EXHIBIT B

Storm Sewer Evaluation Final Report

City of Parma Heights Storm Modeling – Phase II

March 18, 2019

AECOM Project No. 60554795

Prepared for

CITY OF PARMA HEIGHTS

AECOM

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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. Fourteen (14) flow meters, two (2) rain gauges and one (1) stream gauge were deployed during the period of August 19, 2018 to October 14, 2018 (56 days).

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.

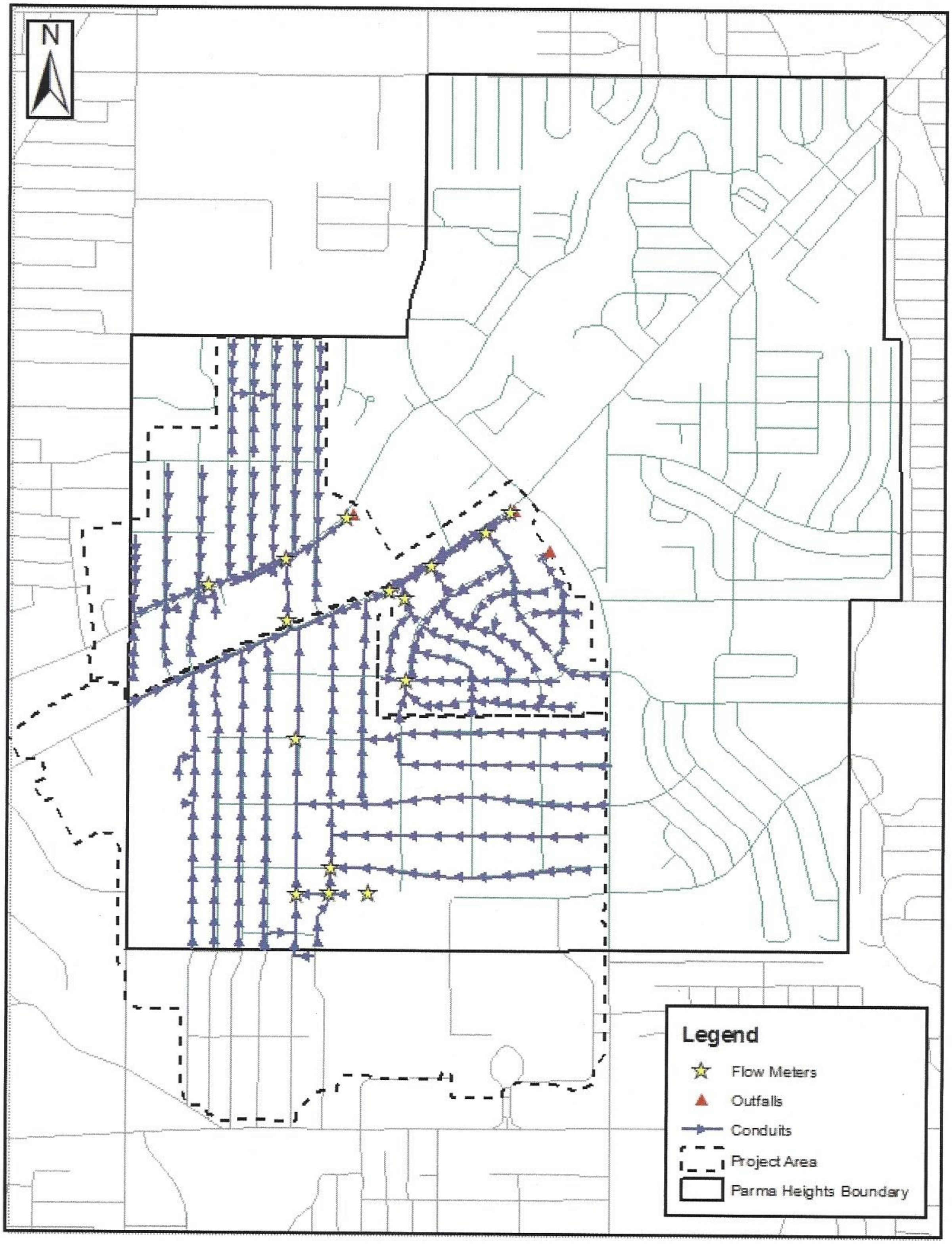
AECOM proposed conducting a storm sewer system flow monitoring and modeling project in the area with flooding complaints. The storm sewer monitoring would quantify the flooding and provide data that could be used in a hydraulic model.

