EXHIBIT A

Storm Sewer Evaluation Report

City of Parma Heights Storm Modeling

April 9, 2018

AECOM Project No. 60554795

Prepared for

CITY OF PARMA HEIGHTS

AECOM

1300 E. 9th St. Suite 500
Cleveland, OH 44115

Figures

Tables

Appendix

Acronyms and Abbreviations

ARI	Annual Return Interval
A	Area
City	City of Parma Heights
CSO	Combined Sewer Overflow
CHI	Computations Hydraulic International
CP	Concrete Pipe
HGL	Hydraulic Grade Line
LF	Linear Feet
MGD or mgd	Million Gallons per Day
NEORSD	Northeast Ohio Regional Sewer District
PCSWMM	Personal Computer Storm Water Management Model
Q	Flow
SRTC	Sensitivity-Based Ratio Tuning Calibration
V	Volume
WWF	Wet Weather Flow

1.0 EXECUTIVE SUMMARY

This report documents and summarizes the storm sewer hydraulic modeling study conducted in the City of Parma Heights, Ohio, south-west system. Thirteen (13) flow meters and two (2) rain gauges were deployed during the period of August 23, 2017 to November 23, 2017.

The purpose of the flow monitoring was to document flows in the storm sewer system during wet weather periods. The flow and rainfall data was used in calibrating the project model AECOM built as part of this study. Once calibrated, the sewer model was used to analyze the existing sewer system's performance under wet weather conditions, and alternatives for improvement.

Section 2 of this report gives a brief introduction and discusses the current issues in the system.

Section 3 of this report discusses the placement of rain gauges and flow meters throughout the system.

Section 4 of this report describes various hydraulic model attributes and the development process.

Section 5 of this report describes model standards and the calibration process.

Section 6 of this report reviews baseline assumptions and concerns concerning the hydraulic model.

Section 7 of this report presents a hydraulic capacity and flooding analysis for the design storms and two historical events.

Section 8 of this report presents AECOMs recommendations.

Section 9 is the report conclusion.

2.0 INTRODUCTION

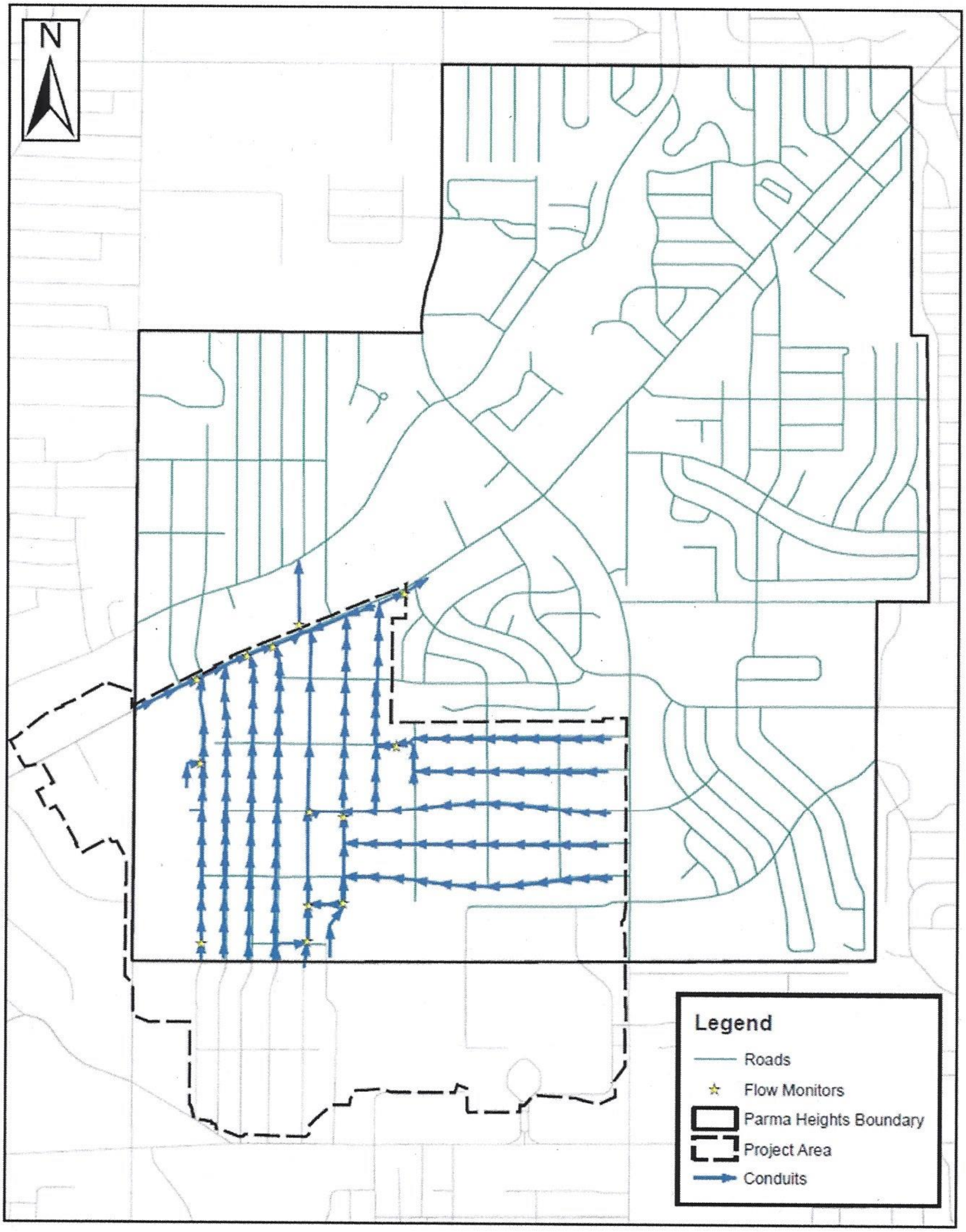
The City of Parma Heights has experienced significant storm sewer and overland flow flooding over the past several years. This caused extensive damage to private property including basement flooding and other surface flooding in the area south of Pearl Road and west of York Road. The City Engineer (Neff and Associates) met with engineers from Cuyahoga County Department of Public Works and AECOM in August 2017 to discuss a plan of action to better understand the flooding the causes of flooding and quantify the types of storms that activate flooding issues.

AECOM proposed conducting a storm sewer system flow monitoring and modeling project in the area with flooding complaints. The storm sewer monitoring would quantify the flooding and provide data that could be used in a hydraulic model. The flow monitoring project data is provided in a separate report. This report summarized the hydraulic model.

2.1 PURPOSE

The purpose of this report is to summarize the findings of the storm sewer modeling. This report documents how the model was developed, how flow data was utilized to calibrate the model and provide simulations for various storms events. Recommendations for system improvements to alleviate flooding are also provided.

Figure 2-1: Project Location



3.0 DATA COLLECTION AND FIELD INVESTIGATION

After studying the areas existing sewer system and performing field reconnaissance, AECOM discussed locations for flow meters and rain gauges with the City. These locations focused on major pipe networks and outside tributary areas. Deployment of flow meters and rain gauges began in the middle of August 2017. A total of 13 flow meters and 2 rain gauges were installed throughout the project area. The locations of these metering instruments along with the meter catchments are depicted on Figure 3-1.

Figure 3-1: Flow Monitoring and Rain Gauge Site Locations, August 2017

3.1 FLOW MONITORING

There were 13 flow meters installed at different locations throughout the project area. Existing sewer system maps were used to select the installation locations of the flow meters. The objectives in selecting meter locations included isolation of the study area from upstream flows, general system flow characterization and understanding and detection of flow anomalies in the collection system. The overall tributary area was divided into relatively equal sub-catchments. Field reconnaissance was conducted to determine the suitability of the selected manholes for the installations based on manhole accessibility, pipe conditions and hydraulic conditions, such as slopes, and bends Flow monitors were calibrated at each location, both during installation and periodically during the flow monitoring period, to facilitate the collection of accurate data. The final flow monitoring locations selected met the goals of the project and the practical limitations of the flow monitoring equipment. Damage to the equipment caused by debris or vandalism can compromise flow data. Therefore, the monitor installations were inspected periodically. The installed flow meters recorded depth and velocity information in 5-minute intervals. This data was used to assess the sewer characteristics under both dry and wet weather conditions. Since these are storm sewer pipes, dry weather flow was assumed to be zero.

Flow meters FM 6, FM 10, FM 11, and FM 12 monitored inflows from areas located outside the project boundary. The overland flow from Nathan Hale Park was measured by meters FM13 (main flow), and FM9 (overflow). Meters FM 1 and FM 2 were located on the two system outlets. Meter FM3 was located upstream of FM 2. Table 3-1 and Figure 3-2 show the meter characteristics and schematic.

3.2 FIELD RESULTS

The flow meters used depth and velocity data to calculate flow. Flow depth is converted to a cross sectional area (A) based on the size and geometry of the pipe. The velocity sensor of the meter measures the velocity (V) of the flow which is used to calculate the total flow (Q) from the equation $Q = A \times V$. Hydrographs were developed from the meter data and flow characteristics were analyzed. Meter data quality was very acceptable overall. Main issues with metering data are caused from debris in the pipe or a problematic velocity sensor. Table 3-1 shows the quality of data observed for each meter.

Data was identified as questionable due to flow behavior that could not be explained accounting for rainfall and site conditions. Table 3-1 details periods of questionable data for each of the wet-weather events and describes which parameter was "questionable." All questionable data was due to velocity dropping out intermittently during the event. Data was missing at FM3 during two storm events, and the first half of a third one in October, due to meter malfunction. For these events, FM2 data was used to calibrate the entire FM2 and FM3 catchment. The table also shows that pipe surcharge was only observed at meters FM5 and FM6, during the flow monitoring period, and for one event only, on October 8, 2017. This event was comparable to a 1 to 2-year design storm.

Table 3-1: Data Quality Summary

Data Quality not in Question
Missing Data
Questionable Velocity Data
Questionable Depth Data
Questionable Depth/Velocity Data
Pipe Surcharged
No Flow

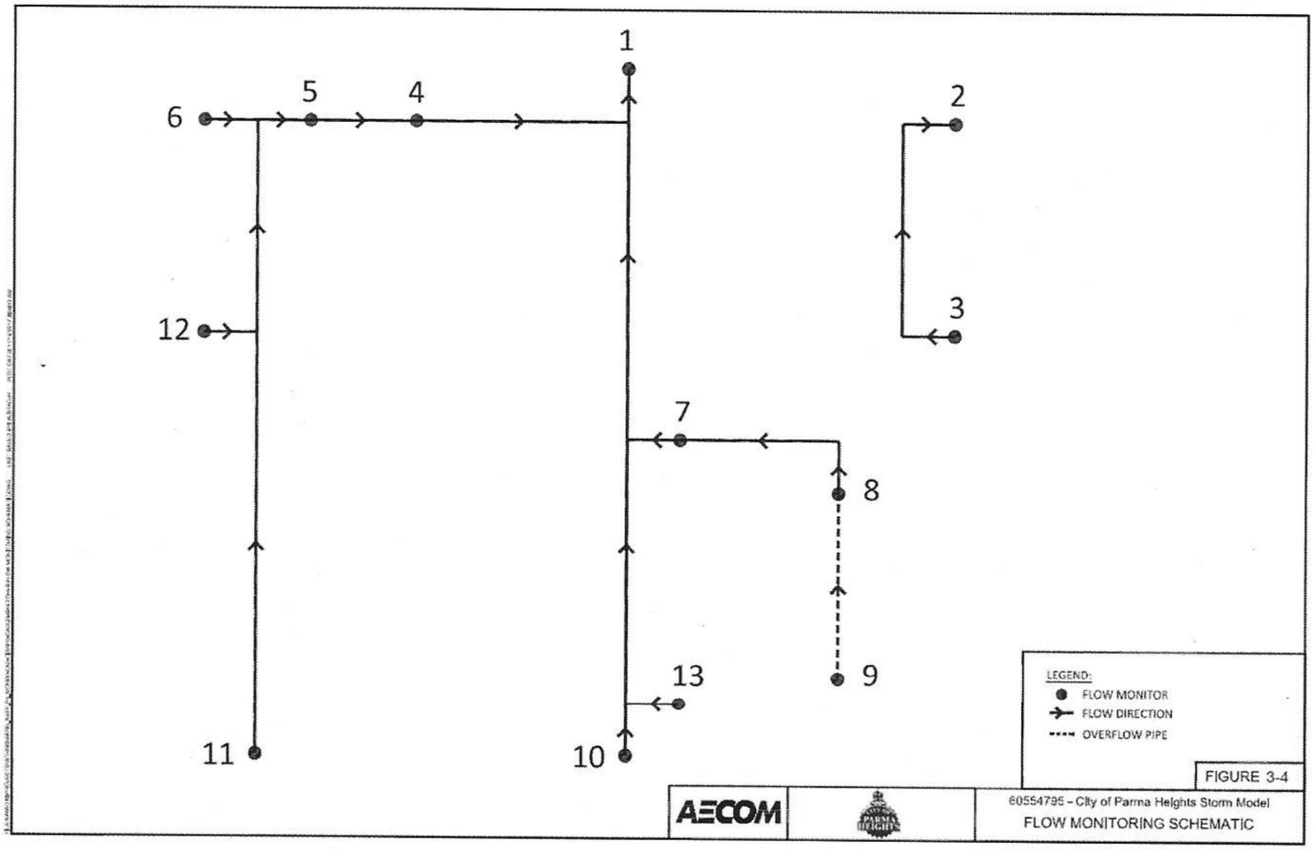
Start Date	Storm Duration (hr)	RainTotal (in)	Peak hour Intensity (in/hr)	FM-1	FM-2	FM-3	FM-4	FM-5	FM-6	FM-7	FM-8	FM-9	FM-10	FM-11	FM-12	FM-13
9/4/17 9:40 AM	29.6	0.42	0.21													
9/6/17 5:00 PM	1.3	0.17	0.17													
10/6/17 5:15 AM	9.1	0.26	0.14													
10/8/17 1:10 AM	34.5	2.17	1.10													
10/11/17 4:50 AM	11.7	0.19	0.10													
11/1/17 9:05 AM	20.0	0.89	0.38													
11/3/17 1:30 AM	6.1	0.78	0.49													
11/5/17 3:50 AM	27.5	0.99	0.55													

