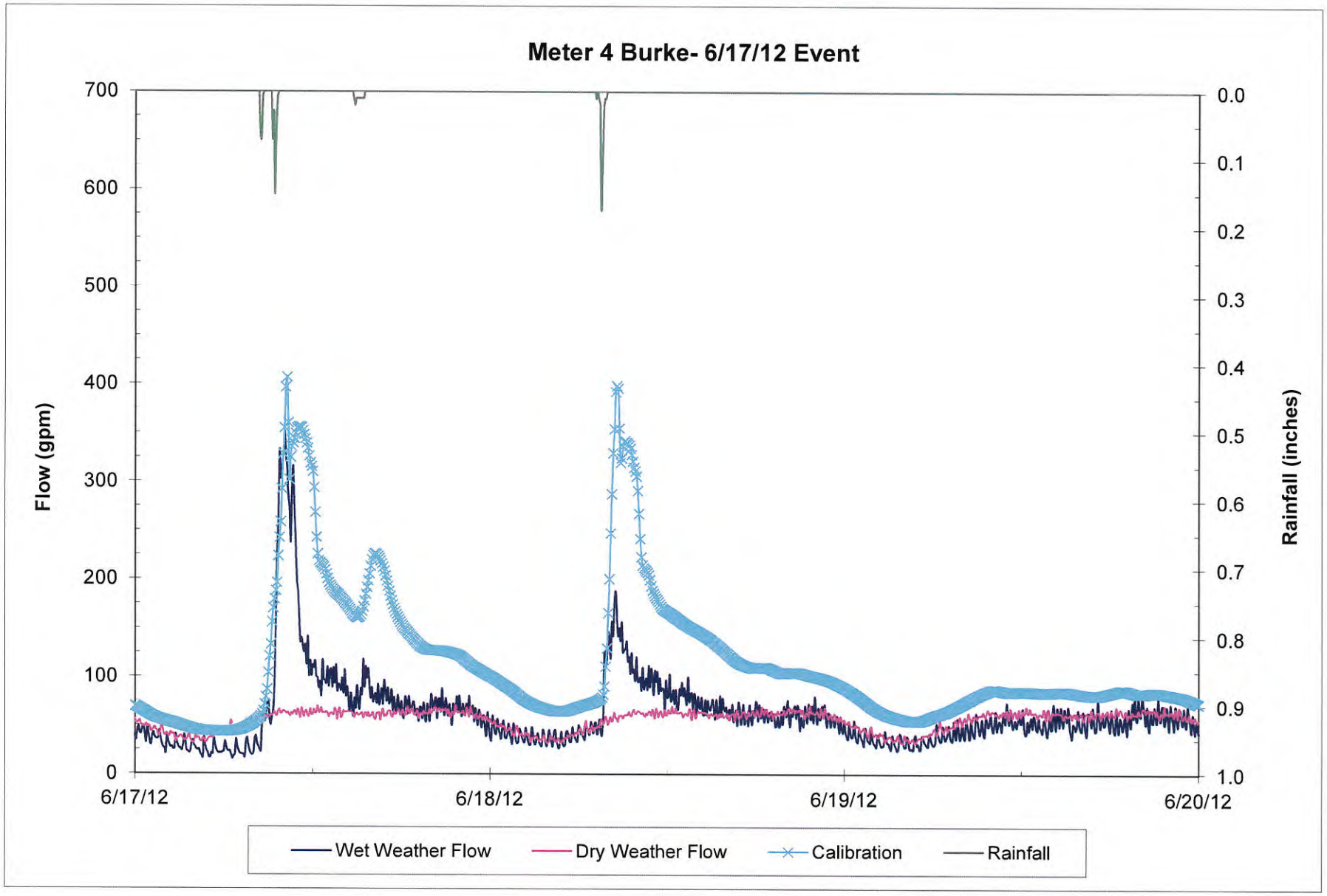
Calibration Periods	Data Comments	Volume (MG)			Peak Flow (gpm)			Peak Depth (ft)		
		Meter	Model	Percent Difference (%)	Meter	Model	Percent Difference (%)	Meter	Model	Difference (ft)
April 11, 2013		0.61	0.57	-6%	391	391	0%	0.83	3.52	2.68
April 19, 2013		0.67	0.52	-23%	460	390	-15%	1.23	1.95	0.72
April 24, 2013		0.67	0.55	-18%	421	373	-11%	1.08	3.26	2.18
June 10, 2013		0.50	0.59	18%	362	333	-8%	0.94	1.01	0.07
June 26, 2013		0.59	0.66	12%	439	420	-4%	1.70	4.95	3.25
Total No. of Available Periods		5			5			5		
Total No. of Periods Meeting Wet-Weather Calibration Criteria		3			5			1		