An initial study (Phase I) of the area was limited to the storm sewers south of Pearl Road, between West 130th and York Rd. Upon model calibration, a hydraulic analysis was conducted and it was determined that the model was under predicting downstream conditions of storm system tributary to Big Creek. AECOM suggested expanding the model to include all storm sewers in the system to improve hydraulic accuracy. Phase II was proposed and approved by City Council in May 2018, in August 2018 AECOM began expanding the model and metering the new areas.

2.1 PURPOSE

The purpose of this report is to summarize the findings of the storm sewer modeling for Phase II. This report documents 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 2018. A total of fourteen (14) flow meters, two (2) rain gauges and one (1) stream gauge were deployed during the monitoring period.

There were four (4) meters re-installed to Phase I metering locations to capture flows from the Phase I tributary areas. These meters were re-named for Phase II but are related to their Phase I names in Table 3-1 below.

Table 3-1: Re-Installed Meter Locations

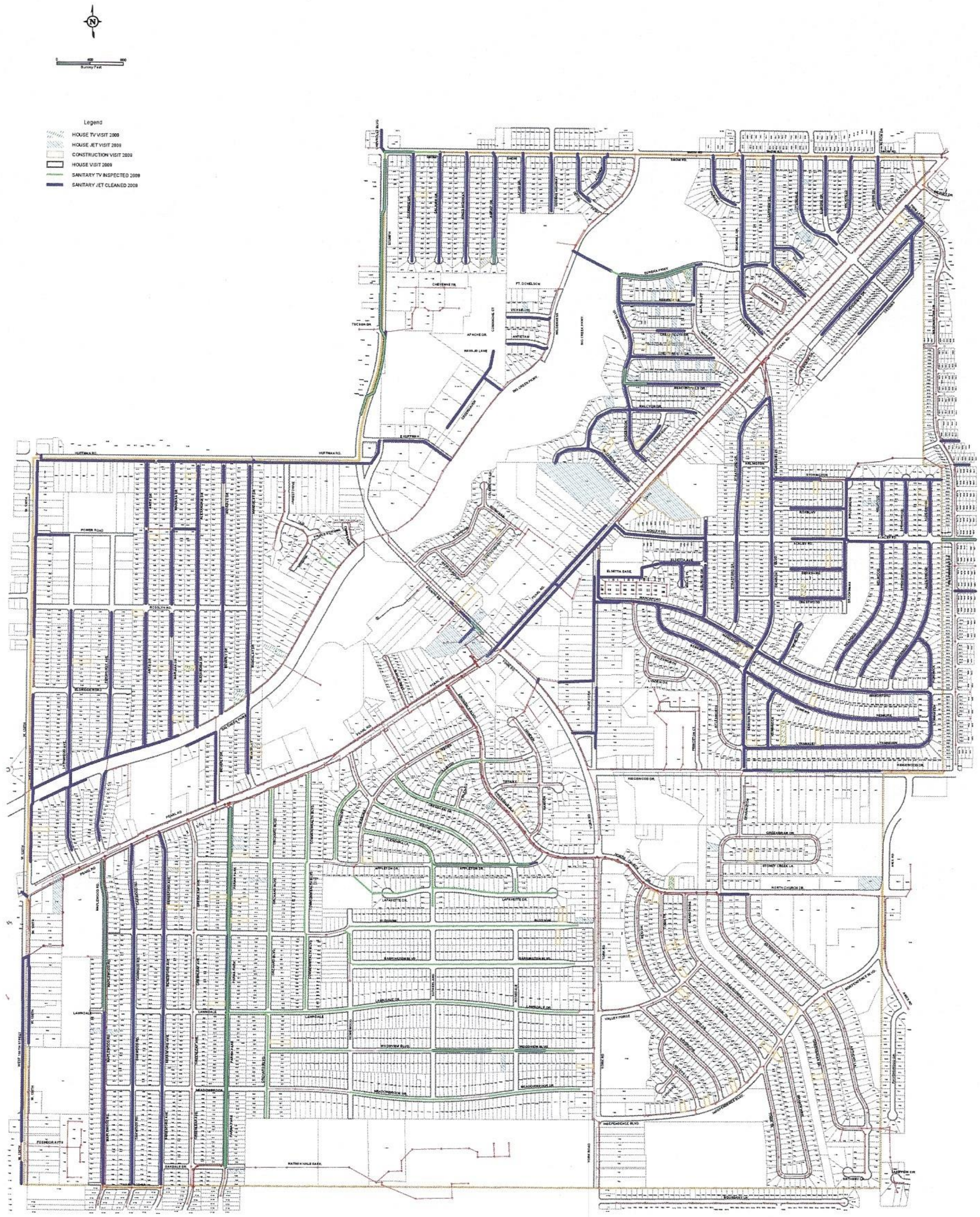
Phase I	Phase 2
FM 1	FM 11
FM 2	FM 12
FM 9	FM 10
FM 13	FM 14