Table 3-2: Flow Meter Locations for Storm Sewer Study

2017 Parma Heights Flow Monitoring Sites

Checked	Site #	MH Number	Street/Location	Meter SN #	Pipe Size	US Pipe	DS Pipe	Installed	Removed	Comments:
	1	-	Int of Beverly and Pearl	2691	130" x 62"	130" x 62"	130" x 62"	8/23/2017	11/6/2017	
	2	-	6475 Pearl	2685	48"	48"	54"	8/23/2017	11/15/2017	
	3	-	11967 Blossum	2694	42"	42"	45"	8/22/2017	11/15/2017	No reaction to the 10/9/2017 rain event.
	4	-	Int of Greenleak and Pearl	2689	71" x 60"	71" x 60"	71" x 60"	8/22/2017	11/6/2017	
	5	-	Int of Bereford and Pearl	2693	71" x 60"	71" x 60"	71" x 60"	8/22/2017	11/6/2017	
	6	-	Int of Maplewood and Pearl	2687	60"	60"	60"	8/22/2017	11/15/2017	
	7	-	Int of Lawndale and Parmapark	2695	54"	54"	54"	8/21/2017	11/15/2017	
	8	-	6891 Orchard	2697	48"	48"	54"	8/21/2017	11/15/2017	
	9	-	7015 Orchard	2216	30"	30"	30"	8/21/2017	11/15/2017	Overflow Meter. Small or no reaction.
	10	-	Int of Parmapark and Oakdale	2688	48"	30"	48"	8/22/2017	11/14/2017	
	11	-	7071 Maplewood	2629	48"	48"	48"	8/21/2017	11/14/2017	
	12	-	6815 Maplewood	2698	42"	54"	54"	8/21/2017	11/14/2017	
	13	-	7022 Parmapark	2692	42"	48"	48"	8/21/2017	11/14/2017	
Rain Gauge Sites										
Site #			Street/Location		Telemetry			Installed	Removed	Comments:
1			Parmapark Elementary School - 6800 Commonwealth Blvd	372439	Telog			8/22/2017	11/16/2017	Rain events:
2			Innvictus High School - 7099 W 130th	372256	Telog			8/22/2017	11/16/2017	Rain events:

Figure 3-2: Flow Meter Schematic for Storm Sewer Study



3.3 RAINFALL GAUGES

Two rain gauges were installed at different locations around the project areas to monitor the wet weather effects on the system. Because rain events produce more rainfall in certain areas over others, these rain gauges helped explain the different responses in the system from each individual storm. The eastern rain gauge, known as RG-1, was located at Parma Park Elementary School; the western rain gauge, known as RG-2, was located at Invictus High School. The locations of the two rain gauges are shown on Figure 3-1.

The tipping bucket rain gauges used on this project recorded every 0.01 inch of rain and reported data in 5-minute intervals using a Telog data collector. On October 5, RG-1 data was found questionable, with much lesser intensities than RG-2. On this day, RG-2 data was used as a substitute.

Two NEORSD rain gauges were used to confirm collected data. Those are the Brook Park and Parma rain gauges. These two rain gauges were used to collect rainfall data from April 19, 2017 and June 30, 2017 both of which had recorded flooding in the project area.

3.4 RAINFALL DATA

The flow meters and rain gauges were installed for an extended period of time and numerous wet weather events were observed. Not all storms created a reaction in the system. Some small storms produced so little precipitation that no sewer response was apparent. Other storms were so large that the system did not return to dry-weather flow conditions before the next rain event. In these cases, storms occurring within 12 hours of each other were considered a single event. Events ranged from 0.01" to 2.17" of rain. In all, there were twenty-nine (29) events recorded during the flow monitoring period.

Figure 3-3 compares the recorded rainfall events, and for the two large storm events that occurred on 4/19/17 and 6/30/17, to the 6-hour design storms by plotting total rainfall depth versus peak hour intensity. The storm return periods, or recurrence intervals, can be estimated. It should be noted that the event on April 19 was close to a 25-year design storm. The dates for analyzed storms are listed in full in Table 3-3.

Figure 3-3: Rainfall Recurrence Intervals

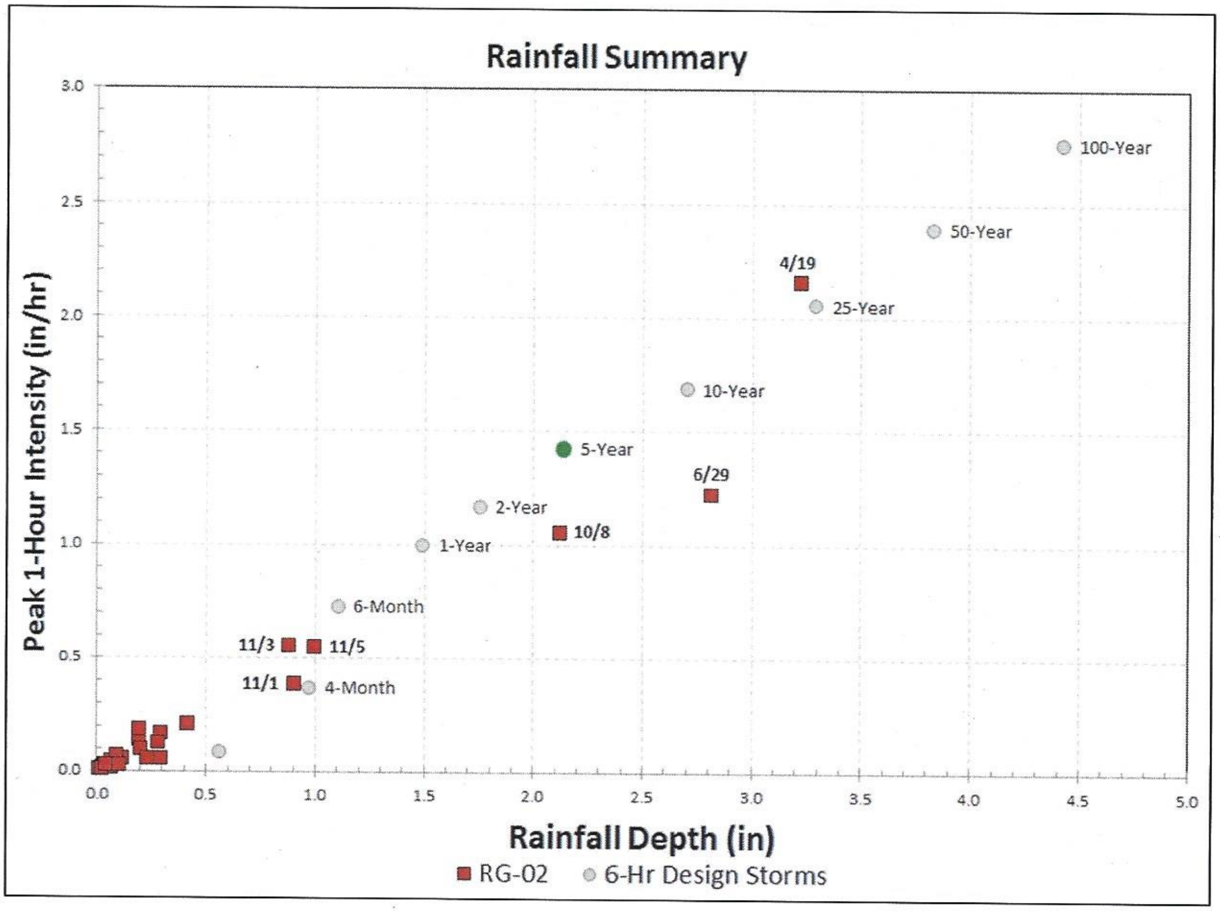


Table 3-3: Analyzed Storm Events

Statistical Summary of Storm Events (Averaged)					
#	Rain Start	Rain End	Duration (Hour)	Peak Intensity (Inch/Hour)	Total Depth (Inch)
1	9/4/2017 9:40	9/5/2017 15:15	29.6	0.21	0.42
2	9/6/17 17:00	9/6/17 18:20	1.3	0.17	0.17
3	10/6/17 5:15	10/6/17 14:20	9.1	0.14	0.26
4	10/8/17 1:10	10/9/17 11:40	34.5	1.10	2.17
5	10/11/17 4:50	10/11/17 16:30	11.7	0.10	0.19
6	11/1/17 9:05	11/2/17 5:05	20.0	0.38	0.89
7	11/3/17 1:30	11/3/17 7:35	6.1	0.49	0.78
8	11/5/17 3:50	11/6/17 7:20	27.5	0.55	0.99
	4/19/17 10:25	4/21/17 2:50	40.4	2.16	3.22
	6/29/17 19:00	7/1/17 6:40	35.7	1.23	2.81

4.0 COLLECTION SYSTEM MODEL DEVELOPMENT

The purpose of this section of the report is to describe how the model of the project areas sewer system was built.

4.1 MODELING SOFTWARE

The comprehensive hydraulic model was developed using Personal Computer Storm Water Management Model (PCSWMM) version 5.1.011. This modeling software was developed by Computation Hydraulics International (CHI) and combines GIS support capabilities with hydrologic and hydraulic model analysis. This software provides a continuous simulation of runoff quantity and quality over long term durations. PCSWMM also has the multifaceted ability to operate with various management applications.

4.2 MODEL APPROACH

The model was created based off record drawings obtained from the online NEORSD GIS portal. Record drawings date from the early 1920s to the early-2000s. Almost 110 pages of record drawings were reviewed. Information obtained from these record drawings included:

- Manhole locations, elevations, and inverts.
- Pipe lengths, shapes, upstream and downstream inverts, installation date.

In some instances, record drawings contained conflicting information. These conflicts were then resolved through field investigation and updated in the model to accurately represent the hydraulic capabilities of the system.

Using this GIS information, a skeletonized model was created to represent the storm sewer system. All pipes within the project area were used in model development; along with smaller pipes that demonstrated significant hydraulic impact on the system. In total: 55,054 feet of pipe and 178 manholes are referenced in the model. Pipe diameters, or heights for box culverts, range from 12 to 72 inches.

This GIS information conforms to the North American Datum (NAD83) datum, measured in U.S. survey foot. Record elevations were originally detailed using an unknown datum and converted into NAD83 standards by adding 573.23 feet. This number was derived by averaging elevation differences between older and newer record drawings.

4.3 HYDRAULIC MODELING SPECIFICATIONS

This section describes the standards that were applied in creating the hydraulic model.

Manhole/Model Nodes

- Entrance and exit head losses in manholes were set to zero by default. In special circumstances, such as sharp angles in a pipe network, additional manhole losses were considered.

- Rim and invert elevations and geometry of existing manholes shall be based on the most recent record drawings available. Other sources, such as field inspections and surveys, were also utilized.

- Manholes were assumed to have a diameter of four feet, or a cross-sectional area of 12.56 square feet (sf) as the default value.