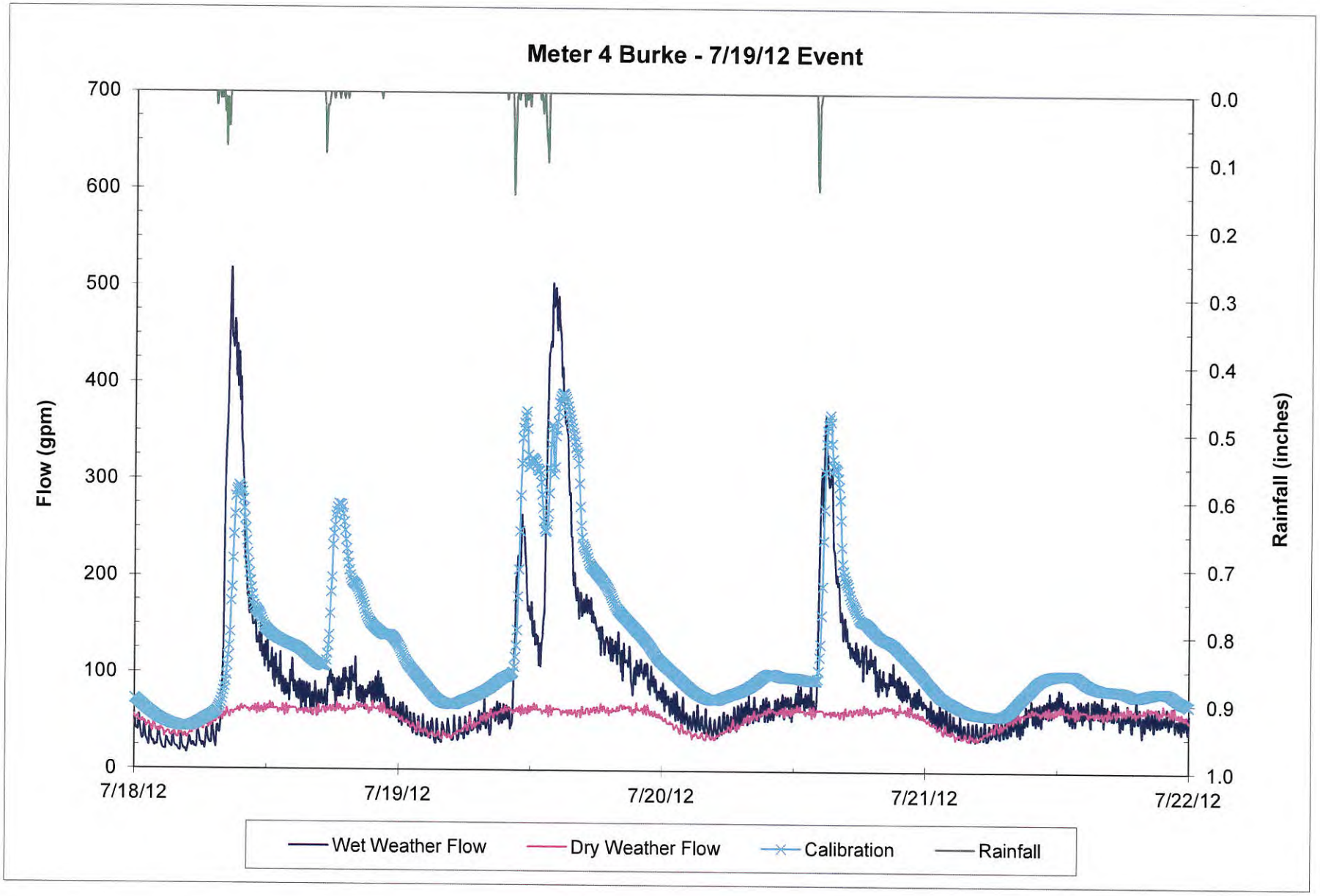
Wet-Weather Calibration Criteria

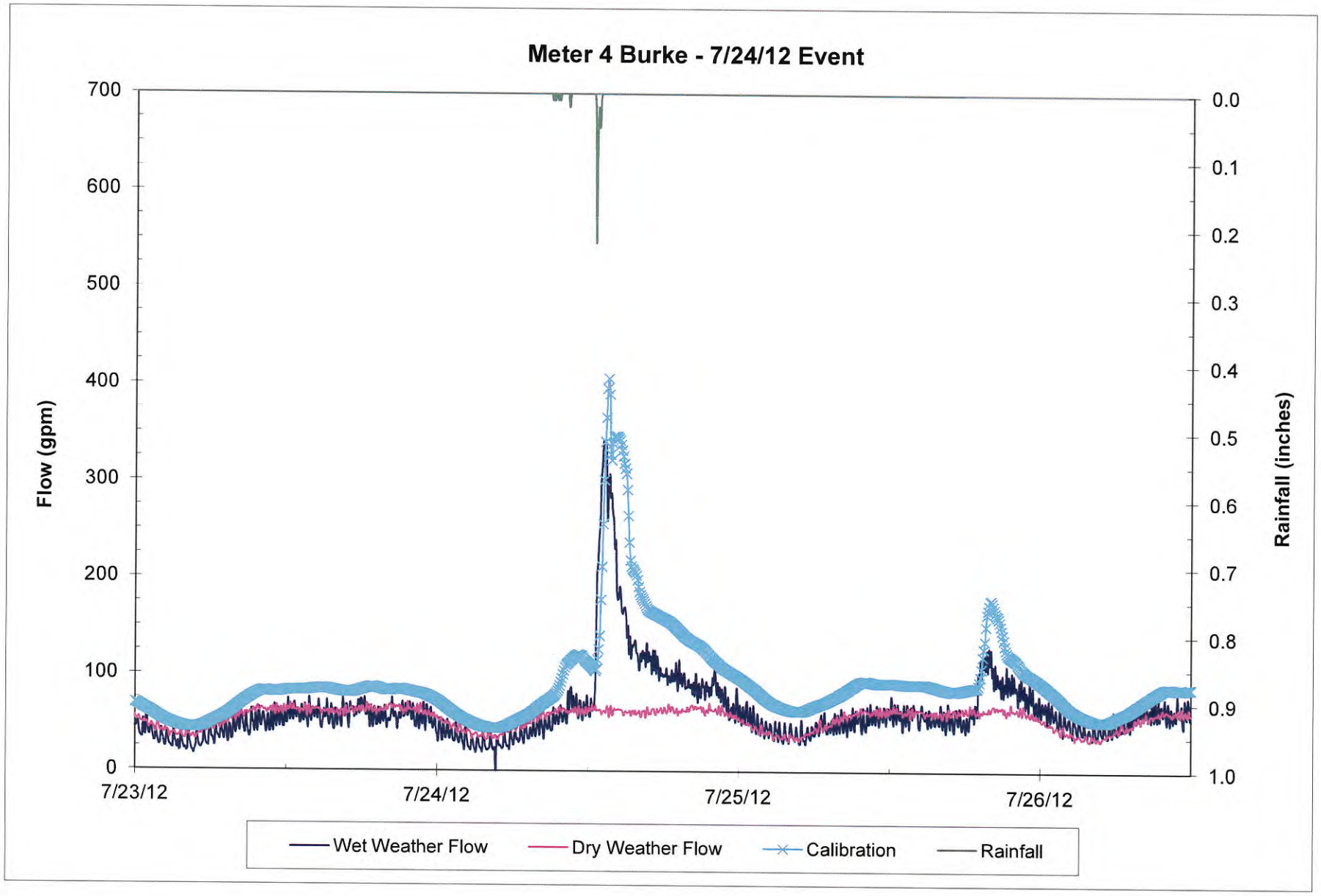
Criteria	Calibration Standard
Peak Flow Rate	-15% to +25% of Observed
Wet Weather Volume	-10% to +20% of Observed
Peak Depth	±0.33 ft at non-surcharged locations
	-0.33 feet to +1.67 feet at surcharged locations


Meter 4 Burke - 5/21/12 Event
Flow (gpm)
700
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
5/21/12
5/22/12
5/23/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 4 Burke - 6/1/12 Event
Flow (gpm)
700
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
5/31/12
6/1/12
6/2/12
6/3/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 4 Burke- 6/17/12 Event
Flow (gpm)
Rainfall (inches)
700
600
500
400
300
200
100
0
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
6/17/12
6/18/12
6/19/12
6/20/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 4 Burke - 7/19/12 Event
Flow (gpm)
700
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
7/18/12
7/19/12
7/20/12
7/21/12
7/22/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall


Meter 4 Burke - 7/24/12 Event
Flow (gpm)
700
600
500
400
300
200
100
0
Rainfall (inches)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
7/23/12
7/24/12
7/25/12
7/26/12
Wet Weather Flow
Dry Weather Flow
Calibration
Rainfall

Appendix B

Geotechnical Report

GEOTECHNICAL EXPLORATION

BETHEL LIFT STATION EXPANSION

2950 S.R. 125

TATE TOWNSHIP,
CLERMONT COUNTY, OHIO

Prepared for: **County of Clermont**
Water Resources Department
Thelen Project No.: **120697NE**

THELEN ASSOCIATES, INC.
Geotechnical • Testing Engineers
• 1780 Carillon Boulevard, Cincinnati, Ohio 45240-2795 / 513-825-4350 / Fax 513-825-4756
www.thelenassoc.com
Offices
Erlanger, Kentucky
Lexington, Kentucky
Cincinnati, Ohio
Dayton, Ohio

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Cincinnati, Ohio
Dayton, Ohio

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October 5, 2012

County of Clermont
Water Resources Department
4400 Haskell Lane
Batavia, Ohio 45103

Attention: Mr. Chris Rowland, P.E.

RE: Geotechnical Exploration
Bethel Lift Station Expansion
2950 S.R. 125
Tate Township, Clermont County, Ohio

Ladies and Gentlemen:

Contained herein are the results of a geotechnical exploration performed for the expansion of the Bethel Lift Station, located at 2950 S.R. 125 in Tate Township, Clermont County, Ohio. These services were requested by the Project Engineer, Mr. Chris Rowland, Clermont county Water Resources Department, in an email on August 23, 2012. Our services were completed per our extended Contract for Professional Services, P.O. 11003621-00 dated June 28, 2011.

We are enclosing with this report a reprint of "Important Information About Your Geotechnical Engineering Report" published by ASFE, Professional Firms Practicing in the Geosciences, which our firm would like to introduce to you at this time.

We appreciate the opportunity to provide the geotechnical exploration for the lift station upgrades. Should you have any questions concerning the information, conclusions or recommendations contained in this report, please do not hesitate to contact us.

Respectfully submitted,
THELEN ASSOCIATES, INC.

STATE OF OHIO
CHRISTOPHER C. HAMANT
E-63811
REGISTERED PROFESSIONAL ENGINEER

Christopher C. Hamant, P.E.
Senior Geotechnical Engineer

10/5/2012

Arthur T. Sturbaum, P.E.
Senior Geotechnical Engineer

CCH:ATS:ph
120697NE
Copies submitted: Client (e)
FTC&H, Inc. (e)

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APPENDIX

THELEN ASSOCIATES, INC.
Geotechnical • Testing Engineers
• 1780 Carillon Boulevard, Cincinnati, Ohio 45240-2795 / 513-825-4350 / Fax 513-825-4756
www.thelenassoc.com
Offices
Erlanger, Kentucky
Lexington, Kentucky
Cincinnati, Ohio
Dayton, Ohio

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October 5, 2012

GEOTECHNICAL EXPLORATION
BETHEL LIFT STATION EXPANSION
2950 S.R. 125
TATE TOWNSHIP,
CLERMONT COUNTY, OHIO

1.0 INTRODUCTION

This report contains the results of a geotechnical exploration performed for the expansion of the Bethel Sanitary Lift Station located at 2950 S.R. 125 in Tate Township, Clermont County, Ohio.

2.0 SCOPE

The purposes of this geotechnical exploration were to determine the general subsurface profile in the area of the proposed new facilities and to relate the engineering properties of the subsurface soils; that is their strength, classification and compressibility characteristics to foundation support for the proposed structures and to related construction.

3.0 PROJECT CHARACTERISTICS

The project is being designed by Fishbeck Thompson Carr and Huber, Inc. (FTC&H) in collaboration with the Clermont County Water Resources Department.

The expansion of the Bethel Lift Station is anticipated to include either a rectangular tank next to the existing facilities, with a base elevation near El. 800 or a circular, above-ground tank, which may be set further east on the site.

Details of the proposed tank were not complete at the time these services were performed. We anticipate that the walls of either tank type will be supported on a reinforced concrete mat foundation, the surface of which will slope toward the center of the tank. The thickness of the mat-slab at the perimeter is typically 12 to 18 inches and the tank is anticipated to be open-top.

Earthwork for the new facilities will consist predominantly of the cuts required to install the tank. Grades surrounding the tank will likely remain relatively unchanged.

4.0 FIELD EXPLORATION

To determine the subsurface profile at the site, six (6) test borings were performed. The test boring locations were requested and staked in the field by representatives of FTC&H, Inc. These locations are indicated on the Boring Plan, Drawing 120697NE-1, which was prepared from an electronic copy of the existing site plan provided by FTC&H.

Ground surface elevations for the test borings were determined by the FTC&H survey crew as they relate to Mean Sea Level (MSL). The provided elevations were transferred to the test boring logs.

The test borings were performed with a track-mounted drill rig and hollow-stem augers. Two-inch O.D. split-spoon samples were obtained in advance of augering in accordance with the procedures of ASTM D1586. This procedure is described as the standard drive sample method and results in the standard penetration test. In addition to split-spoon sampling, two (2) 3-inch O.D. Shelby tubes were obtained according to the procedures of ASTM D1587.

Test Boring 2 was extended into the interbedded shale and limestone bedrock by rock coring with an NXM-size diamond tipped core barrel.

Representative portions of the split-spoon samples were placed in glass jars and the Shelby tubes were capped and taped to maintain the soils at their in situ moisture contents. Rock cores were placed in cardboard core boxes in order of sample recovery. All samples were appropriately marked in the field for future identification.

Concurrent with the drilling operation, the Drilling Technician prepared field test boring logs of the subsurface profile, noting sample types and depths, soil and bedrock stratifications, standard penetration test resistances (N-values), groundwater levels or the lack thereof and other pertinent data.

5.0 LABORATORY REVIEW

Following completion of the test borings, the samples were returned to our Soil Mechanics Laboratory where they were reviewed and visually classified by the Project Geotechnical Engineer. Representative samples were selected for laboratory testing, including natural moisture content and Atterberg limits classification tests and unconfined compressive strength tests. Rock cores were logged per Ohio Department of Transportation Visual Criteria and were photographed. A tabulation of the laboratory test results and the rock core photographs are included in the Appendix to this report.

Based on the Project Geotechnical Engineer's visual classification of the samples, the Drilling Technician's field logs and the results of the laboratory tests, the final test boring logs were prepared. Copies of these logs are included in the Appendix along with a Soil Classification Sheet describing the terms and symbols used in their preparation.