Two (2) of the fourteen (14) flow meters were installed in the original Phase I area to provide additional information and to confirm storm sewer response in areas where the hydraulic model lacked information. The flow meters are FM 9 and FM 10.

FM 15 is the stream gauge.

The locations of these metering instruments are depicted on Figure 3-1.

PARMA HEIGHTS SERVICE MAP



CUYAHOGA COUNTY
SANITARY ENGINEER
ROBERT C. KLAIBER Jr., P.E., P.S.

NOTE: THIS MAP IS FOR REVIEW PURPOSES ONLY. MAINLINES MAY HAVE BEEN JET CLEANED OR TV INSPECTED MULTIPLE TIMES OVER THE DATES SHOWN, THOUGH SYMBOLIZED ON THE MAP ONLY ONCE. ADDRESS RECORDS ARE POSSIBLY UNDER-REPRESENTED.

Figure 3-1: Flow Monitoring and Rain Gauge Site Locations, August 2018

3.1 FLOW MONITORING

There were 14 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.

The overland flow from Nathan Hale Park was measured by meters FM14 (main flow), and FM13 (overflow). Meters FM 1 and FM 6 were located on the two system outlets. Table 3-3 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 3-2 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 3-2 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 FM12 during two storm events due to meter malfunction.

The table also shows that pipe surcharge was only observed at meters FM7 for one event only, on October 6, 2018. This event was comparable to a 4-6 month storm.

Table 3-2: Data Quality Summary

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

Start Date	Storm Duration (hr)	Rain Total (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	FM-14	FM-15
8/21/2018 16:25	5.1	0.32	0.24															
8/29/18 15:20	1.1	0.61	0.60															
9/8/18 1:10	82.3	2.00	0.20															
9/26/18 2:20	7.9	1.17	0.54															
10/6/2018 4:30	25.6	1.33	0.63															