- Storage nodes were used to represent surface runoff for areas outside the project boundaries.

Conduits

- Conduit length, shape, size, and invert elevations were based on the most recent record drawings available. Other sources, such as field inspections and surveys, were also utilized.

- Hydraulic friction losses for all conduits shall be calculated using recommended Manning's roughness coefficient values (Table 4-1).

Weirs were also input into the model when necessary to represent flow transfers from outside catchments runoff into the project area during surface flooding.

Table 4-1: Roughness Coefficient Values Used

Conduit Material	Abbreviation	Manning Roughness
ABS	ABS	0.0125
Brick	BRK	0.017
Cast In Place Concrete	CIP	0.0143
Cast Iron Pipe	CAS	0.0143
Centrifugally Cast Fiberglass Reinforced Polymer Mortar/HOBAS	FRP	0.013
Concrete Pipe (non-reinforced)	CP	0.015
Corrugated Metal Pipe	CMP	0.024
Ductile Iron Pipe	DIP	0.0143
Polyethylene	PE	0.0143
Polyvinyl Chloride	PVC	0.0125
Reinforced Concrete Pipe	RCP	0.015
Reinforced Plastic Pipe (Truss Pipe)	RPM	0.0143
Segmented Block	SBK	0.020
Steel Pipe	SP	0.0143
Stone	STN	0.017
Vitrified Clay Pipe	VCP	0.0143

4.4 MODEL DEVELOPMENT

Two other model elements are critical for calibration. These are the creation of catchment areas for each flow meter (and the appropriate subcatchments for all but the smallest metering areas) and flow application, which is defining which sewer node to apply the flow originating from a given catchment. This section will explain the creation of catchments and the flow application in those catchments.

Subcatchment Development

After all the physical system elements were developed, catchment areas were delineated. These area delineations are based on sewer connectivity, ground surface contours, uniform land use, and flow monitoring placement. After general catchments were developed, more detailed subcatchments were created using model layout and existing parcel information. These subcatchment boundaries were created at various hydraulic control points; such as, system braches, significant junctions, and flow monitoring locations. Figure 4-1 contains an example of branch and flow monitor subcatchment divisions.

Subcatchment sizes are generally 30-50 acres for residential areas but for this project they were about 25 acres. The largest subcatchment is located in area 13-2 and is 171.1 acres. The smallest subcatchment is located in area 13-2 (Nathan Hale Park) and is 1.49 acres. Acreage for most of these areas was calculated using GIS. There were three areas where the acreage was increased manually due to lack of tributary understanding and flow volumes. These areas included Area 6, 12 and 13.

Figure 4-1: Subcatchment Example

Subcatchment Flow Influences

There are four main factors that influence subcatchment response to rainfall; percentage of impervious area, drainage width, flow length and slope. Each subcatchment within a catchment area consists of the same values for these four factors.

Impervious areas are mainly artificial structures such as pavements, rooftops, and compacted soil. These areas are the primary source of storm water runoff as the ground surface is sealed which eliminates rainwater infiltration and natural groundwater recharge. Impervious area values for the project area range from 16.5% to 65%. These percentages were determined by extrapolating data from the USGS National Land Use Database and parcel information for each subcatchment and then using flow data to accurately calibrate a representative value.

Subcatchment width is defined as the ratio of the contributing area to its flow length. Widths began at an estimated value and were adjusted to mimic the shape and time of concentration of the metered hydrographs.

Flow length was automatically calculated by PCSWMM using the formula below:

$$\textbf{Flow Length (ft)} = \frac{\textbf{Contributing Area (ft}^2\textbf{)}}{\textbf{Width (ft)}} \qquad \text{Equation 4-1}$$

Slope is the average slope of the subcatchment area. This parameter was commonly left as a constant to simplify calibration efforts. Values are derived from existing contours. In certain cases, this value was adjusted. Values range from 0.267% to 3.0%.

Seasonal Variations

The model accounts for seasonal variances by using variable evaporation rate constants and a groundwater modulator.

Fluctuating temperatures throughout the year strongly affect the groundwater tables. Usually, groundwater tables are highest in the winter, due to a lack of evaporation, and lowest in the summer months, with spring and fall being transitional periods. Depending on pipe and manhole elevations and locations, this fluctuation in groundwater can have a large effect on infiltration rates. To account for this, the Climatology function in PCSWMM was utilized with monthly evaporation averages specific to Cleveland, OH (Table 4-2). These values were found in the EPA National Stormwater Calculator.

Table 4-2: Monthly Evaporation Constants (in/day)

Jan	Feb	Mar	Apr	May	Jun
0.04	0.08	0.15	0.2	0.25	0.29
Jul	Aug	Sep	Oct	Nov	Dec
0.29	0.25	0.19	0.15	0.13	0.08

The groundwater component creator was used to simulate groundwater infiltration rates and attributes are specific for each individual catchment. This application is influenced by evaporation rates and rainfall. Only two areas utilized this function, Areas 12 and 13. The groundwater application is the final component utilized for flow calibration.

4.5 DEFINITION OF FEATURES

Naming Conventions

- Catchments – Catchments are labels with an "S" preceding the associated flow monitor that recorded flows for that area.

  Example: S8 contains the sanitary sewer that is tributary to flow monitor 8.

- Subcatchments – Each subcatchment is labeled similarly to the catchment it is a part of, but is followed with a dash and a number. The numbers were assigned starting at the area closest to the flow monitor moving in a clockwise rotation. Example: S8-1, S8-2, S8-3, etc. There were a few subcatchments utilized to represent overland flow. These subcatchments are labeled as SOF followed with a dash and number. Example: SOF-1, SOF-2, SOF-3, ect.

- Pipes – Pipes are labeled by SWMM's default functionality. They are given a number in order they were created preceded by a "C" for conduit.

  For example: C120 would be the 120th conduit created during model build out.

- Manholes - Manholes are labeled by SWMM's default functionality. They are given a number in order they were created preceded by a "J" for junction.

  For example: J15 would be the 15th junction created during model build out.

Pipe Information

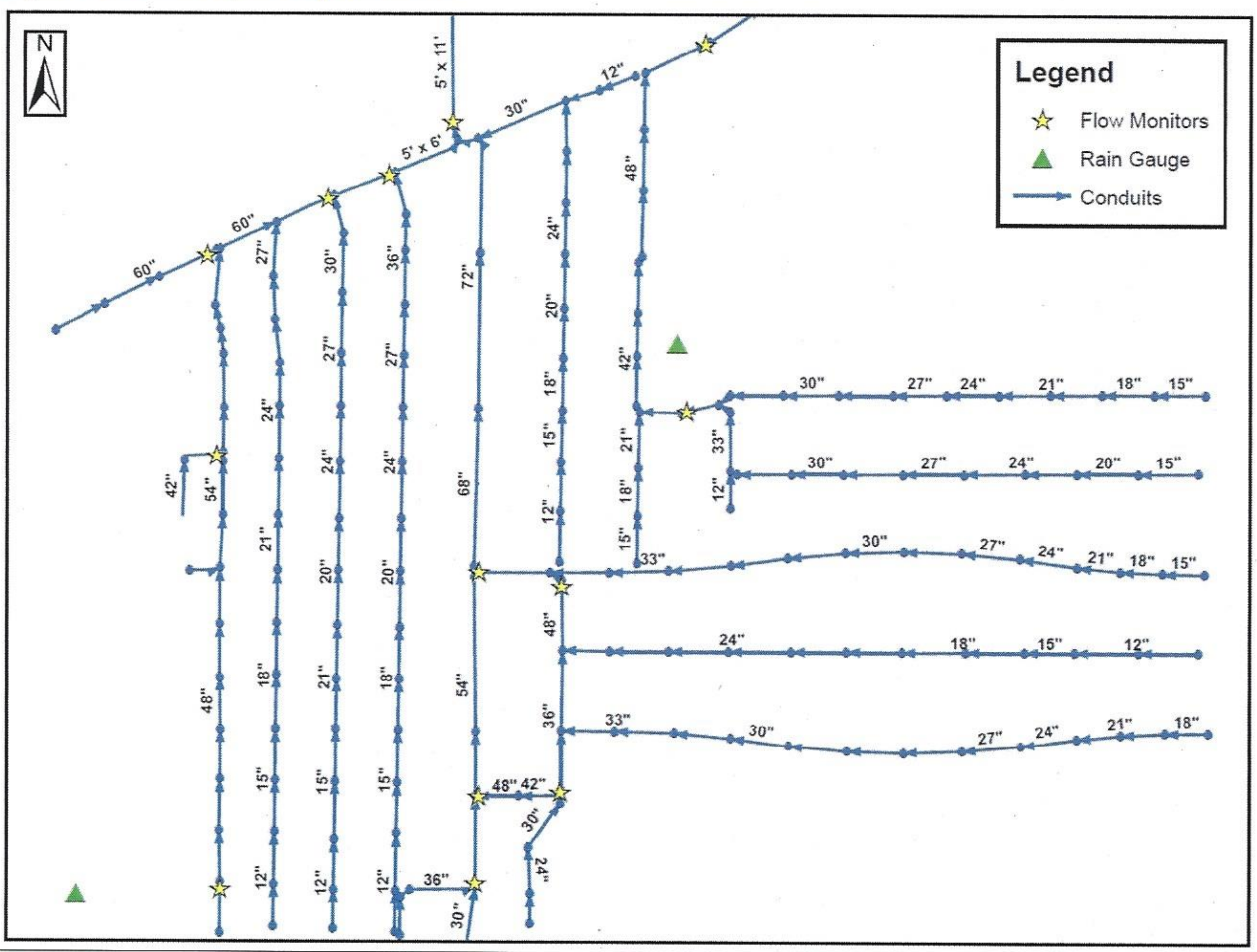
All pipe dimensions are listed in feet. Most of the pipes constructed in Parma Heights are concrete (CP). Pipes are generally circular in shape, with the exception being the sewer along Pearl Rd between Oakwood Rd and Parma Park Blvd. These sewers are rectangular in shape; either 5' x 6' or 5' x 11'. FM 1 was installed on a 5' x 11'rectangular sewer pipe heading north along Beverly Drive.

Table 4-3 lists the number of different pipe segments that were modeled by pipe diameter, as well as the total sewer length. Figure 4-2 shows an overview of the modeled system with each pipe section labeled with its diameter.

Table 4-3: Modeled Pipe Details

Pipe Details		
Size	Number	Length
Circular		
12"	13	3572'
15"	12	3622'
18"	11	3297'
20"	7	2198'
21"	11	3222'
24"	20	6170'
27"	12	3738'
30"	24	6820'
33"	8	2441'
36"	8	2260'
42"	8	1853'
48"	16	4760'
54"	15	4649'
60"	5	1679'
68"	1	911'
72"	2	1575'
Box		
5'x6'	4	1283'
5'x11'	2	1004'

Figure 4-2: Modeled System Pipe Diameters



4.6 PROCESS AND METHOD OF BUILDING THE MODEL

The general process for developing the sewer model was the following:

1. Create and label all junctions, conduits, storages, weirs, and all other unique system attributes. These should accurately reflect the most recent record drawings. Perform any necessary field investigations to confirm or update information.
2. Create catchments based on flow meter locations.
3. Create subcatchments for all storm sewer branches within each catchment.
4. Calibrate model to calibration standards.

Junction Flow Assignment

After all subcatchments were created and base flows, or ground water infiltration, values calculated, junctions contained within the area were analyzed. In order to input these flow values into the system, each subcatchment needs to be assigned a junction to receive the flow/runoff.

5.0 COLLECTION SYSTEM MODEL CALIBRATION

The purpose of this section of the report is to describe how the model of the project areas storm sewer system was calibrated.

5.1 CALIBRATION INTRODUCTION AND STANDARDS

After GIS development, the model was calibrated to replicate rainfall response using data collected during the monitoring period. Specific storms that were recorded during the flow monitoring period were selected for calibration and the results were analyzed. Important parameters include rate of flow and depth of water throughout the system.

Standards for these parameters are similar to the accepted standards used by the Northeast Ohio Regional Sewer District (NEORSD). These standards include separate criteria for rain event selection and wet weather flow (WWF).

Rain Event Criteria

- At least three (3) acceptable storm events available at each flow monitor.
- An inter-event period of 12-hours was used in developing rainfall event statistics.

Wet Weather Flow Criteria

- All rainfall events with a peak hourly rainfall intensity greater than or equal to 0.25 in/hr were used and held to criteria for wet weather calibration.
- Observed and model-predicted hydrographs had to meet the following criteria in at least 60% (3) of the rainfall events observed during the monitoring period:
    - Timing of the peaks and troughs similar to the event durations.
    - Peak flows at each significant peak in the range of -20% to +20%.
    - Volume of flow in the range -20% to +20%.
    - Surcharged flow depths in the range -0.33 feet to +1.64 feet.
    - Non-surcharged flow depth within range of ± 0.33 feet.