The dashed lines on the test boring logs identify changes between soil and bedrock types which were determined by interpolation between the samples and should be considered to be approximate. Only changes which occur within samples can be

precisely determined and are indicated by solid lines on the logs. The transition between soil and bedrock types may be abrupt or gradual.

6.0 EXISTING SITE CONDITIONS

The existing lift station is located along the east bank of Poplar Creek. The flow line of the creek is on the order of 20 feet below the ground surface surrounding the lift station. The expansion is to extend east from the existing facility, away from the creek bed. This area rises along the nose of a small ridge, such that there is 11 feet of relief between the easternmost boring (Test Boring 3) and the existing facilities. The area has been regraded to create a level driveway for a metal storage building. The entire area is clear, with the majority maintained as lawn. The storage building is accessed via gravel pavement.

A 12-inch diameter PVC sanitary sewer is located along the west side of the gravel drive between Borings 1 and 5 and bisects the site in a southwest to northeast fashion. The sewer is on the order of 11 feet below existing grades along this portion of the alignment.

7.0 SUBSURFACE PROFILE

Surficial materials at the site range from gravel pavements to stiff clayey fills or native topsoils. Test Borings 2 through 4 encountered between 2 and 12 inches of topsoil underlain immediately by stiff loessal deposits. Test Borings 1, 5 and 6 encountered 4.5, 7.0 and 5.1 feet of fill. The bulk of this fill is described as stiff to very stiff in consistency and is comprised of silty clay with some shale. The upper 2 feet of fill in Test Boring 6 is a crushed limestone, which is utilized as a pavement surrounding portions of the existing facility. Natural moisture content of the fills of Test Borings 1 and 6 are between 9.4 and 13.6 percent.

Underlying the fill (or directly beneath the topsoil) is typically a zone of stiff to very stiff loessal silty clay. These were windblown deposits following regional glacial advances. The natural moisture content of the loess from 0.2 to 1.5 feet in Test Boring 4 was found to be 16.3 percent.

Test Borings 2 and 4 encountered glacial soils. These materials are stiff to very stiff blends of the four (4) generally accepted soil types; clay, silt, sand and gravel as deposited by the advance of glaciers across the area. The glacial tills are often overconsolidated, as they were overridden by ice sheets following their deposition. Natural moisture contents of the glacial till are in the mid to upper teens. The soils classify CL according to the Unified Soil Classification System (USCS) with a liquid limit of 47 percent and a plasticity index of 20.

Test Borings 3, 4 and 6 encountered a zone of residual clay, which is a term utilized for soils which have weathered from the underlying bedrock, where the shale portion of the bedrock has weathered to a clay, yet may still posses bedding planes. The residual soils have limestone layers from the interbedded bedrock system. Natural moisture contents from the residual soils are in the mid-teens. The Atterberg limits of the residual clay ranged from 45 to 59 percent, with plasticity indices of 25 to 32, classifying this material CL to CH (USCS). An unconfined compressive strength test of the material immediately beneath this sample (5.5 to 6.0 feet) yielded an unconfined compressive strength of 5,770 pounds per square foot at a natural dry density of 112.7 pounds per cubic foot (pcf).

Samples of the silty clay and clay in Test Boring 2 were found to have unconfined compressive strengths of between 8,050 and 10,770 pounds per square foot (psf) at natural dry densities of 108.7 to 110.4 pounds per cubic foot (pcf). The sample from 6.2 to 6.7 feet was found to have a liquid limit of 53 percent and a plasticity index of 30, classifying CH according to the Unified Soil Classification System (USCS).

The residual soils increase in moisture content and decrease in stiffness immediately above the bedrock, with moisture contents between 32.4 and 38.1 percent.

All of the borings were extended a minimum 2 samples into bedrock, interbedded shale and limestone. The depths to the surface of the bedrock varied between 4.5 feet (Test Boring 1) and 9.5 feet (Test Borings 2, 3 and 5). The surface of the bedrock is described as severely or highly weathered, where the shale portion of the interbedded shale and limestone bedrock is brown and has nearly weathered to a clay. Natural moisture contents of the highly weathered shale portion of the bedrock were in the low to mid teens.

Underlying the highly weathered zones of bedrock is an olive brown weathered or slightly weathered shale and limestone bedrock. An Atterberg liquid limit of the weathered bedrock in Test Boring 3 indicates a liquid limit of 30 and a plasticity index of 13, classifying the material CL (USCS). The natural moisture content of the sample was 11.0 percent. The weathered zones are relatively thin, transitioning to gray, unweathered bedrock, interbedded shale and limestone, often within 1 or 2 sample thicknesses (2.5 to 5.0 feet) from the bedrock surface. Test Borings 3 and 6 were terminated within the slightly weathered bedrock zone.

The surface of the gray, unweathered bedrock, interbedded shale and limestone occurred at depths between 9.5 and 12.0 feet from the ground surface, assuming that Test Borings 3 and 6 were in close proximity to the gray shale. Natural moisture contents of the shale portion of the unweathered bedrock averaged between 6 and 8 percent. The shale is described as very weak to weak.

The limestone component of the bedrock is gray, crystalline, fossiliferous and often vuggy. The limestone remains unweathered relative to the shale. The limestone is consistently described as very strong.

Table 1 indicates the depth and elevation at which the surface of the highly weathered and unweathered bedrock, interbedded shale and limestone, was encountered in each boring. The table also indicates the depth at which split spoon refusal was encountered. Refusal is defined as requiring more than 50 blows of the sample hammer to advance the split spoon sampler less than 6 inches.

Table 1: Bedrock Depths and Elevations

		Highly Weathered Shale and Limestone (bedrock)		Unweathered (gray) Shale and Limestone (bedrock)		Refusal in bedrock (>50 blows/6")	
Boring	Ground Surface Elevation (ft.,MSL)	Depth to Top (ft.)	El. Top (ft., MSL)	Depth to Top (ft.)	El. Top (ft., MSL)	Depth (ft.)	El.(ft., MSL)
1	830.2	4.5	825.7	10.0	820.2	5.0	825.2
2	834.6	9.5	825.1	12.0	822.6	10.5	824.1
3	836.8	9.5	827.3	>11.4	<825.4	10.5	826.3
4	833.2	7.0	826.2	9.5	823.7	10.5	822.7
5	826.0	9.5	816.5	12.0	814.0	13.0	813.0
6	825.3	7.0	818.3	>11.0	<814.3	10.5	814.8

Test Boring 2 was extended into the unweathered bedrock by rock coring from 12.5 to 35.0 feet from the ground surface. The rock cores were logged to determine the thicknesses and percentages of shale and limestone as well as to determine the rock quality designation (RQD), which is defined by the percentage of intact pieces of the bedrock core which are 4 inches or longer. RQD's of the runs range from 16 percent to 92 percent, generally increasing with depth. The limestone beds comprised 33 to 51 percent of each sampling interval at thicknesses varying from ¼ inch to 14 inches. The 14 inch bed of limestone had very thin shale partings which remained intact during drilling and should be considered nearly consistent with the limestone in terms of strength for excavatability.

Groundwater readings were recorded by the Drilling Technician during drilling, at the completion of drilling, and prior to backfilling the test boring holes. These readings were

noted on the bases of the individual test boring logs. Groundwater was noted during the advancement of Test Borings 3, 4 and 6 as trace water at 7.0 feet and 9.0 feet in Test Borings 3 and 4 and as water trapped within the crushed limestone pavement at 0.2 feet in Test Boring 6. Test Borings 4 and 6 were dry at completion. Test Boring 3 contained trace groundwater at 11.0 feet, near the base of the boring.

Water was introduced to Test Boring 2 as a coolant used during the rock coring. Water remained at 1.5 feet from the ground surface at the completion of drilling, suggesting a limited amount of figures or voids which would drain the water from the hole.

Immediately prior to backfilling the test borings, final groundwater readings were obtained. With the exception of the core water in Test Boring 2 and water draining into the hole from the gravel pavement in Test Boring 6, all borings were dry.

8.0 CONCLUSIONS AND RECOMMENDATIONS

8.1 Comments and Limitations

Based upon our engineering reconnaissance of the site, the test borings, a visual examination of the samples, the laboratory tests, our understanding of the proposed construction and our experience as Consulting Soil and Foundation Engineers in Southwestern Ohio, we have reached the following conclusions and make the following recommendations.

The conclusions and recommendations of this report have been derived by relating the general principles of the discipline of Geotechnical Engineering to the proposed construction outlined by the Project Characteristics section of this report. Because changes in surface, subsurface, climatic and economic conditions can occur with time and location, we recommend for our mutual interest that the use of this report be restricted to this specific project.

Our understanding of the proposed design and construction is based on the documents provided to us at the time this report was prepared and which are referenced in the Project Characteristics, Section 3.0 of this report. We recommend that our office be retained to review the final design documents, plans and specifications, to assess any impact changes, additions or revisions in these documents may have on the conclusions and recommendations of this geotechnical report. Any changes or modifications which are made in the field during the construction phase which alter site grading, structure locations, infrastructure or other related site work should also be reviewed by our office prior to their implementation.

If conditions are encountered in the field during construction which vary from the facts of this report, we recommend that our office be contacted immediately to review the changed conditions in the field and make appropriate recommendations.

The scope of our services did not include any environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air at or around this site.

It is our understanding that the time frame for beginning and completing the foundation and site work for this project will be continuous, without interruption or delay. Should interruptions or delays occur, our office should be kept appraised to determine what recommendations must be modified accordingly.

We have performed the test borings and laboratory tests for our evaluation of the site conditions and for the formulation of the conclusions and recommendations of this report. We assume no responsibility for the interpretation or extrapolation of the data by others.