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

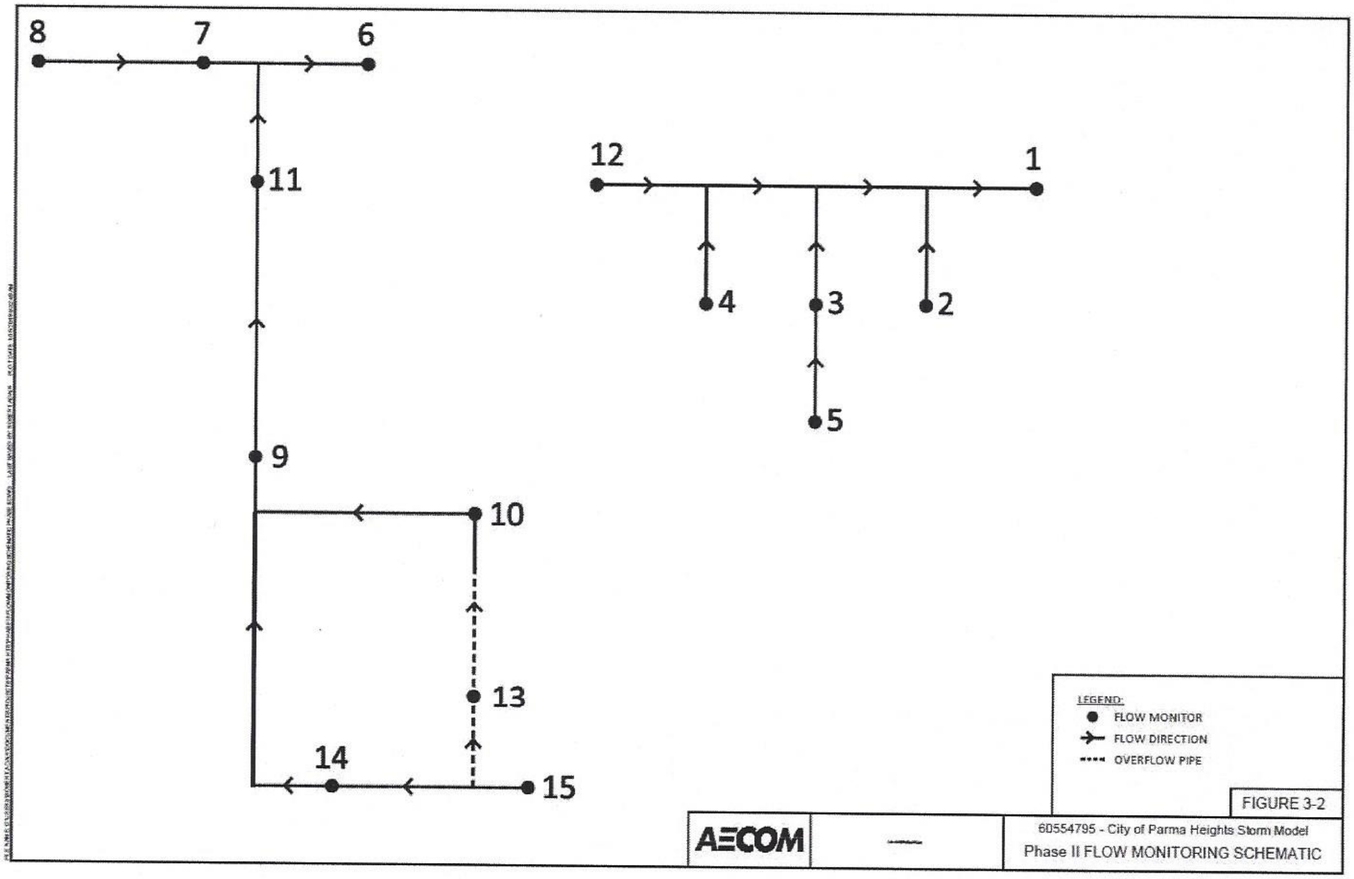
Year: Fall 2018
Location: Parma Heights Phase II Storm Sewer Flow Monitoring

FM Site #	Street/Location	Meter SN #	Monitored Pipe Size	US Pipe	DS Pipe	Site Comments:
1	6355 Pearl Rd.	2234	60" US	60"	60"	South curb lane near driveway
2	Kingsdale Blvd. south of Pearl Rd.	2692	33" DS	33"	33"	Canterline of road at Appaartment driveway
3	6425 Pearl Rd	2232	48"	48"	48"	Harry Buffalo Drive, SW pipe
4	Rochelle Blvd. south of Pearl Rd.	2688	24" US	24"	24"	Street
5	Appleton Dr. at Sherborn Rd.	2695	24" SW US	24"	24"	Street, flow from SW pipe
6	6476 Big Creek Parkway	2236	120" x 55"	120" x 55"	120" x 55"	Meter in CB at St. John Bosco drive
7	Beverly and Big Creek (West Pipe)	2690	130" x 55"	130" x 55"	130" x 55"	Center median of Big Creek Pkwy at Beverly
8	Anita Dr. and Big Creek Pkwy.	2199	48" US	48"	48"	Street
9	Parma Park Blvd. and Blossom Ave.	2689	66" US	66"	72"	Street
10	Orchard Blvd. and Meadowbrook Dr.	2694	33" US	30"	42"	Street, intersection, East pipe
11	Beverly north of Pearl Rd.	2699	130" x 62"	130" x 62"	130" x 62"	Street
12	6475 Pearl Rd.	2231	48" US	48"	54"	South Sidewalk near dry cleaners
13	7015 Orchard Blvd.	2218	30" US	30"	30"	Street
14	7022 Parma Park Blvd.	2697	48" US	48"	48"	Street
15	Creek behind 12091 Meadowbrook	Echo 510				Echo nailed to tree near creek, Level only