5.2 MODEL CALIBRATION

Wet Weather Flow Definition

Stormwater enters the system through points of direct connection. Footing/foundation drains, roof drains, downspouts, drains from window wells, driveway drains, groundwater/basement sump pumps, and even streams can account for direct connections.

Groundwater also enters the storm sewer by infiltration through cracks or leaks sewer pipes. These issues can be related to age of sewer, loose joints, poor design, poor installation, or root intrusion. Pipes that lie beneath the groundwater table, or cross under rivers/streams, are particularly susceptible to infiltration. The average sewer pipe has a stated design life of 20-50 years, depending on the material used. Other sources of infiltration include downspout or lateral connections to houses and businesses. These connections usually go prolonged periods without inspection and can be cracked or damaged. Some areas in the project area demonstrated groundwater response. Similar issues regarding manholes can contribute to infiltration.

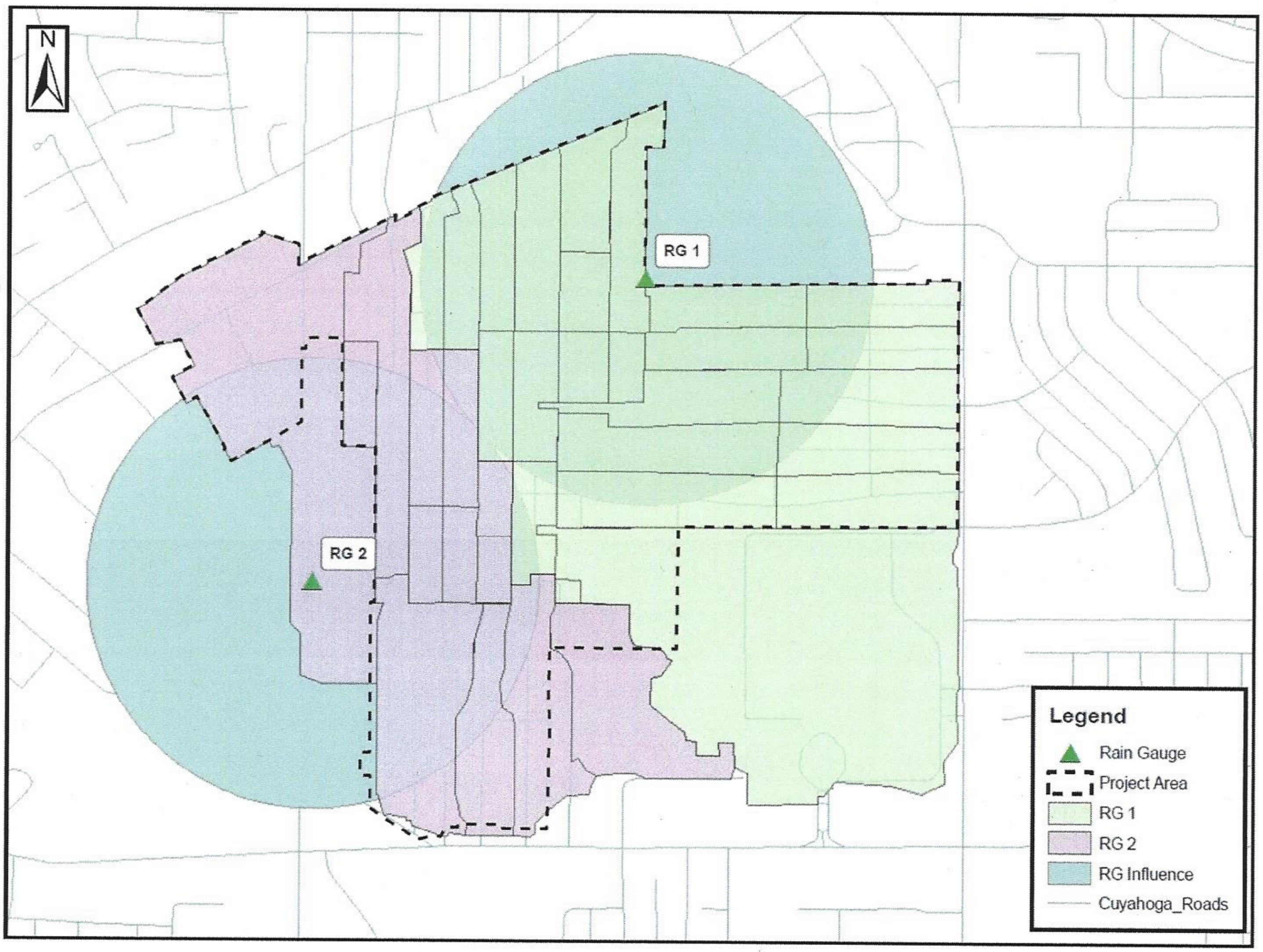
Rain Events

Observed rain events were simulated in PCSWMM by utilizing its time series function. During the monitoring period, two rain gauges were installed within the project area; so two different rain gauge time series were created. The storm events were reproduced based on the 5-min intensities recorded at each location. These locations can be seen in Figure 5-1. Each subcatchment was then assigned a rain gauge time series based on proximity to the installation locations as shown in Figure 5-1. The number of subcatchments assigned to each rain gauge is shown in Table 5-1.

Table 5-1: Rain Gauge Characteristics

Rain Gauge	# of Subcatchments	Area of Coverage (acres)	Total Recorded Rainfall (in)	Max Rainfall Intensity (in/hr)
RG 1	27	410	7.23	1.13
RG 2	13	298	7.59	1.06

Figure 5-1: Rain Gauge Assignment



Wet Weather Flow Calibration

There are two main factors when calibrating wet weather flow (WWF).

- Peak Flow
- Total Flow Volume

Peak flow is the maximum flow rate that the flow monitor records during the peak, or most intense, duration of a storm event.

Total flow volume is the total amount of flow recorded during the duration of the storm event and its runoff period. For this project the runoff period varied based on sewer response. Typical runoff periods ranged from 12 hours to 48 hours.

The two subcatchment parameters utilized in WWF calibration are:

1. Drainage Width – used to calibrate peak flow
2. Percent Impervious – used to calibrate flow volume

Drainage width is defined as the width of the overland flow path (feet). A base value of 500 was set for all subcatchments within the model. Decreasing the width produces a smaller peak flow, and increasing the width produces a larger peak value.

Percent impervious is defined as the percent of area covered by impervious materials such as pavement, roofs, or compacted soil. These values were initially determined using the National Land Use Database and parcel information. Initial percent impervious values can be found in Table 5-2. Decreasing the percent impervious value produces a decrease the total flow volume, and increasing the percent impervious value increases the total flow volume.

Table 5-2: Initial Percent Impervious Values

Area	Percent Impervious
1	28.05
2	34.98
3	30.44
4	26.71
5	46.07
6	76.85
7	22.00
8	25.98
9	-
10	23.96
11	38.63
12	10.45
13	6.01

Utilizing PCSWMM's Sensitivity-Based Ratio Tuning Calibration (SRTC) tool, both width and percent impervious values are able to be calibrated at the same time. This tool allows the user to input an uncertainty percentage, ranging from 0%-100%, for multiple attributes. The model then analyzes different values for these attributes within the range of uncertainty. Modifiable results are interpolated between values and the user can manually adjust the parameters until calibration standards are met. This function allows the observed flow meter data to be opened and compared to calibration results. This also allows the combined width and percent impervious impact to be observed during calibration. Width and percent impervious values are similar for all subcatchments within the same area.

Once peak and flow volume have been calibrated within the standards, groundwater effects are applied and calibrated.

Groundwater Influence

There are many factors that influence the groundwater effect on a storm sewer system. An important one is water table elevation. The water table elevation must be higher than the lowest pipe invert in order to infiltrate the system. This elevation is determined based on soil infiltration rates, soil porosity, structural health of the system, and time of year.

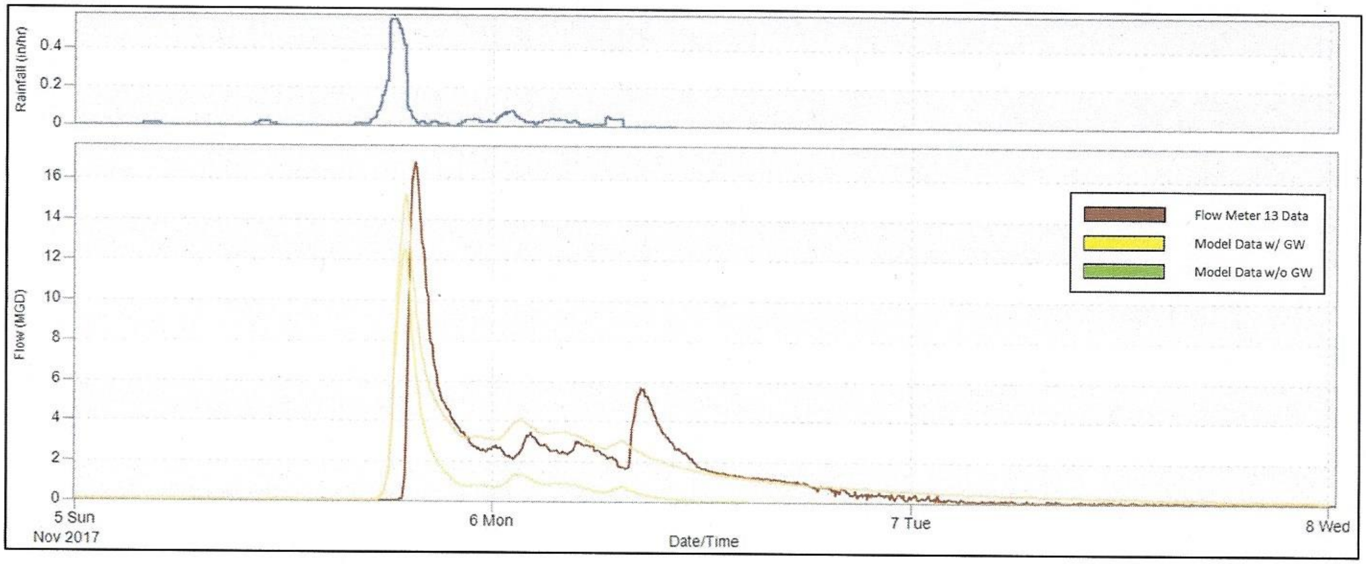
PCSWMM also has an infiltration tool that accounts for soil saturation, but this function was not used for this project.

Structural health of the sewer system is another major factor contributing to groundwater influence. Structural deficiencies in joints and pipes allow excess water to infiltrate through cracks and leaks. If the groundwater in the surrounding area is high in comparison to the pipe elevations, then large volumes of excess water can enter the system.

The time of year is also an important factor. Since evaporation rates change throughout the year (see the seasonal variation discussion in section four), groundwater levels change accordingly. Typically, groundwater effects are higher during the winter when evaporation rates are low and lower during the summer when evaporation rates are higher. Spring and fall have medium values as they are transitional seasons. As an example of groundwater response, Figure 5-2 shows the same storm simulated with and without the groundwater component turned on.

Local, or adjacent, bodies of water such as ponds, streams, or rivers, can have an effect on groundwater response as well. For older sewers or sewers located near bodies of water, it is important to model groundwater effect.

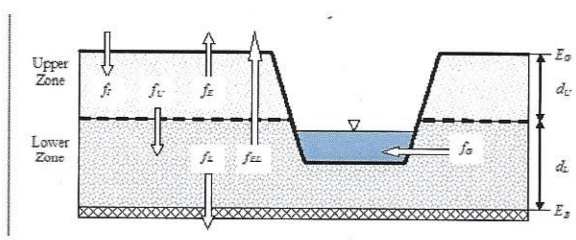
Figure 5-2: Groundwater Component Effect



Modeling Groundwater

The groundwater module in PCSWMM consists of two levels: an upper, unsaturated zone and a lower, saturated zone. Flow enters through infiltration (f_I) and can exit via evaporation (f_{EU}), percolation to the deep groundwater (f_L), or it can enter the collection system as lateral groundwater inflow (f_G). This graphic (Figure 5-3) shows a stream channel, but the same scheme applies for piped flow.