The earthwork and backfilling recommendations of this report presume that the earthwork will be monitored continuously by an Engineering Technician under the

direction of a Registered Professional Geotechnical Engineer. We recommend that Clermont County contract these services directly with Thelen Associates, Inc.

After you have had an opportunity to study this geotechnical report and to discuss its implications with the design team, we recommend that a meeting be held with the design team members to review the plans and specifications in light of the geotechnical report, paying particular attention to the possible implications of the geotechnical report with respect to potential construction problems and construction procedures which may be standard in the industry, but not consistent with our recommendations. This meeting should be held prior to submitting the contract documents in the market place for bidding.

It is our opinion that the site is suitable for the expansion of the lift station. The main impediment to the construction will be the potential need to cut significant portions of relatively tough bedrock if the decision is made to bury the proposed lift station tank.

Even if the at-grade, circular tank is used we anticipate that the excavations for the lift station will penetrate the relatively shallow fills and at least extend into the surface of the bedrock, interbedded shale and limestone. The following sections of this report contains specific recommendations pertaining to the design and construction of the proposed structure and related site development.

8.2 Bedrock Excavation

At the time this report was prepared, the design of the pump station had not been completed. The station may be sited to avoid extensive bedrock excavation, however, dependent upon other requirements of the design process, excavations may extend as much as 26 feet below existing grades. As noted in Table 1, bedrock, interbedded shale and limestone is as shallow as 4.5 feet from existing grade and was consistently encountered within 9.5 feet. The surface of the unweathered, gray shale and limestone bedrock was consistently located within 12 feet of existing grades. Split spoon refusal was

encountered in the bedrock as shallow as 5.0 feet from existing grade (Test Boring 1) and was typically at 10.5 feet from grade. The depth of split spoon refusal is often used as a contract delineator where excavation of soil and soft rock excavation transitions to a bedrock excavation pay item. If bedrock excavation is required and is specified as a separate unit price item in the contract documents, we recommend using the depths identified in the test borings for bedrock split spoon refusal as the depth at which the pay item would be enacted.

It has been our experience that bulk bedrock excavations into the weathered and highly weathered portions of the bedrock can be performed with conventional track-mounted excavating equipment. Excavation difficulty increases into the weathered and slightly weathered zones and further increases in gray, unweathered bedrock, interbedded shale and limestone. Excavations into these materials may require hydraulically driven hammers or rams and larger excavating equipment. The Contractors bidding the project should be familiar with the local bedrock and should review the results of the borings, specifically the rock cores performed in Test Boring 2.

8.3 Lateral Earth Pressures

The lift station may extend as much as 26 feet below surrounding grades. Below grade walls of the structure will have to be designed to resist lateral earth pressures based upon the depth of unbalanced fill between the interior and exterior of the tank. Lateral earth pressures should be based upon at-rest conditions unless the Structural Engineer feels that the walls are able to deflect sufficiently to allow for active pressures to initialize. The degree of displacement is largely dependent upon backfill materials being utilized, however, recognizing that the backfill will generally be well-compacted granular materials immediately adjacent to the structures, a translation of 0.001H would allow for the use of active earth pressure design. This would be as little as 0.31 inches on a 26 foot deep tank. If the tank is cylindrical, or if it has a permanent concrete roof, we anticipate that the Structural Engineer will consider at-rest earth pressures.

Unless the design of the structures can assure that water levels inside the structures will remain equivalent to exterior water levels, undrained backfill conditions should be utilized in the design.

The sidewalls of the tank should be designed based upon at-rest lateral earth pressures. For undrained conditions and assuming full saturation of backfill from periodic flooding of Poplar Creek, an equivalent weight (EFW_o) of 105 pounds per cubic foot (pcf) should be utilized. If a foundation level gravity drain is included in the backfill and the tank backfill remains in a drained condition during flooding, the EFW_o can be reduced to 75 pcf. If active earth pressures are acceptable based upon allowed deflections, these forces should be based upon an EFW_a at 90 pcf for undrained conditions and 45 pcf for drained conditions.

The lateral earth pressures should be applied based on a triangular distribution and should be supplemented if there are surcharges for traffic and or sloping backfill.

Much of the tank may extend into the bedrock profile, which will effectively transfer zero lateral load. If the overdig width can be controlled, the lateral pressures may be reduced by applying lateral pressure theories for narrow backfill zones. Many of these theories were developed for silos, where a relatively narrow column of soil provides a reduced lateral pressure. If requested, Thelen can provide additional services to evaluate appropriate reductions to the lateral loads and assist in preparing specifications to limit the overdigs to remain consistent with the assumptions.

8.4 Buoyancy

While the groundwater table during our field services was not encountered, if the area is prone to flooding from Poplar Creek, the backfill may become saturated and the proposed structures may be subject to buoyant (uplift) forces. Wastewater structures typically use pressure relief values (prvs) to avoid buoyancy considerations in the design and construction. Provided that pressure relief values are installed which will

quickly allow the tank to accept water to the interior at a rate consistent with groundwater fluctuations, design considerations to resist buoyant forces can be avoided.

8.5 Footings/Foundations

If the excavation for the base of the lift station tank will extend not extend below the stiff native soils, the footings or mat foundation should be proportioned based upon an allowable soil bearing pressure of 3,000 pounds per square foot (psf). We anticipate mat foundation loads to be significantly less. Excavations extended into the bedrock (below El. 816.5) would be capable of supporting allowable loads of 6,000 psf or more. Given these loading parameters and the effective unloading of the site with the bulk removal of soils, we anticipate that well prepared bearing subgrades will result in settlements of ½ inch or less.

Once excavation is completed, the exposed bearing materials should be reviewed by the Project Geotechnical Engineer or a representative thereof. The bearing surfaces should consist of stiff clayey materials or bedrock, interbedded shale and limestone. The bulk of loose or otherwise disturbed materials should be removed from the bearing surface and should be free of loose or frozen material prior to casting foundations.

It is expected that significant quantities of reinforcing steel will be required within the mat foundation/slab. We recommend that the excavation be extended slightly below design elevation and returned to design grade with a concrete "mud mat" immediately following the cut to protect the bearing materials and to provide a firm, stable surface upon which to work during the installation of reinforcing steel.

8.6 Shoring

It may be possible to lay back excavations along sides of the structure where existing facilities are not immediately adjacent. The extent and steepness of laid back excavations

should be consistent with standards of the Occupational Safety and Health Administration (OSHA). Preliminary assumptions can be drawn from the test boring logs for the allowable steepness of these excavations. The actual construction should then be monitored continuously by an OSHA "Competent Person" provided by the Contractor.

Shoring will likely be required to provide temporary support in areas where there are existing foundations, utilities, etc. within the excavation limits. If the shoring can be internally braced and dewatered, the shoring can be proportioned using an at-rest EFW_o of 75 pcf plus building and pavement loads. Non-braced (cantilevered) shoring can be designed based upon active earth pressures and an EFW_a of 45 pcf, plus surcharges.

The design, selection and construction of the shoring system should be the responsibility of the Contractor. The contract documents should require a submittal process which allows for the review of design calculations and drawings by the Project Structural Engineer and Project Geotechnical Engineer prior to construction. Passive resistances within the soils or bedrock below the foundation level are dependent upon the material type at the base of the cut. Assuming that the cuts will extend into the interbedded shale and limestone bedrock, the bedrock can be assumed to provide a lateral passive resistance of 5,000 psf beginning 3 feet below the base of the cut.

8.7 Backfill

The recommendations for earth pressures of the structure walls assume that the material surrounding the new structure will be well compacted. The backfill materials can consist of the clayey soils encountered within the upper 4.5 to 9.5 feet of the site. If the perimeter of the structure is to be drained, backfill within 2 feet of the walls of structures should consist of free-draining granular fill hydraulically connected to a sump or gravity flow drainage outlet. The granular drainage layer should be separated from clayey backfill with a filter fabric.

All backfill for the structure walls, pipe trenches or embankment fill should be placed in shallow, level lifts, 6 to 8 inches in thickness and compacted with appropriate equipment, such as a sheepsfoot roller for clayey soils or a vibratory compactor for granular materials. The backfill should be compacted to at least 95 percent of the maximum dry density per the standard Proctor moisture-density test, ASTM D698, or to 75 percent relative density per ASTM D4253 and D4254, whichever is more applicable based upon the amount of silt and clay (fines) in the backfill material.

Caution should be taken when compacting fill adjacent to the walls of the structures so that the walls do not become overstressed from the compaction equipment.

8.8 General Site

Under no circumstances should any concrete, pavement or fill be placed over frozen or saturated soils. In addition, frozen soil should not be used as compacted fill or backfill.

Because of the previous development in many portions of the site, the possibility remains that unanticipated encumbrances, such as additional buried structures, wells, etc. may be encountered at discrete areas throughout the project site. We recommend that the Project Geotechnical Engineer be contacted to review these areas.