Rain Gauge Sites

Site #	Street/Location		Comments:
PHCCC_RG1	CCC West Public Safety Training	372260	Homewood Ave	East roof of building
PHPPES_RG2	Parma Park Elementary School	372241	6800 Commonwealth Blvd.	Roof

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 southern rain gauge, known as RG-1, was located at CCC West Public Safety Training; the northern rain gauge, known as RG-2, was located at Parma Park Elementary 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.

Two NEORSD rain gauges were used to collect rainfall data from August 6th, 2018 which had recorded flooding in the project area. They are the Brook Park and Parma rain gauges.

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 1.33" of rainfall. In all, there were fourteen (14) events recorded during the flow monitoring period.

Figure 3-3 compares the recorded rainfall events, and the large storm event that occurred on 8/6/2018 that produced flooding, 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. There were three (3) events with a 4 to 6-month recurrence interval. The maximum event occurred on October 6, 2018. The dates for analyzed storms are listed in full in Table 3-4.

The values for RG1 were primarily used in the data analysis and modeling because the flow meter data responded better to rainfall recorded by that rain gauge. Figure 3-3 and Table 3-4 display values for this rain gauge.

Figure 3-3: Rainfall Recurrence Intervals (RG1)

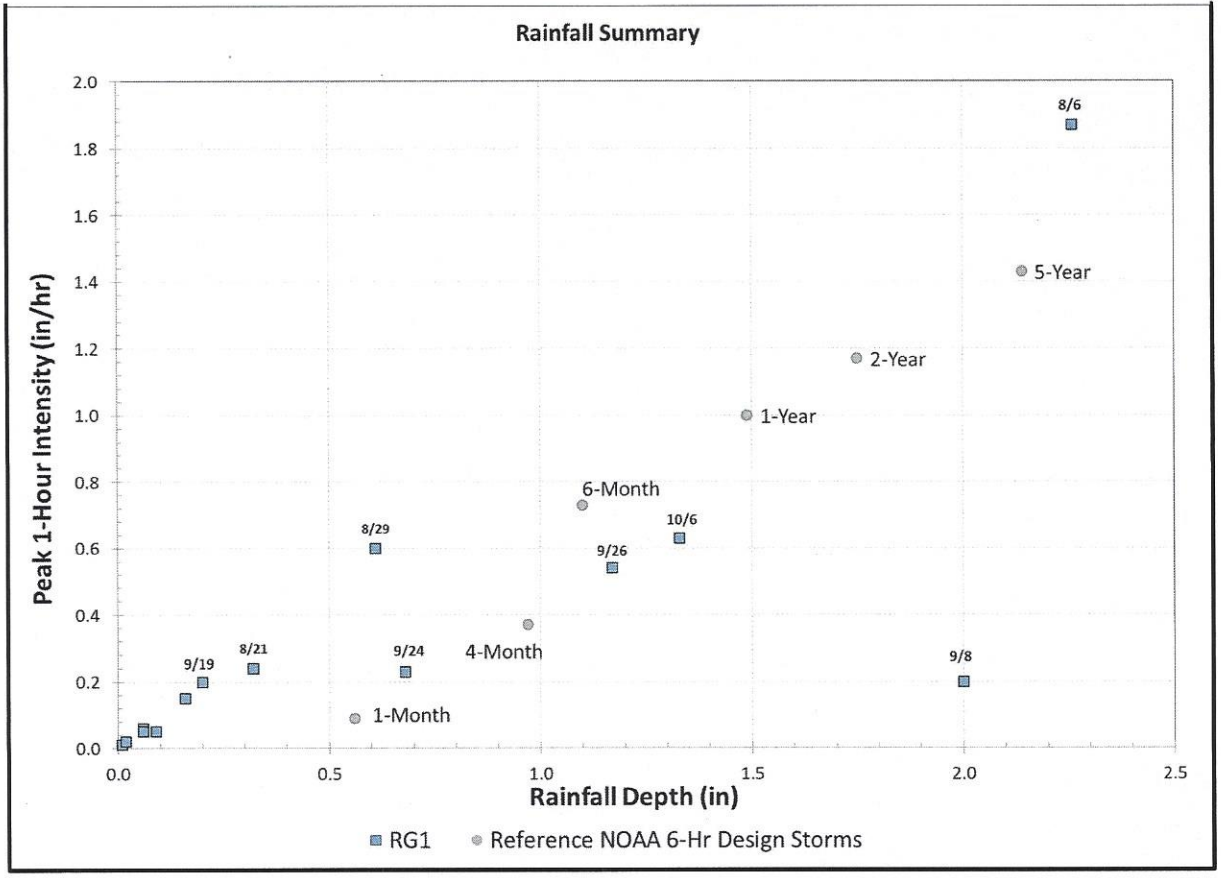


Table 3-4: Analyzed Storm Events (RG1)

Statistical Summary of Storm Events					
#	Start Date/Time	End Date/Time	Duration (hr)	Peak Hourly Intensity (in/hr)	Total Rain (in)
1	8/21/2018 16:25	8/21/2018 21:30	5.08	0.24	0.32
2	9/6/2018 8:10	9/6/2018 9:15	1.08	0.15	0.16
3	9/8/2018 1:10	9/11/2018 11:25	82.25	0.2	2
4	9/24/2018 16:05	9/25/2018 4:00	11.92	0.23	0.68
5	9/26/2018 2:20	9/26/2018 10:15	7.92	0.54	1.17
6	10/6/2018 4:30	10/7/2018 6:05	25.58	0.63	1.33
	8/6/2018 23:40	8/7/2018 5:20	5.67	1.87	2.26

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. The model for Phase II was built by adding to the Phase I model.

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. Information obtained from these record drawings included:

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

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: 111,735 feet of pipe and 441 manholes are referenced in the model; this includes the original Phase I model and the Phase II expansion. Pipe diameters, or heights for box culverts, range from 12 to 60 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 from Phase I.

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 14-1, originally 13-2, and is 209.7 acres. The smallest subcatchment is located in area 4-2 and is 1.01 acres. Acreage for most of these areas was calculated using GIS.

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 15.67% to 75.33%. 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:

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

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 0.11% to 4.17%.

Seasonal Variations

The model accounts for seasonal variances by using variable evaporation rate constants and a groundwater modulator. Groundwater effect was not applied to Phase II. For more details on groundwater effects in the hydraulic model refer to the **Storm Sewer Evaluation Report** submitted on December 29, 2017.

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.

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

  For example: "C120" would be the 120th conduit created during Phase I model creation. "120" would be the 120th conduit created during Phase II model expansion.

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

  For example: "J15" would be the 15th junction created during Phase I model build out. "15" would be the 15th junction created during Phase II model expansion.

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'.

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 and 4-3 show an overview of the modeled system with each pipe section labeled with its diameter. Figure 4-2 displays the Phase I system and Figure 4-3 displays the Phase II system.