Figure 5-3: SWMM Groundwater Diagram



Where:

d_U = depth of the upper zone

d_L = depth of the lower zone

f_U = percolation from the upper to lower zone which depends on the upper zone moisture content θ and depth

f_{EL} = evapotranspiration from the lower zone, which is a function of the depth of the upper zone

f_I = infiltration from the surface

f_{EU} = evapotranspiration from the upper zone which is a fixed fraction of the un-used surface evaporation

f_L = percolation from the lower zone to deep groundwater which depends on the lower zone depth

f_G = lateral groundwater interflow to the drainage system, which depends on the lower zone depth as well as the depth in the receiving channel or node.

Groundwater is routed through the system via aquifers. Each subcatchment has its own aquifer that is created with the groundwater component creator. This creates basic groundwater parameters used for routing infiltration from rainfall. Surface elevations for aquifers are similar to the bottom invert elevation of the lowest manhole within each catchment. These aquifer elevations are different for each catchment and each aquifer only reacts within its assigned catchment. The main attribute used to calibrate groundwater response is the A1 coefficient. This coefficient is a multiplier that affects the amount of groundwater flow within a conduit or pipe. Values range from 0 to 0.8. The larger the value is, the more influence, or infiltration. This value also affects the volume of groundwater flow.

Other parameters include the Lower Groundwater Loss Rate, which determines the rate of seepage through the aquifer, and the initial Water Table Elevation. The Lower Groundwater Loss Rate affects the runoff time of each catchment. Figure 5-1 shows a prolonged runoff period, or falling limb, in the hydrograph, and correlates with a lower runoff value.

Level Calibration

Once flow calibration is complete, the depth of water needs to be calibrated in order to match observed data. Correct water levels ensure that no false reports of surcharging or flooding are reported by the model. This will also produce accurate overflow results for regulated areas.

Levels are calibrated by adjusting the Manning's roughness coefficient in the pipes, or through pipe size manipulation. Adjusting the Manning's coefficient changes the velocity of flow traveling through the conduits. Adjusting the pipe size changes the pipe area, which affects the level. Neither parameter effects flow calibration based on the principle of uniform flow, or the Continuity of Flow (Equation 5-1).

$$\boldsymbol{Q = V_1 x A_1 = V_2 x A_2} \qquad \text{Equation 5-1}$$

If the roughness is altered, then the velocity changes and the area of pipe, that contains water, changes, if the pipe size is altered, then the velocity changes. Since velocity is not calibrated, it acts as the balancing force between flow and level values.

5.3 SYSTEM WIDE CALIBRATION ISSUES

There were various issues that prolonged calibration efforts and relate to inconsistencies in model results.

Downstream Levels and Effects

Since the interaction downstream of flow monitor 1 is not known, the levels in flow monitor 1 are reported slightly lower than some of the recorded values. This is due to an open outfall modelled at the end of conduit C746. In actuality, there are other storm pipes with varying flow that help sustain levels in the monitored pipe.

Unknown Factors

Other unknown factors that can affect calibration include blockages or other flow restrictions, relief pipes, undocumented overflows, silt build-up, inaccurate record drawings, or other sources of surface flooding that travel to the project area.

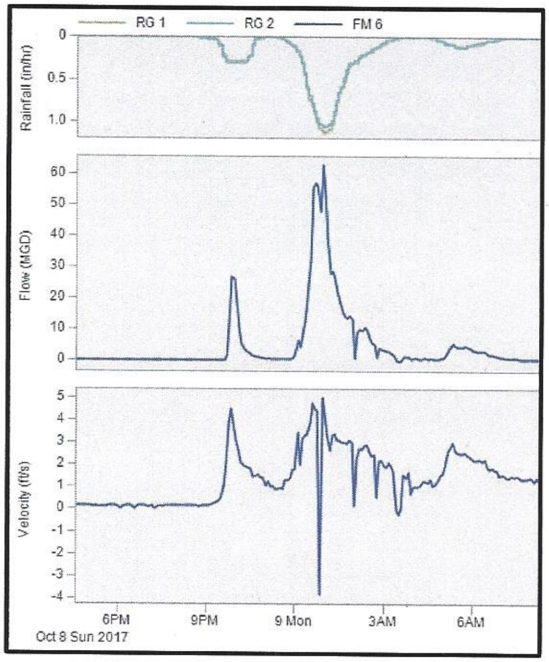
Variable Meter Data

Certain flow monitoring locations proved difficult for monitoring. Bad sensor responses can be caused by a variety of problems that make it difficult for accurate data collection. These include silt buildup, low velocities, debris covering the sensor, and shape of pipe. There were five flow monitors that produced results that made calibration difficult during certain time periods. Poor velocity readings were often the culprit.

- FM2 – One questionable velocity on Sept 4th.
- FM3 – Data was missing during the Oct. 6th, 8th and 11th storms due to meter failure.
- FM6 – Two questionable velocities on Oct. 8th and Nov. 1st.
- FM8 – Questionable velocities on Oct. 11th, Nov. 3rd, and Nov. 5th.
- FM10. – Questionable velocity on Oct 8th.

Figure 5-4 is an illustration of questionable velocity for meter FM6, but shows that the flow data (peak flow, general hydraulic pattern) is not severely affected.

Figure 5-4: FM 6 Questionable Velocity Example



5.4 MODEL VERIFICATION

After calibration, model data was exported to Excel for calibration analysis. Hydrographs and statistical plots were created for each flow monitor and compared to 8 various storm events throughout the monitoring period. This analysis includes the five calibration events along with 3 additional storms varying in intensity (Table 5-3). The model was also run outside the flow monitoring period, for the two major events that occurred in spring 2017.

Model verification results can be located in Appendix A.

Table 5-3: Model Verification Storm Events

Statistical Summary of Storm Events (Averaged)					
#	Rain Start	Rain End	Duration (Hour)	Peak Intensity (Inch/Hour)	Total Depth (Inch)
1	9/4/2017 9:40	9/5/2017 15:15	29.6	0.21	0.42
2	9/6/17 17:00	9/6/17 18:20	1.3	0.17	0.17
3	10/6/17 5:15	10/6/17 14:20	9.1	0.14	0.26
4	10/8/17 1:10	10/9/17 11:40	34.5	1.10	2.17
5	10/11/17 4:50	10/11/17 16:30	11.7	0.10	0.19
6	11/1/17 9:05	11/2/17 5:05	20.0	0.38	0.89
7	11/3/17 1:30	11/3/17 7:35	6.1	0.49	0.78
8	11/5/17 3:50	11/6/17 7:20	27.5	0.55	0.99
	4/19/17 10:25	4/21/17 2:50	40.4	2.16	3.22
	6/29/17 19:00	7/1/17 6:40	35.7	1.23	2.81

6.0 DEVELOPMENT OF EXISTING AND BASELINE CONDITIONS

6.1 DEVELOPMENT OF DESIGN CONDITIONS

There are many adjustable factors that affect model predictions and responses. These range from specific technical settings to unknown characteristics of the storm sewer being modeled. For this project, the technical settings in PCSWMM were left to operate under the default functionality. The unknown sewer characteristics were guided by a set of engineering assumptions utilizing previous modeling experience.

6.1.1 ENGINEERING ASSUMPTIONS

While all available information was utilized during this project, some assumptions were applied when building and calibrating the model.

Record drawing accuracy - Record drawing elevations and layouts were considered accurate. If discrepancies important to the model were found, or where record drawings were missing, field investigations resolved the difference and accurate data was inserted into the model.

Pipe roughness coefficients - Pipe roughness coefficients were assumed as 0.015 for all pipes throughout the system. This correlated with the average value for clay and concrete pipes (Table 4-1). Most conduits throughout the City are constructed of these materials. Roughness was adjusted for calibration purposes if necessary.

Manhole diameters - Manhole diameters were assumed four feet in diameters for all manholes created in the model.

Level of silt in pipes - Silt influence was not included unless noted during field investigation or through previous knowledge.

Flow restrictions or obstructions - The sanitary system was assumed to have no blockages or obstructions when modeling.

Structural health of the system - Structural health of the system was accounted for when calibrating groundwater effect into the system. Areas with high infiltration rates are considered less structurally stable than areas with low infiltration rates. .

Storm sewer or drain connections - The sewer system is assumed to work as a separate sewer system. No sanitary interaction was developed.

6.1.2 OTHER CONCERNS

The tributary area and storm runoff responses for areas 6, 12 and 13 are still unknown. These three areas demonstrate higher flows than what original model predictions include. Each areas percent impervious value and total area was increased manually to match recorded flows. There are two possibilities to this response. First, the tributary area reacts differently under various sized storms due to runoff, groundwater response, and intensity. Unfortunately, not enough data could be collected to perform an accurate analysis in these areas. Secondly, surface flooding or runoff from other catchments may be interacting with the storm sewers in this area causing higher flow readings than what are normally recorded. This could be due to debris in other pipe networks or in curb inlets causing blockage and restrictions. Both of these issues would cause more flow to travel along major flow paths (e.g., roadway surface, right-of-way, boulevards, ditches, swales, and backyards) to minor flow paths (e.g. pipe collection systems) outside of the original tributary area.

Another issue is that there are no recorded drawings of the sewer pipes in Area 12. According to aerial images there seems to be storm inlets throughout the vacant lots southwest of the metered pipe. It is unknown how far this pipe network extends or what the condition of these pipes are. For this model the adjacent vacant area was assumed to be the tributary limit and the area was manually adjusted with the percent impervious area to increase modeled flow.

Flow metering areas 3, 7 and 10 were difficult to calibrate using standard methods. To increase peak flows and volumes for calibration storms false storages and conduits were created to mimic runoff from arterial streets downstream of their tributary areas. Flow capturing issues along York Rd and West Pleasant Valley Rd could result in flow traveling along streets curbs and into the metering areas. This makes the most sense for areas 3 and 7 where there are steep changes in elevation from upstream to downstream allowing flow to naturally travel into the area. Results were positive with model predictions for the October 8th storm, which was the largest recorded storm of the monitoring period, and other calibration storms being brought into standards.

6.1.3 FUTURE IMPROVEMENTS

Other than the proposed storage basin there are no other future improvement plans currently existing for this area.

7.0 HYDRAULIC ANALYSIS

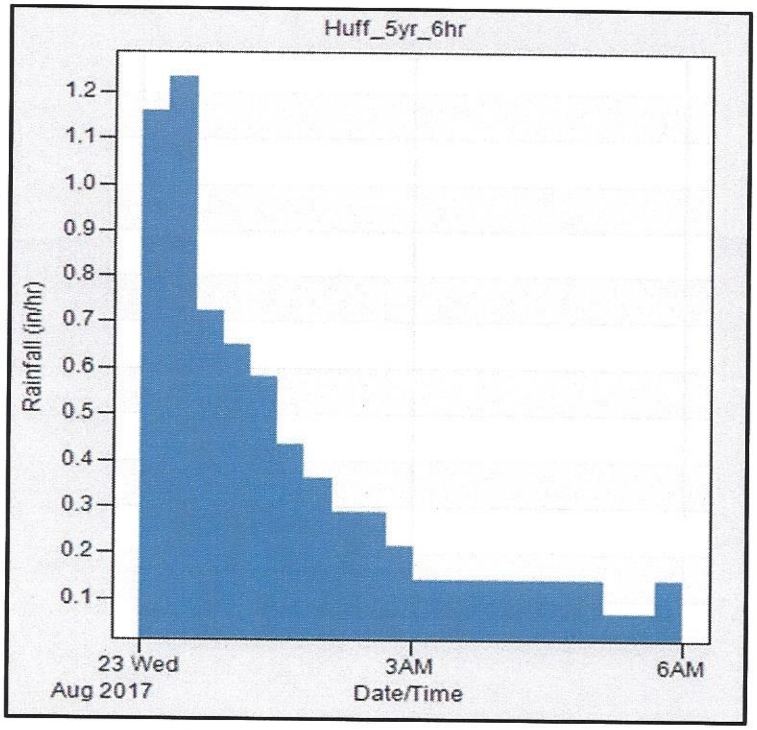
7.1 DESIGN STORMS

The current systems hydraulic performance was analyzed against three design storms. Each storm has a 6 hour duration with average return intervals (ARI) of 5 years, 10 years, and 100 years. These Precipitation Frequency Estimates were taken from the NOAA Atlas 14 values for Cleveland Hopkins Airport. These values can be seen below in Table 7-1. An example hydrograph for these three events is shown in Figure 7-1. This is a First Quartile hydrograph for a 0-10 square mile area. The project area is 680.5 acres or about 1.06 square miles.