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APPENDIX

ASFE Report Information

Tabulation of Laboratory Tests

Unconfined Compression Test Forms

Boring Plan, Drawing 120697NE-1

Test Boring Logs

Soil Classification Sheet

Photographs of Rock Cores

Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

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THELEN ASSOCIATES, INC.
Geotechnical • Testing Engineers
• 1780 Carillon Boulevard, Cincinnati, Ohio 45240-2795 / 513-825-4350 / Fax 513-825-4756
www.thelenassoc.com
Offices
Erlanger, Kentucky
Lexington, Kentucky
Cincinnati, Ohio
Dayton, Ohio

Clermont County
Water Resourced Department
Geotechnical Exploration
Bethel Lift Station Expansion
2950 S.R. 125
Clermont County, Ohio
120697NE

TABULATION OF LABORATORY TESTS

Boring No.	Sample No.	Depth (ft.)		Moisture Content (%)	Atterberg Limits (%)			Gradation Analysis (%)				Natural Dry Density, PCF	Unconfined Compressive Strength, PSF	USCS Classification
		From	To		LL	PL	PI	Gravel	Sand	Silt	Clay			
1	1	0.0	1.5	23.3										
	2	2.5	4.0	9.4										
	3	5.0	6.5	17.4										
	4	7.5	9.0	8.2										
	5	10.0	11.5	8.2										
2	1	0.0	0.3	21.4	47	27	20							CL
	2	0.3	1.5	15.9										
	PT-3	5.14	5.4	16.6								110.4	10,720	
	PT-3	6.2	6.7	19.9	53	23	30					108.47	8,050	CH
	4	7.0	8.5	38.1										
	5	10.0	10.8	11.0	30	17	13							CL
4	1B	0.2	1.5	16.3										
	2	2.5	4.0	17.1										
	3	5.0	6.5	32.4	59	27	32							CH
	4	7.5	9.0	20.4										
	5	10.0	11.5	6.2										
6	2	2.5	4.0	13.6										
	PT-3	5.3	5.5		45	22	23							CL
	PT-3	5.5	6.0	18.0								112.7	5,770	
	4	7.0	8.5	16.1										
	5	10.0	10.6	16.6										

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UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOIL, ASTM - D2166
UNIT WEIGHT AND NATURAL MOISTURE

CLIENT : **Clermont County, Water Resources Department**
PROJECT : **G.E., Bethel Lift Station Expansion, 2950 S.R. 125**
LOCATION : **Clermont County, Ohio**

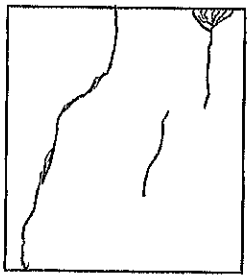
PROJECT NUMBER : **120697NE**
BORING NUMBER : **2** SAMPLE NUMBER : **PT-3** LAB NUMBER :
SAMPLE DESCRIPTION : **Brown moist very stiff SILTY CLAY, trace fine gravel** DEPTH (FT.): **5.1** to **5.4**

SAMPLE OBTAINED BY : **SHELBY TUBE** CONDITION: **UNDISTURBED** DATE : **09/26/12**

NATURAL UNIT WEIGHT

AVERAGE DIAMETER (in.)	2.84
HEIGHT (in.)	3.55
HEIGHT TO DIAMETER RATIO	1.25
AVERAGE AREA (sq. ft.)	0.0438
VOLUME (cu. ft.)	0.0130
WET WEIGHT (lbs.)	1.67
DRY WEIGHT (lbs.)	1.43
DRY DENSITY (pcf)	**110.4**

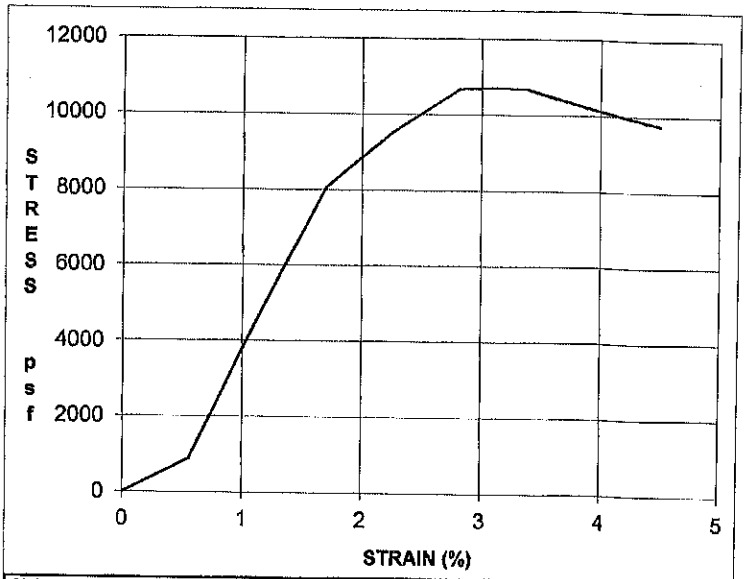
FAILURE SHAPE



WATER CONTENT AFTER SHEAR

CAN NUMBER	A2
WET WEIGHT + CAN (lbs.)	2.01
DRY WEIGHT + CAN (lbs.)	1.77
WEIGHT WATER (lbs.)	0.24
WEIGHT CAN (lbs.)	0.34
WEIGHT SOLID (lbs.)	1.43
MOISTURE (%)	**16.6**
LOAD CELL NUMBER	CELL

DEFORM DIAL .001 IN.	LOAD CELL	LOAD LBS	STRAIN %	CORR AREA SQ. FT.	STRESS PSF
0	0	0	0	0.0438	0
20	39.0	39.0	0.6	0.0441	884
40	205.0	205.0	1.1	0.0443	4622
60	359.0	359.0	1.7	0.0446	8049
80	427.0	427.0	2.3	0.0449	9518
100	482.0	482.0	2.8	0.0451	10683
110	485.0	485.0	3.1	0.0453	10718
120	485.0	485.0	3.4	0.0454	10687
140	464.0	464.0	3.9	0.0456	10164
160	448.0	448.0	4.5	0.0459	9756



AVERAGE RATE OF STRAIN TO FAILURE (% per minute)	**1.7**
STRAIN AT FAILURE (%)	**3.1**
UNCONFINED COMPRESSIVE STRENGTH (psf)	**10,720**
SHEAR STRENGTH (psf)	**5360**

REMARKS :

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• 1780 Carillon Boulevard, Cincinnati, Ohio 45240-2795 / 513-825-4350 / Fax 513-825-4756
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UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOIL, ASTM - D2166
UNIT WEIGHT AND NATURAL MOISTURE

CLIENT : **Clermont County, Water Resources Department**
PROJECT : **G.E., Bethel Lift Station Expansion, 2950 S.R. 125**
LOCATION : **Clermont County, Ohio**

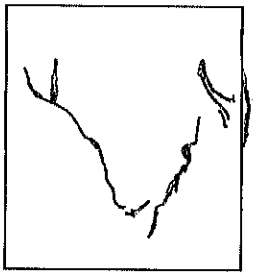
PROJECT NUMBER : **120697NE**
BORING NUMBER : **2** SAMPLE NUMBER : **PT-3** LAB NUMBER :
SAMPLE DESCRIPTION : **Brown moist very stiff CLAY, trace bedding planes (residual)** DEPTH (FT.): **6.2** to **6.7**

SAMPLE OBTAINED BY : **SHELBY TUBE** CONDITION: **UNDISTURBED** DATE : **09/26/12**

NATURAL UNIT WEIGHT

AVERAGE DIAMETER (in.)	2.87
HEIGHT (in.)	5.58
HEIGHT TO DIAMETER RATIO	1.94
AVERAGE AREA (sq. ft.)	0.0449
VOLUME (cu. ft.)	0.0209
WET WEIGHT (lbs.)	2.72
DRY WEIGHT (lbs.)	2.27
DRY DENSITY (pcf)	**108.7**

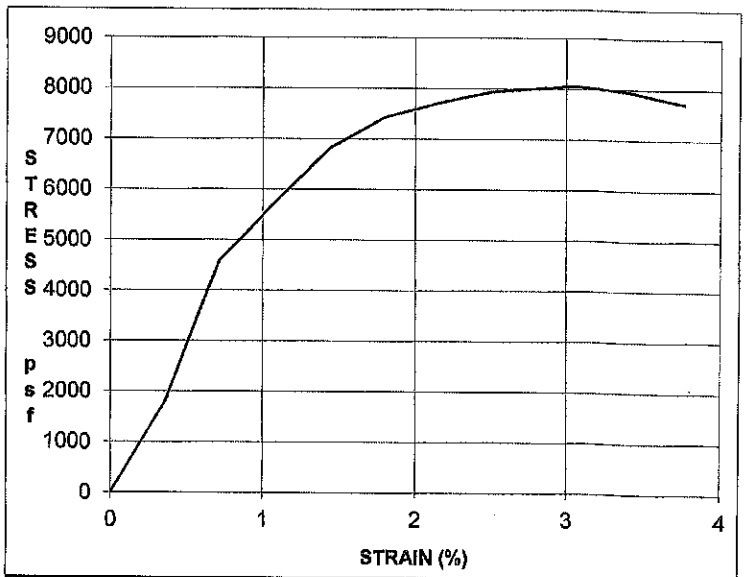
FAILURE SHAPE



WATER CONTENT AFTER SHEAR

CAN NUMBER	B3
WET WEIGHT + CAN (lbs.)	3.14
DRY WEIGHT + CAN (lbs.)	2.69
WEIGHT WATER (lbs.)	0.45
WEIGHT CAN (lbs.)	0.43
WEIGHT SOLID (lbs.)	2.27
MOISTURE (%)	**19.9**
LOAD CELL NUMBER	CELL

DEFORM DIAL .001 IN.	LOAD CELL	LOAD LBS.	STRAIN %	CORR. AREA SQ. FT.	STRESS PSF
0	0	0	0	0.0449	0
20	80.0	80.0	0.4	0.0451	1775
40	207.0	207.0	0.7	0.0452	4576
60	260.0	260.0	1.1	0.0454	5727
80	310.0	310.0	1.4	0.0456	6804
100	339.0	339.0	1.8	0.0457	7413
120	354.0	354.0	2.2	0.0459	7713
140	366.0	366.0	2.5	0.0461	7945
170	373.0	373.0	3.0	0.0463	8053
190	370.0	370.0	3.4	0.0465	7958
210	360.0	360.0	3.8	0.0467	7715



AVERAGE RATE OF STRAIN TO FAILURE (% per minute)	**1.1**
STRAIN AT FAILURE (%)	**3.0**
UNCONFINED COMPRESSIVE STRENGTH (psf)	**8,050**
SHEAR STRENGTH (psf)	**4025**

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UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOIL, ASTM - D2166
UNIT WEIGHT AND NATURAL MOISTURE

CLIENT : **Clermont County, Water Resources Department**
PROJECT : **G.E., Bethel Lift Station Expansion, 2950 S.R. 125**
LOCATION : **Clermont County, Ohio**

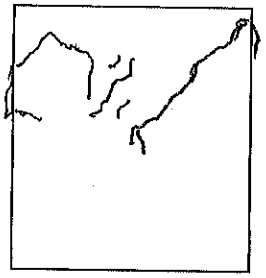
PROJECT NUMBER : **120697NE** LAB NUMBER :
BORING NUMBER : **6** SAMPLE NUMBER : **PT-3** DEPTH (FT.): **5.5** to **6.0**
SAMPLE DESCRIPTION : **Brown moist very stiff SILTY CLAY, trace bedding planes and limestone floaters (residual)**

SAMPLE OBTAINED BY : **SHELBY TUBE** CONDITION: **UNDISTURBED** DATE : **09/27/12**

NATURAL UNIT WEIGHT

AVERAGE DIAMETER (in.)	2.85
HEIGHT (in.)	5.58
HEIGHT TO DIAMETER RATIO	1.95
AVERAGE AREA (sq. ft.)	0.0444
VOLUME (cu. ft.)	0.0207
WET WEIGHT (lbs.)	2.75
DRY WEIGHT (lbs.)	2.33
DRY DENSITY (pcf)	**112.7**

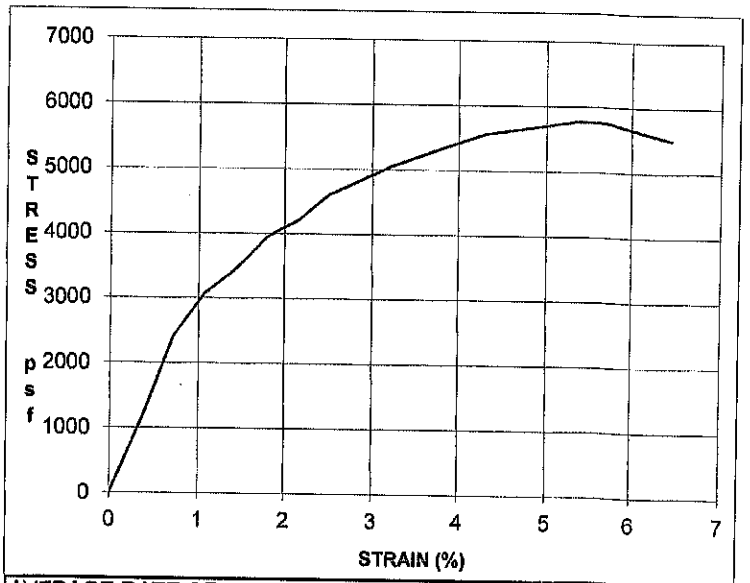
FAILURE SHAPE



WATER CONTENT AFTER SHEAR

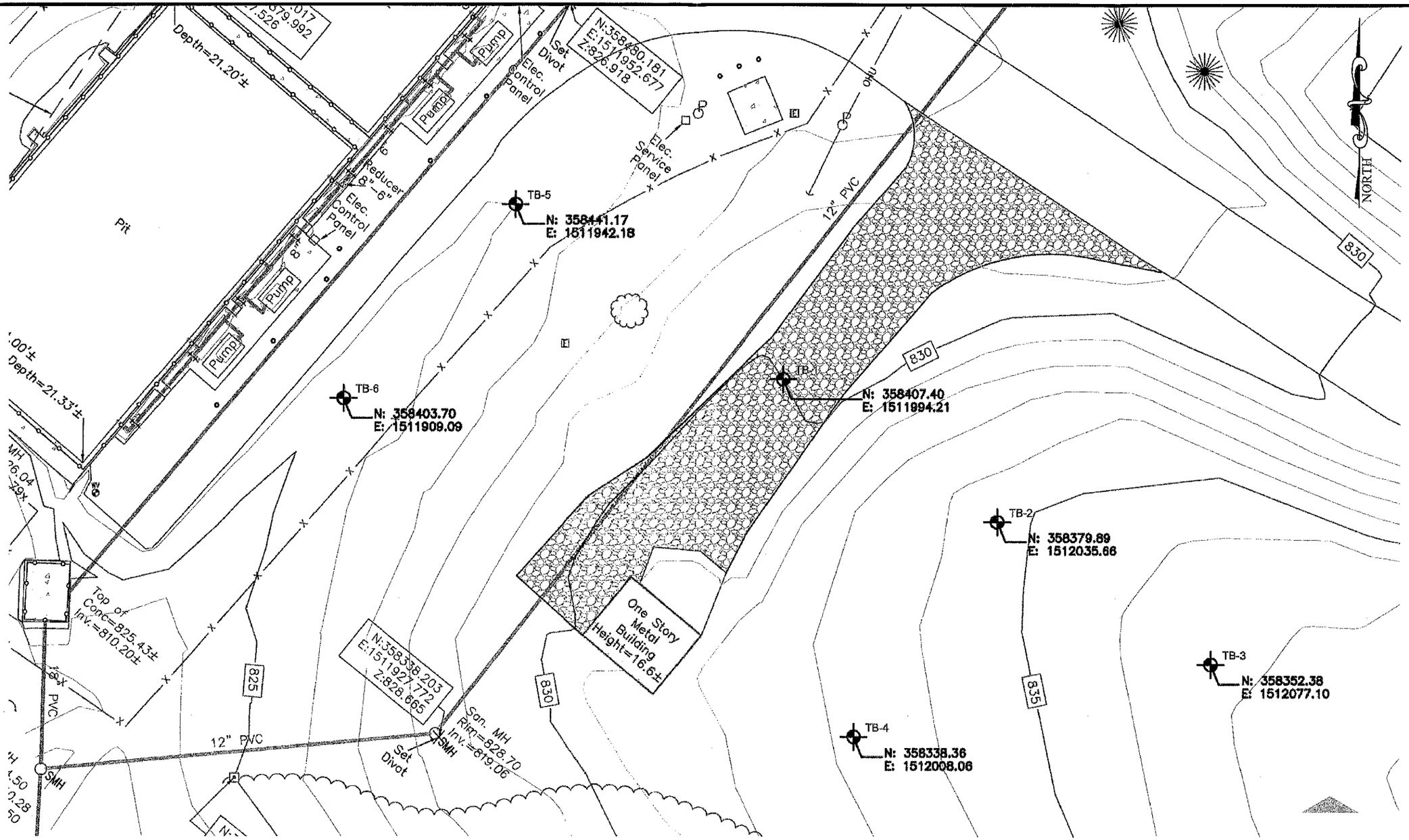
CAN NUMBER	T1
WET WEIGHT + CAN (lbs.)	3.17
DRY WEIGHT + CAN (lbs.)	2.76
WEIGHT WATER (lbs.)	0.42
WEIGHT CAN (lbs.)	0.43
WEIGHT SOLID (lbs.)	2.32
MOISTURE (%)	**18.0**
LOAD CELL NUMBER	CELL

DEFORM DIAL .001 IN.	LOAD CELL	LOAD LBS.	STRAIN %	CORR. AREA SQ. FT.	STRESS PSF
0	0	0	0	0.0444	0
20	51.0	51.0	0.4	0.0446	1144
40	108.0	108.0	0.7	0.0447	2415
60	138.0	138.0	1.1	0.0449	3074
80	155.0	155.0	1.4	0.0451	3440
100	178.0	178.0	1.8	0.0452	3936
120	191.0	191.0	2.2	0.0454	4208
140	210.0	210.0	2.5	0.0456	4610
180	232.0	232.0	3.2	0.0459	5056
240	258.0	258.0	4.3	0.0464	5560
300	271.0	271.0	5.4	0.0469	5774
320	271.0	271.0	5.7	0.0471	5752
360	261.0	261.0	6.5	0.0475	5498



AVERAGE RATE OF STRAIN TO FAILURE (% per minute)	**1.1**
STRAIN AT FAILURE (%)	**5.4**
UNCONFINED COMPRESSIVE STRENGTH (psf)	**5,770**
SHEAR STRENGTH (psf)	**2885**

REMARKS :



Base Drawing provided by Fishbeck, Thompson, Carr & Huber, Inc.

INDICATES TEST BORING LOCATION

Notes

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Lexington, Kentucky • Erlanger, Kentucky • Dayton, Ohio

Title:	BORING PLAN	Project:	Geotechnical Exploration, Bethel Lift Station Expansion	Scale:	NTS
Client:	Clermont County Water Resources Department	Location:	Clermont County, Ohio	Date:	9/14/12
				Drawing No.:	120697NE-1

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LOG OF TEST BORING

CLIENT: County of Clermont, Water Resources Department
PROJECT: Geotechnical Exploration, Bethel Lift Station Expansion
2950 S.R. 125, Clermont County, OH
LOCATION OF BORING: As shown on Boring Plan, Drawing 120697NE-1

BORING #: 1
PROJECT #: 120697NE
PAGE #: 1 of 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
830.2	Ground Surface	0.0	0						
828.2	Mixed brown and gray moist very stiff FILL, silty clay, trace sand, gravel and shale.	2.0		I	1	DS	17-17-9	10	56
825.7	Mixed brown moist stiff FILL, silty clay with sand and limestone fragments.	4.5		I	2	DS	5-9-50/6"	10	56
823.2	Interbedded brown moist very weak highly weathered SHALE and gray very strong unweathered LIMESTONE (bedrock).	7.0		I	3	DS	5-50/6"	8	67
820.2	Interbedded gray, trace olive brown moist very weak slightly weathered SHALE and very strong unweathered LIMESTONE (bedrock).	10.0		I	4	DS	9-50/4"	12	100
819.3	Interbedded gray moist very weak SHALE and gray very strong unweathered LIMESTONE (bedrock).	10.9	10	I	5	DS	18-50/4"	12	100
	Bottom of test boring at 10.9 feet.		20 30 40						

Datum: Mean Sea Level
Surface Elevation: 830.2 ft.
Date Started: 9/4/2012
Date Completed: 9/4/2012
Hammer Weight: 140 lb.
Hammer Drop: 30 in.
Pipe Size: 2 in. O.D.
Hole Diameter: 6 in.
Rock Core Diameter:
Boring Method: 2 1/4
Drill Rig: TD-1
Foreman: ML
Engineer: C. Hamant

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: 24 Hrs., Dry
Backfilled: 24 Hrs.