Table 7-1: Design Storm Values

Design Storms				
Storm	ARI (years)	Duration (hrs)	Depth (in)	Peak Intensity (in/hr)
#1	5	6.0	2.17	1.23
#2	10	6.0	2.56	1.451
#3	100	6.0	4.16	2.357

Figure 7-1: First Quartile Hydrograph (5 Year Storm)



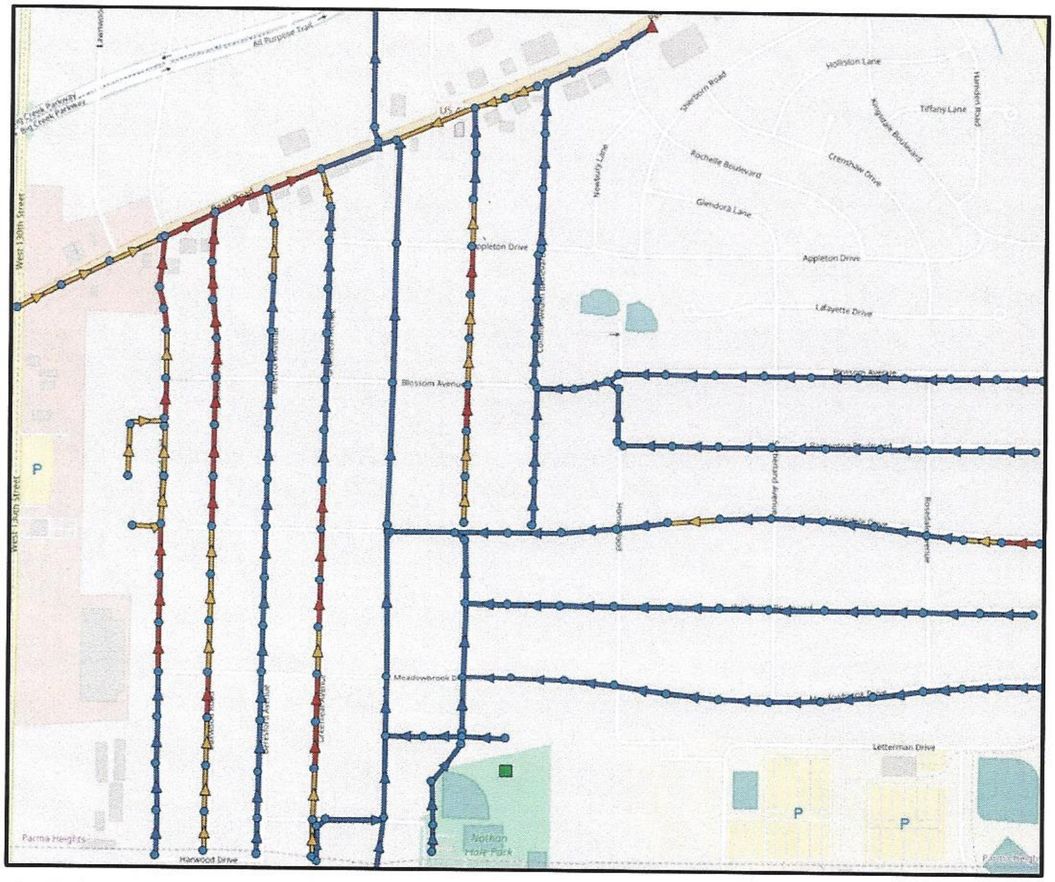
7.2 EXISTING SYSTEM CAPACITY ANALYSIS

For each design storm a render of hydraulic restrictions was created. These renders were applied using the Render and Query functions for modeled conduits that are available in SWMM. The following renders were applied;

- A pipe is shown to be operating at or above full capacity when the max/full depth is equal to one and the max/full flow is equal to or greater than one. A pipe at full capacity is shown in red.
- A pipe is shown to be hydraulically restricted by a downstream issue when the max/full depth is equal to one and the max/full flow is less than one. A pipe under downstream hydraulic restriction is shown in orange.
- Pipes operating without any restrictions are shown as blue.
- Manholes with a Total Flooded Volume greater than 0.001 MG are highlighted as green. Manholes without flooding are rendered blue.

A plan view of the project area with rendered pipes is provided for each design storm in Figure 7-2 through Figure 7-4. As expected, the more severe the storm the greater hydraulic restriction. Table 7-2 summarizes the design storm results and reports surcharged conduits, flooded manholes, and flooded volumes.

Figure 7-2: 5-Year 6-Hour Storm Results



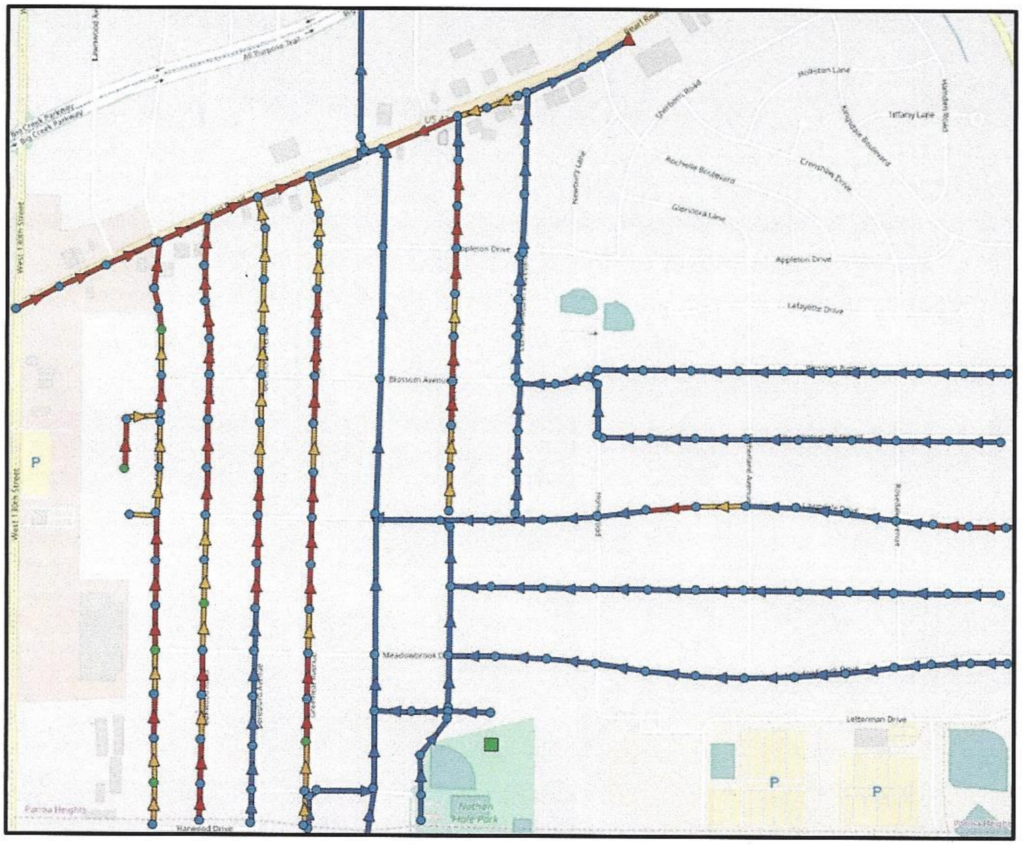


Figure 7-3: 10-Year 6-Hour Storm Results

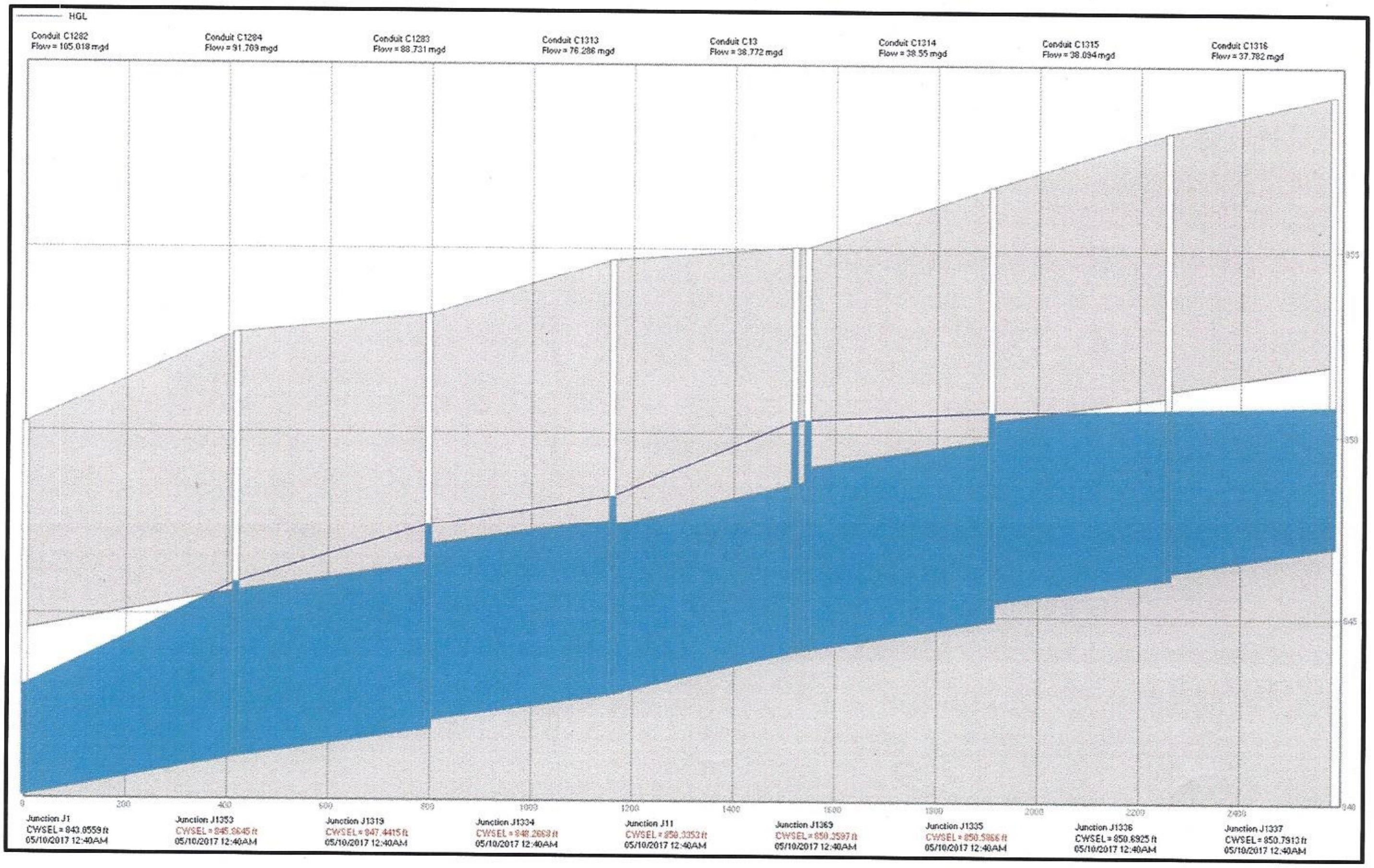
Figure 7-4: 100-Year 6-Hour Storm Results

Table 7-2: Design Storm System Summary

Design Stom	Number of Surcharged Pipes	Number of Pipes at Capacity	Number of Pipes Hydraulically Limited	Number of Flooded Manholes	Total Flooded Volume (MG)
5 year 6 hour	52	32	20	2	0.016
10 year 6 hour	81	51	30	8	0.196
100 year 6 hour	120	89	31	24	5.339

Maximum Hydraulic Grade Lines (HGL) for surcharged conduits can be found in Appendix B. Figure 7-5 shows an example for Pearl Rd. which demonstrates surcharging for the 5 Year design storm.

Figure 7-5: 5-Year 6-Hour Design Storm HGL on Pearl Rd



7.3 RECORDED FLOODING STORM ANALYSIS

Recorded flooding reports were provided by Neff and Associates for the April 19, 2017 and the June 30, 2017 storm events. These two events were outside the flow monitoring period but the calibrated hydraulic model could be run to allow comparison between simulation and observed data. Overall, the model predicted similar behavior as what was reported, with a good correlation between surcharged pipe locations and reported flooding locations. On streets where flooding was reported while no pipe surcharge is predicted, flooding may have been caused by saturated catch basins and consequential surface runoff.

Rainfall comparisons for the 5 year design storm and the recorded events can be reviewed in Section 3. The return frequency on April 19 was similar to the 25 year design storm while June 30 was comparable to the 5 year design storm, with a larger rainfall depth but slightly lower peak hourly intensity.

After model calibration, recorded rainfall data was collected from the NEORSD Brook Park rain gauge and imported into SWMM. The results were compared to areas of reported flooding for the April 19, 2017 and the June 30, 2017 storm events. Figures 7-6 and 7-7 show the surcharged conduits and the reported flooding parcels respectively for April 19 and June 30.