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

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LOG OF TEST BORING

CLIENT: County of Clermont, Water Resources Department
BORING #: 2
PROJECT: Geotechnical Exploration, Bethel Lift Station Expansion
2950 S.R. 125, Clermont County, OH
PROJECT #: 120697NE
PAGE #: 1 of 1
LOCATION OF BORING: As shown on Boring Plan, Drawing 120697NE-1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (In.)	Recovery (%)
834.6	Ground Surface	0.0	0						
834.3	TOPSOIL	0.3		I	1	DS	3-3-5	18	100
832.6	Brown moist very stiff SILTY CLAY, trace hairlike roots.	2.0							
	Brown slightly moist very stiff SILTY CLAY, trace sand and fine gravel (CL).			I	2	DS	7-11-10	15	83
828.6		6.0							
827.6	Brown moist very stiff CLAY, trace bedding plans (residual (CH).	7.0		U	3	PS		22	92
	Brown moist medium stiff SILTY CLAY with iron oxide concretions.			I	4	DS	5-7-7	18	100
825.1		9.5	10	I	5	DS	30-50/3"	100	100
822.1	Interbedded gray, trace brown moist very weak slightly weathered SHALE and gray very strong unweathered limestone (bedrock).	12.5							
819.1	Interbedded gray moist very weak unweathered SHALE and gray, crystalline, fosilliferous, occasionally vuggy very strong unweathered LIMESTONE. Shale beds are 1/2 inch to 12-1/2 inches thick and comprise 81% of this interval, assuming that lost core is shale. Limestone beds are 1/8 inch to 2-1/2 inches in thickness and comprise 19% of this interval.	15.5		I	6	RC	16%	30	83
				I	7	RC	36%	60	100
814.1	Interbedded gray moist weak to slightly strong unweathered SHALE and gray, crystalline, fosilliferous, occasionally vuggy very strong unweathered LIMESTONE. Shale beds are 1/4 inch to 5 inches thick and comprise 67% of this interval. Limestone beds are 1/4 inch to 2 -1/2 inches in thickness and comprise 33% of this interval.	20.5	20						
				I	8	RC	72%	60	100
809.1	Interbedded gray moist very weak to slightly strong unweathered SHALE and gray, crystalline, fosilliferous, occasionally vuggy very strong unweathered LIMESTONE. Shale beds are 1/2 inch to 14 inches thick and comprise 49% of this interval. Limestone beds are 2 inches to 1.4 inches in thickness (with thin calcareous shale partings) and comprise 51% of this interval.	25.5		I	9	RC	83%	60	100
804.1	Interbedded gray moist very weak to slightly strong unweathered SHALE and gray, crystalline, fosilliferous, occasionally vuggy very strong unweathered LIMESTONE. Shale beds are 1/2 inch to 6-1/2 inches thick and comprise 49% of this interval. Limestone beds are 1/2 inch to 6-3/4 inches in thickness (with thin calcareous shale partings) and comprise 51% of this interval.	30.5	30	I	10	RC	92%	54	100
799.6		35.0							
	Interbedded gray moist weak to slightly strong unweathered SHALE and gray, crystalline, fosilliferous, occasionally vuggy very strong unweathered LIMESTONE. Shale beds are 1/2 inch to 4-1/2 inches thick and comprise 66% of this interval. Limestone beds are 1/2 inch to 9-1/2 inches in thickness (with thin calcareous shale partings) and comprise 34% of this interval.		40						
	Split spoon refusal and bottom of test boring at 35.0 feet.								

Datum: Mean Sea Level
Hammer Weight: 140 lb.
Hole Diameter: 6 in.
Drill Rig: TD-1
Surface Elevation: 834.6 ft.
Hammer Drop: 30 in.
Rock Core Diameter: 1.875 in.
Foreman: ML
Date Started: 9/5/2012
Pipe Size: 2 in. O.D.
Boring Method: 2 1/4 HSA
Engineer: C. Hamant
Date Completed: 9/5/2012

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: Core Water Only
At Completion: 1.5
After:
Backfilled: Immediately

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

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LOG OF TEST BORING

CLIENT: County of Clermont, Water Resources Department
PROJECT: Geotechnical Exploration, Bethel Lift Station Expansion
2950 S.R. 125, Clermont County, OH
LOCATION OF BORING: As shown on Boring Plan, Drawing 120697NE-1

BORING #: 3
PROJECT #: 120697NE
PAGE #: 1 of 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (In.)	Recovery (%)
836.8	Ground Surface	0.0	0						
835.8	TOPSOIL	1.0		I	1A 1B	DS	2-5-6	18	100
834.8	Mottled brown, slightly moist very stiff SILTY CLAY, trace fine sand.	2.0							
832.3	Mottled light brown moist stiff SILTY CLAY, trace fine sand and iron oxide stains.	4.5		I	2	DS	6-8-5	18	100
829.8	Brown moist very stiff, CLAY, trace bedding planes (residual).	7.0		I	3	DS	4-5-8	18	100
827.3	Brown moist stiff SILTY CLAY, trace bedding planes, iron oxide stains and soft clay layers (residual).	9.5		I	4	DS	4-4-14	18	100
825.9	Interbedded brown and gray moist very weak severely weathered SHALE and gray very strong unweathered LIMESTONE (bedrock).	10.9	10	I	5	DS	8-10-50/5	17	94
825.4	Interbedded gray, trace brown moist very weak slightly weathered SHALE and gray very strong unweathered LIMESTONE (bedrock).	11.4							
	Split spoon refusal and bottom of test boring at 11.4 feet.		20, 30, 40						

Datum: Mean Sea Level	Hammer Weight: 140 lb.	Hole Diameter: 6 in.	Drill Rig: TD-1
Surface Elevation: 836.8 ft.	Hammer Drop: 30 in.	Rock Core Diameter: 1.875 in.	Foreman: ML
Date Started: 9/11/2012	Pipe Size: 2 in. O.D.	Boring Method: 2 1/4	Engineer: C. Hamant
Date Completed: 9/11/2012			

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH

First Noted	7.0 ft., Trace
At Completion	Trace at 11.0 ft.
After	22 Hrs., Dry
Backfilled	22 Hrs.

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

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LOG OF TEST BORING

CLIENT: County of Clermont, Water Resources Department
BORING #: 4
PROJECT: Geotechnical Exploration, Bethel Lift Station Expansion
2950 S.R. 125, Clermont County, OH
PROJECT #: 120697NE
PAGE #: 1 of 1
LOCATION OF BORING: As shown on Boring Plan, Drawing 120697NE-1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
833.2	Ground Surface	0.0	0						
833.0	TOPSOIL	0.2							
831.2	Brown moist stiff to very stiff SILTY CLAY, trace fine sand and hairlike roots.	2.0		I	1A 1B	DS	2-2-5	18	100
828.7	Brown moist very stiff sandy SILTY CLAY, trace fine gravel and iron oxide stains (glacial till).	4.5		I	2	DS	5-6-11	18	100
826.2	Brown moist very stiff CLAY, trace bedding planes and iron oxide stains (residual) (CH).	7.0		I	3	DS	4-6-8	18	100
823.7	Interbedded olive brown and gray, trace brown moist very weak moderately weathered SHALE and gray very strong unweathered LIMESTONE (bedrock).	9.5		I	4	DS	3-1-1	6	33
822.3	Interbedded gray moist very weak unweathered SHALE and gray very strong unweathered LIMESTONE (bedrock).	10.9	10	I	5	DS	39-50/3	9	83
	Split spoon refusal and bottom of test boring at 10.9 feet.		20						
			30						
			40						

Datum: Mean Sea Level
Hammer Weight: 140 lb.
Hole Diameter: 6 in.
Drill Rig: TD-1
Surface Elevation: 833.2 ft.
Hammer Drop: 30 in.
Rock Core Diameter: 1.875 in.
Foreman: ML
Date Started: 9/4/2012
Pipe Size: 2 in. O.D.
Boring Method: 2 1/4
Engineer: C. Hamant
Date Completed: 9/4/2012

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted 9.0, Trace
At Completion Dry
After 24 Hrs., Dry
Backfilled 24 Hrs.

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

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LOG OF TEST BORING

CLIENT: County of Clermont, Water Resources Department
PROJECT: Geotechnical Exploration, Bethel Lift Station Expansion
2950 S.R. 125, Clermont County, OH
LOCATION OF BORING: As shown on Boring Plan, Drawing 120697NE-1

BORING #: 5
PROJECT #: 120697NE
PAGE #: 1 of 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
826.0	Ground Surface	0.0	0						
824.0	Mixed brown and dark brown moist soft FILL, silty clay and topsoil, trace crushed limestone.	2.0		I	1	DS	3-8-12	6	33
	Mixed gray, trace brown moist stiff FILL, silty clay and shale, trace limestone fragments.			I	2	DS	5-6-7	10	56
819.0		7.0		I	3	DS	5-6-10	15	83
816.5	Brown moist medium stiff to stiff SILTY CLAY, trace iron oxide stains.	9.5		I	4	DS	17-9-7	15	83
814.0	Interbedded brown, trace olive brown and gray moist very weak severely weathered SHALE and gray very strong unweathered LIMESTONE (bedrock).	12.0	10	I	5	DS	5-4-54	6	33
812.7	Interbedded gray moist very weak unweathered SHALE and gray very strong unweathered LIMESTONE (bedrock).	13.3		I	6	DS	16-50/4"	10	100
	Split spoon refusal and bottom of test boring at 13.3 feet.		20						
			30						
			40						

Datum: Mean Sea Level
Surface Elevation: 826.0 ft.
Date Started: 9/4/2012
Date Completed: 9/4/2012
Hammer Weight: 140 lb.
Hammer Drop: 30 in.
Pipe Size: 2 in. O.D.
Hole Diameter: 6 in.
Rock Core Diameter: 1.875 in.
Boring Method: 2 1/4
Drill Rig: TD-1
Foreman: ML
Engineer: C. Hamant

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: 3 Hrs., Dry
Backfilled: 3 Hrs.

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

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LOG OF TEST BORING

CLIENT: County of Clermont, Water Resources Department
PROJECT: Geotechnical Exploration, Bethel Lift Station Expansion
2950 S.R. 125, Clermont County, OH
LOCATION OF BORING: As shown on Boring Plan, Drawing 120697NE-1

BORING #: 6
PROJECT #: 120697NE
PAGE #: 1 of 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
825.3	Ground Surface	0.0	0						
823.3	Dense FILL, crushed limestone.	2.0		D	1	DS	5-31-16	1	6
820.2	Brown moist very stiff FILL, silty clay, trace sand, gravel and iron oxide stains.	5.1		I	2	DS	7-7-8	10	56
818.3	Brown moist very stiff SILTY CLAY, trace bedding planes and limestone floaters (residual) (CL).	7.0		U	3	PT		24	100
815.8	Interbedded olive brown and brown moist very weak highly weathered SHALE and gray very strong unweathered LIMESTONE (bedrock).	9.5		I	4	DS	13-36-20	15	83
814.3	Interbedded gray, trace brown moist very weak slightly weathered SHALE and gray very strong unweathered LIMESTONE (bedrock).	11.0	10	I	5	DS	18-50/6"	8	67
	Split spoon refusal and bottom of test boring at 11.0 feet.		20						
			30						
			40						

Datum: Mean Sea Level
Surface Elevation: 825.3 ft.
Date Started: 9/4/2012
Date Completed: 9/4/2012

Hammer Weight: 140 lb.
Hammer Drop: 30 in.
Pipe Size: 2 in. O.D.

Hole Diameter: 6 in.
Rock Core Diameter: 1.875 in.
Boring Method: 2 1/4

Drill Rig: TD-1
Foreman: ML
Engineer: C. Hamant

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted 0.2
At Completion Dry
After 3.5 Hrs., 7.1 ft.
Backfilled 3.5 Hrs.

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

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SOIL CLASSIFICATION SHEET

NON COHESIVE SOILS
(Silt, Sand, Gravel and Combinations)

Density

Very Loose	- 5 blows/ft. or less
Loose	- 6 to 10 blows/ft.
Medium Dense	- 11 to 30 blows/ft.
Dense	- 31 to 50 blows/ft.
Very Dense	- 51 blows/ft. or more

Relative Properties

Descriptive Term	Percent
Trace	1 – 10
Little	11 – 20
Some	21 – 35
And	36 – 50

Particle Size Identification

Boulders	- 8 inch diameter or more	
Cobbles	- 3 to 8 inch diameter	
Gravel	- Coarse	- 3/4 to 3 inches
	- Fine	- 3/16 to 3/4 inches
Sand	- Coarse	- 2mm to 5mm (dia. of pencil lead)
	- Medium	- 0.45mm to 2mm (dia. of broom straw)
	- Fine	- 0.075mm to 0.45mm (dia. of human hair)
Silt		- 0.005mm to 0.075mm (Cannot see particles)

COHESIVE SOILS
(Clay, Silt and Combinations)

Consistency	Field Identification	Unconfined Compressive Strength (tons/sq. ft.)
Very Soft	Easily penetrated several inches by fist	Less than 0.25
Soft	Easily penetrated several inches by thumb	0.25 – 0.5
Medium Stiff	Can be penetrated several inches by thumb with moderate effort	0.5 – 1.0
Stiff	Readily indented by thumb but penetrated only with great effort	1.0 – 2.0
Very Stiff	Readily indented by thumbnail	2.0 – 4.0
Hard	Indented with difficulty by thumbnail	Over 4.0

Classification on logs are made by visual inspection.

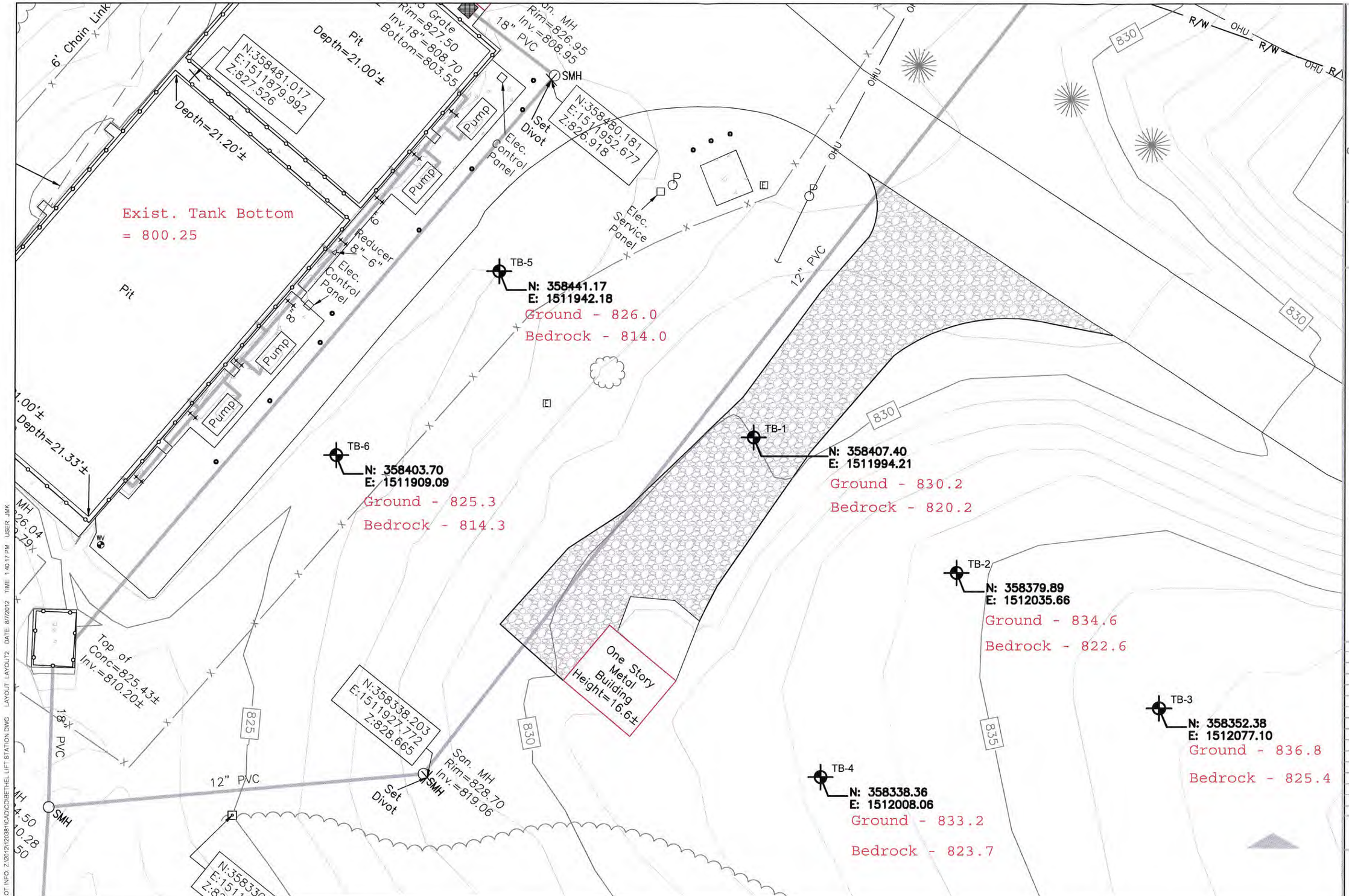
Standard Penetration Test – Driving a 2.0" O.D., 1 3/8" I.D., sampler a distance of 1.0 foot into undisturbed soil with a 140 pound hammer free falling a distance of 30 inches. It is customary to drive the spoon 6 inches to seat into undisturbed soil, then perform the test. The number of hammer blows for seating the spoon and making the tests are recorded for each 6 inches of penetration on the drill log (Example – 6/8/9). The standard penetration test results can be obtained by adding the last two figures (i.e. 8+9=17 blows/ft.). Refusal is defined as greater than 50 blows for 6 inches or less penetration.

Strata Changes – In the column "Soil Descriptions" on the drill log, the horizontal lines represent strata changes. A solid line (————) represents an actually observed change; a dashed line (— — — —) represents an estimated change.

Groundwater observations were made at the times indicated. Porosity of soil strata, weather conditions, site topography, etc., may cause changes in the water levels indicated on the logs.

CLERMONT CO.
WATER RESOURCES
BORING 2
12.5'-25.5'
120697N

PROJECT
Clermont Co
NO.
Clermont Co
Water Resources
Bethel Lift Station
Boring 2
25.5' - 35.0'
120697NE



fr&h
engineers
scientists
architects
constructors

Hard copy is intended to be 11"x17" when plotted. Scale(s) indicated and graphic quality may not be accurate for any other size.

Clermont County
Bethel, Ohio

Bethel Lift Station Storage Project

PROJECT NO.
G120381

FIGURE NO.
1

PLOT INFO: Z:\2012\120381\CAD\CD\BETHEL LIFT STATION.DWG LAYOUT: LAYOUT2 DATE: 8/7/2012 TIME: 1:40:17 PM USER: JMK