Main issues are found to occur along Pearl Rd. These hydraulic bottlenecks limit the hydraulic capacity throughout the arterial streets, causing high surcharge levels and surface flooding. Localized issues are further highlighted during the April 19 storm and results show that 66.5% of the system is either at capacity or limited by a downstream hydraulic condition. These occur along Meadowbrook Rd, Oakwood Rd, Beresford Ave, Parma Park Blvd, Orchard Blvd, Woodview Blvd, and Lawndale Dr.

The pipe coming from Nathan Hale Park does surcharge during the April 19 event but this is due to downstream hydraulic restriction. The results for June 30 are similar to the 5 Year design, which is discussed in the next section (Section 7.4).

Figures 7-6 and 7-7 illustrate the results from the April and June rain events.

Figure 7-6: April 19, 2017 Storm Results

Figure 7-7: June 30, 2017 Storm Results

7.4 5 YEAR DESIGN STORM ANALYSIS

The target level of service for this project is the 5-year 6-hour design storm. Results for this storm are similar to the flooding event that was recorded on June 30, 2017. Rainfall comparisons for the 5 year design storm and the recorded event can be reviewed in Section 3. The return frequency in June was comparable to the 5 year storm, with a larger rainfall depth but slightly lower peak hourly intensity.

After model calibration, the capacity results and hydraulic grade lines (HGL) were compared to determine areas of possible surface flooding. These results were compared to areas of reported flooding for the April 19, 2017 and the June 30, 2017 storm events. Figure 7-8 shows the surcharged conduits and the reported areas. As noted, the 5 year design storm is smaller than the two recorded flooding events.

Main capacity issues occur along Pearl Rd at the intersections of Maplewood Rd and Oakwood Rd. This is mainly due to large inflows along Pearl Rd travelling west and from flows traveling north along Maplewood Rd. Peak flows reached 38.8 MGD along Pearl Rd. and 44.54 MGD along Maplewood Rd.

Other sewer lines experiencing pipe capacity limitations during the 5 year storm are Maplewood Rd, Oakwood Rd, Greenleaf Ave, Orchard Blvd, as well as the very upstream end of Lawndale Dr. On the same streets, these system bottlenecks cause upstream pipe surcharge due to downstream hydraulic restrictions.

The pipe coming from Nathan Hale Park does not surcharge during the design storm even though it accounts for 10.4% of the total flow entering the system and has a peak flow of 22.41 MGD. Overall it is ranked 4th for flow contribution with Areas 1, 6, and 11 ranking ahead.

A flow ranking can be observed in Table 7-3 and a map of the tributary metering areas can be seen in Figure 7-9. Storm sewer profiles and HGLs for the 5-Year 6-Hour Storm can be seen in Appendix B.

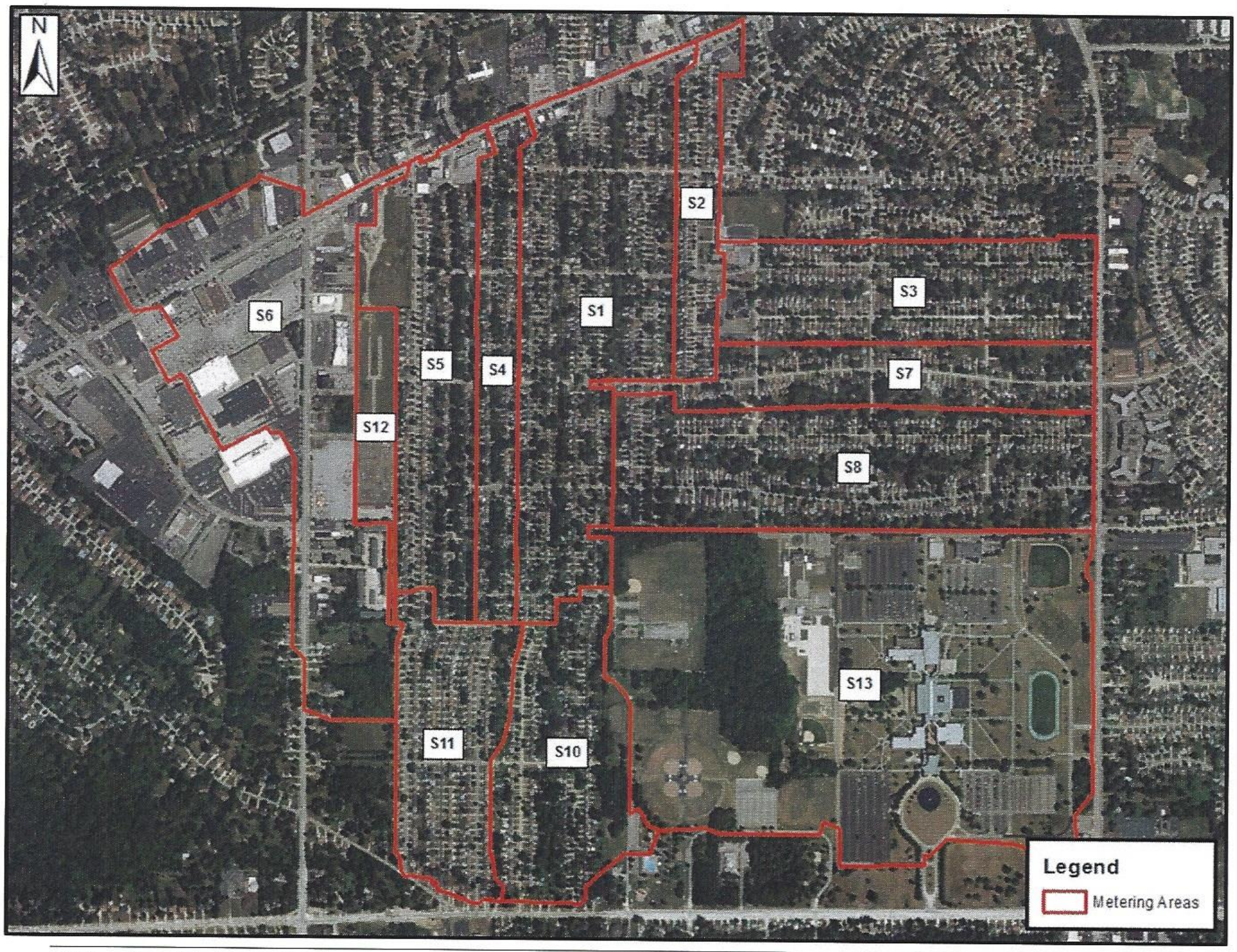
System outlets do not surcharge for the 5, 10, or 100-Year storm but downstream restrictions may cause further surcharging. Since these outlets are modeled as open outfalls, responses in the last few segments will be inaccurate.

Figure 7-8: Predicted Surcharged Conduits and Areas of Report Flooding

Table 7-3: Flow Ranking By Area

Flow Ranking		
Area	Flow (MGD)	% Total Flow
1	40.49	18.72%
6	39.05	18.05%
11	25.80	11.93%
13	22.41	10.36%
3	15.22	7.03%
7	14.63	6.76%
8	13.66	6.31%
5	12.55	5.80%
10	12.33	5.70%
12	10.35	4.78%
2	6.34	2.93%
4	2.13	0.99%
9	1.37	0.63%

Figure 7-9: Metering Tributary Areas

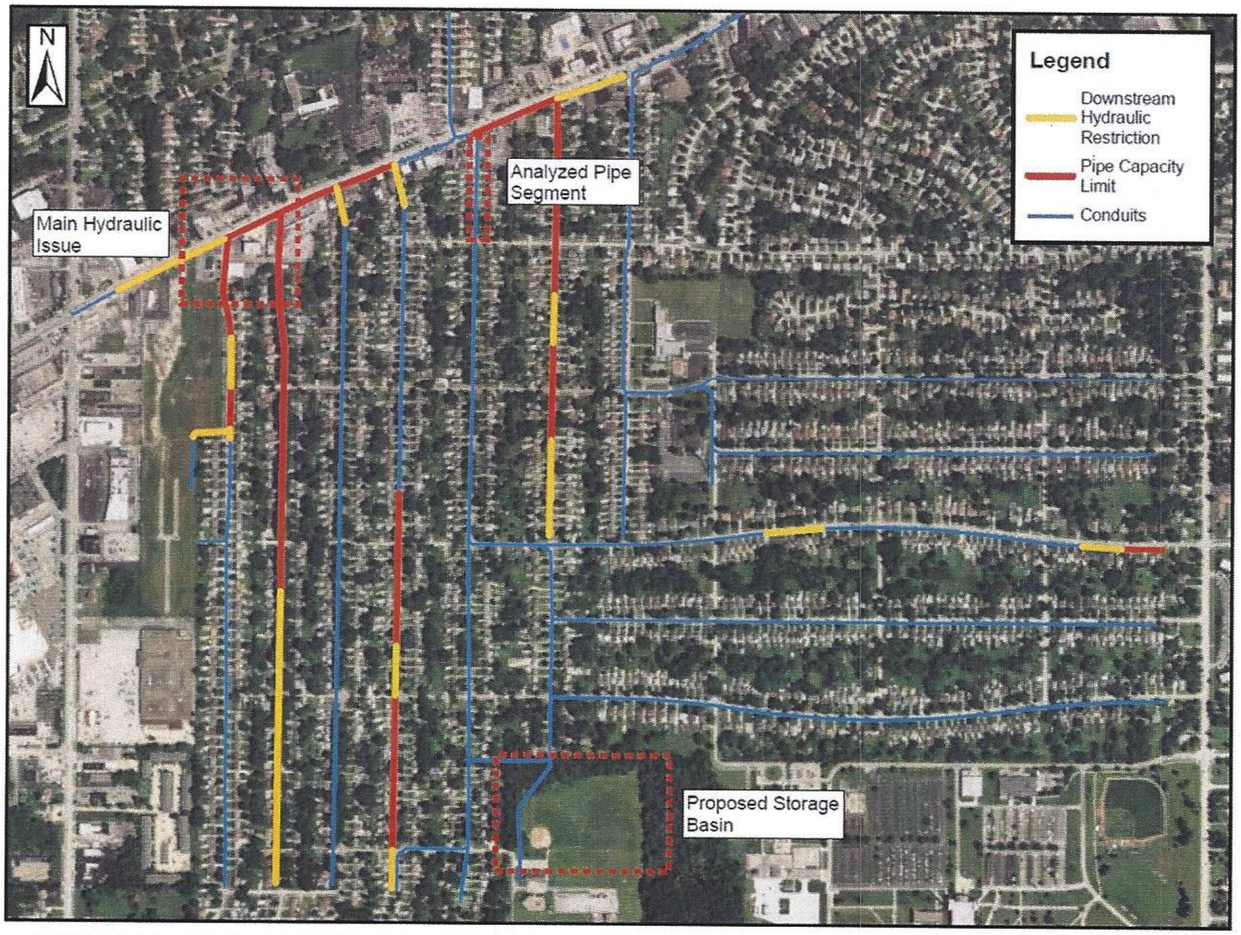


7.5 NATHAN HALE STORAGE BASIN

A proposed Storage Basin is located in Nathan Hall Park in the southeastern section of the project area. Flow Monitor 13 recorded flows from this area. Incoming flows are mainly transported through a swale whose tributary area is about 170 Acres. The exact tributary area was uncalculated but this estimate is based on flow matching through the hydraulic model. This area is of higher elevation than most of the storm system and ranks 4th among incoming flow rate (See Table 7-3). To assess storage basin impact, it was added into the model, and modeled flow data from conduit C1263 was analyzed. This conduit is downstream of Area 13 and is in a location where hydraulic issues are present. This location can be seen in Figure 7-10.

The proposed location has little effect on the hydraulic issues of the system. While total flow is reduced by over 6 million gallons, no hydraulic improvement is observed, at least for the 5-Year design storm. Although for the April storm, surcharging was reduced along Parma Park Rd. Another issue is that the main hydraulic problems are located in an area from where the flows cannot be easily re-routed, and is at an elevation 10-13 feet lower, than the proposed storage location. Figure 7-10 illustrates these issues.

Figure 7-10: Project Overview



8.0 RECOMMENDATIONS

8.1 ALTERNATIVE 1: UPSIZE HYDRAULICALLY LIMITED PIPES

The model predicted that the Nathan Hale Park storage provided minimal relief to the system. One possible solution for increasing hydraulic efficiency would be to increase pipe sizes in areas that restrict flow due to capacity limitations. To accomplish this task an alternative model was created and pipe segments that restricted flow were incrementally increased until the hydraulic issues were resolved. Overall, thirty-seven (37) pipes needed to be upsized to completely eliminate surcharging during the 5 year 6 hour design storm. A summary of results is listed in Table 8-1 includes the initial capacity for each pipe and a new capacity of the upsized pipe. Figure 8-1 illustrates which pipe segments were increased.

Figure 8-1: Location of Increased Pipes

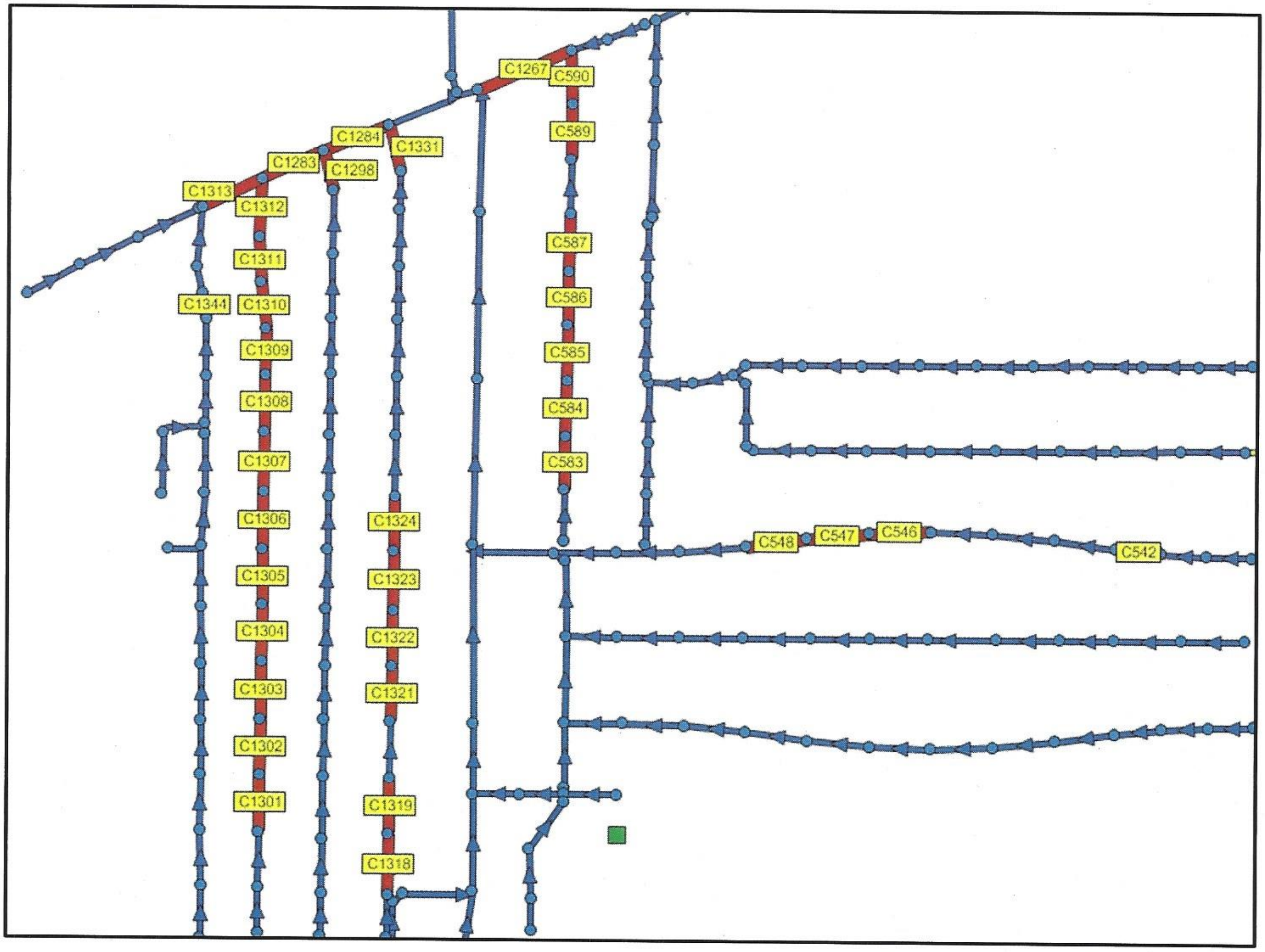


Table 8-1: Increased Pipe Capacity Summary

Name	Shape	Original Size (ft)	Upgraded Size (ft)	Slope	Mannings	Original Full Flow (MGD)	New Full Flow (MGD)	Capacity Increase (MGD)
C1283	Rectangle	5x6	6x6	0.002	0.018	76.28	110.58	34.30
C1284	Rectangle	5x6	6x6	0.002	0.020	60.64	100.27	39.63
C542	CIRCULAR	1.75	2	0.008	0.015	7.77	11.09	3.32
C546	CIRCULAR	2.5	2.75	0.004	0.015	15.18	19.58	4.39
C547	CIRCULAR	2.5	2.75	0.005	0.015	16.07	20.72	4.65
C548	CIRCULAR	2.5	2.75	0.001	0.015	5.96	7.68	1.72
C583	CIRCULAR	1	1.5	0.003	0.015	0.99	2.92	1.93
C584	CIRCULAR	1.25	2	0.003	0.015	2.01	7.04	5.03
C585	CIRCULAR	1.5	2	0.003	0.015	3.27	7.04	3.77
C586	CIRCULAR	1.67	2	0.003	0.015	4.35	7.04	2.69
C587	CIRCULAR	1.67	2.5	0.003	0.015	4.35	12.75	8.40
C589	CIRCULAR	2.5	3.5	0.003	0.015	11.34	27.82	16.48
C590	CIRCULAR	2.5	3.5	0.003	0.015	11.90	29.18	17.28
C1267	CIRCULAR	2.5	3.5	0.004	0.015	14.22	34.88	20.66
C1298	CIRCULAR	2.5	4	0.005	0.015	16.75	58.65	41.90
C1301	CIRCULAR	1.25	1.5	0.003	0.015	1.95	3.18	1.22
C1302	CIRCULAR	1.25	2	0.013	0.015	4.08	14.28	10.20
C1303	CIRCULAR	1.5	2	0.003	0.015	3.19	6.86	3.68
C1304	CIRCULAR	1.5	2.5	0.003	0.015	3.17	12.38	9.21
C1305	CIRCULAR	1.75	2.5	0.003	0.015	4.80	12.42	7.62
C1306	CIRCULAR	1.75	2.5	0.003	0.015	4.85	12.55	7.70
C1307	CIRCULAR	1.75	2.5	0.003	0.015	4.77	12.34	7.57
C1308	CIRCULAR	2	3	0.003	0.015	6.78	20.00	13.22
C1309	CIRCULAR	2	3	0.003	0.015	6.90	20.34	13.44
C1310	CIRCULAR	2.25	3.5	0.003	0.015	9.32	30.27	20.95
C1311	CIRCULAR	2.25	3.5	0.003	0.015	9.40	30.53	21.13
C1312	CIRCULAR	2.25	3.5	0.004	0.015	10.87	35.32	24.45
C1318	CIRCULAR	1	1.5	0.004	0.015	1.21	3.58	2.36
C1319	CIRCULAR	1.25	1.5	0.004	0.015	2.20	3.58	1.38
C1321	CIRCULAR	1.5	2	0.004	0.015	3.71	7.98	4.28
C1322	CIRCULAR	1.5	2	0.003	0.015	3.42	7.37	3.95
C1323	CIRCULAR	1.67	2	0.004	0.015	4.77	7.72	2.95
C1324	CIRCULAR	1.67	2	0.004	0.015	4.90	7.93	3.03
C1331	CIRCULAR	3	3.5	0.006	0.015	28.16	42.48	14.32
C1344	CIRCULAR	4.5	5.5	0.001	0.015	30.18	51.53	21.36

There was one pipe that was increased that is not included in Table 8-1. Pipe segment C1313 which is located on Pearl Rd between Maplewood Rd and Oakwood Rd was increased from a 60" circular pipe to a 5' x 6' rectangular pipe. This increased capacity by 65.42 MGD and greatly improved hydraulic issues along the two intersecting streets.

8.2 ALTERNATIVE 2: RELOCATE STORAGE BASIN SITE

Since the original storage basin site at Nathan Hale Park is relatively far from the hydraulic bottleneck at Pearl Rd and Meadowbrook Rd an alternative would be to build a storage basin closer to the issue. An aerial review shows that a vacant area of land, approximately Nineteen (19) acres in size, is located just south of Pearl Rd between W130th and Maplewood Rd. Figure 8-3 shows the location.

Figure 8-3: Secondary Location for Storage Basin

Considering that Nathan Hale Park is approximately six (6) acres in size, this location has more than enough area to accommodate a storage basin. The legalities and ownership of this land is not currently know and would need to be investigated.

8.3 RECOMMENDATION 1: CATCH BASIN MAINTENANCE

Flow metering analysis and modeling results indicate that subcatchment flows cannot be sufficiently captured by the current collection system. This is indicated by overly predicted peaks during the October 8th storm event. Since the model does not include limitations from under-sized or under-performing catch basins, flows are directly routed to the main storm sewer through inlet nodes which cause higher peak flows to be predicted.

This result combined with reported surface flooding in areas that were unconfirmed by the model suggests that part of the issue could be the cleanliness or performance of the local catch basins. Inspection and cleaning of catch basins along with re-monitoring would assist in confirming this possibility.

8.4 RECOMMENDATION 2: HYDRAULIC MODEL EXPANSION

The current hydraulic model does not consider downstream restrictions past flow meters one and two. This could have considerable effects on the systems response to the 5 year design storm since the flow is currently exiting the system freely through an open outfall. There are two separate storm sewer systems downstream on the current hydraulic model that both discharge into Big Creek. After reviewing the area, model expansion would include approximately eight (8) additional flow monitors, with the reinstallation of flow meters 1 and 2, and would sufficiently capture the remainder of the storm sewer system. Expanding the hydraulic model is recommended since the current model does not predict surcharging along Blossom Avenue or Barrington Boulevard for historical flooding events or for the 5 year design storm. Also, the response along Parma Park Boulevard under predicts the expected results and is an area of main concern since flows from Nathan Hale Park are conveyed though this storm sewer.

9.0 CONCLUSION

A comprehensive hydraulic model of the City of Parma Heights south-west storm sewer system was built to characterize and address the City's recurrent flooding issues. The model was developed and successfully calibrated based on rainfall and flow monitoring data collected between August and November 2017. October 8 storm event had a 1 to 2 year recurrence. Peak flows were generally over predicted by the model because, during this storm, catch basins could not absorb the street runoff and caused flow restrictions, not represented in the model.

The model was used to analyze the existing sewer system's performance under various wet weather conditions, including three design storms (5, 10, and 100 year recurrence, 6 hour duration) and two historical events that both occurred in the spring of 2017. The April 19, 2017 event was close to a 25 year event while the June 30 event compared to a 5 year event. Main pipe capacity limitations during the 5 year storm were predicted along Pearl Rd at the intersections of Maplewood Rd and Oakwood Rd, mainly due to large inflows along Pearl Rd travelling west and from flows traveling north along Maplewood Rd. Other bottlenecks include Maplewood Rd, Oakwood Rd, Greenleaf Ave, and Orchard Blvd. These restrictions create pipe surcharge in the upstream sections. Manhole flooding was predicted to occur at 2 locations. For the 10 and the 100 year storms, hydraulic conditions were worse, with more pipe surcharge and respectively 8 and 24 flooding locations. The model predicted good correlation between surcharged pipe locations and reported flooding locations during the April 19 and June 30, 2017 storm events. On streets where flooding was reported while no pipe surcharge is predicted, flooding may have been caused by surface runoff due to saturated catch basins, in the project area, but also from outside streets east of York Rd.

The proposed Nathan Hale Park storage was modeled, showing no significant resolution of surcharge and flooding issues in the system for the 5 year storm. Being located above most of the storm sewer network, the storage could only capture the overland flows from the Park. The main alternative to eliminate pipe surcharge during the 5 year 6 hour design storm is to upsize up to 37 pipes. Priority should be given to pipes on Pearl Rd, downstream bottleneck, and Maplewood Rd, Oakwood Rd and Orchard Blvd where most flooding locations are reported and predicted. Another alternative for improvement includes the utilization of the vacant land located south of Pearl Rd and west of Maplewood Rd for flow diversion and storage. It is also generally recommended to inspect and maintain catch basins so as to mitigate the risk of saturation, runoff, and storm water infiltration at the parcels.

It is recommended that the hydraulic model be expanded to include downstream tributary areas. This will increase the accuracy of alternative developments and prove a better understanding of possible hydraulic restrictions. This expansion would include the installation of eight (8) flow meters.