Table 4-2: Modeled Pipe Details

Pipe Details		
Size	Number	Length
Circular		
12"	72	17143'
15"	45	11048'
18"	43	11301'
20"	11	3401'
21"	37	7402'
24"	40	10601'
27"	45	10127'
30"	34	8370'
33"	11	3034'
36"	23	5572'
42"	15	3165'
45"	1	21'
48"	21	5686'
54"	23	6597'
60"	5	2002'
68"	1	911'
72"	2	1575'
Box		
4' x 10'	1	173'
4.5' x 10'	5	1322'
5' x 6'	3	919'
5' x 11'	5	1367'

Figure 4-2: Phase I Modeled System Pipe Diameters

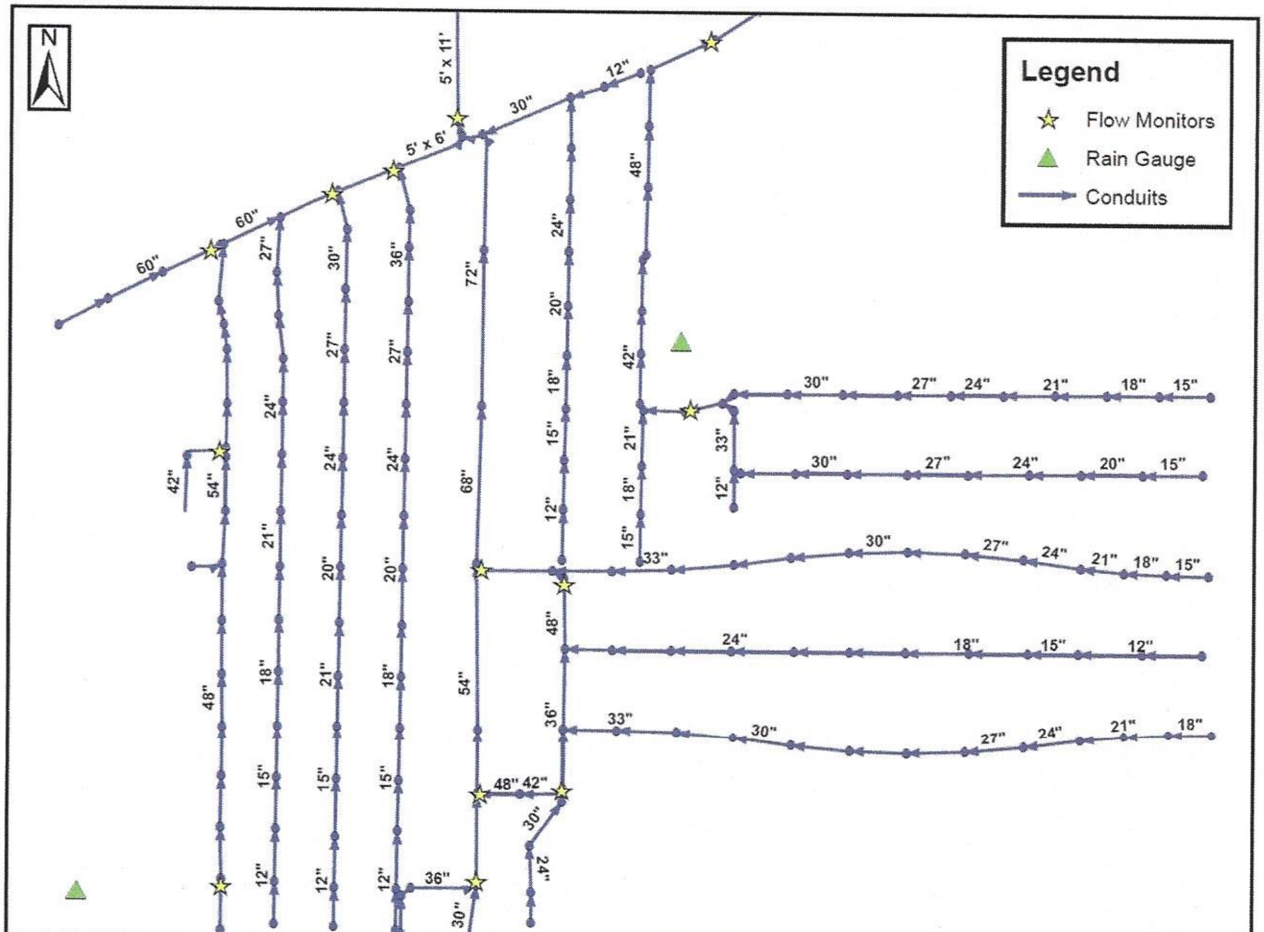
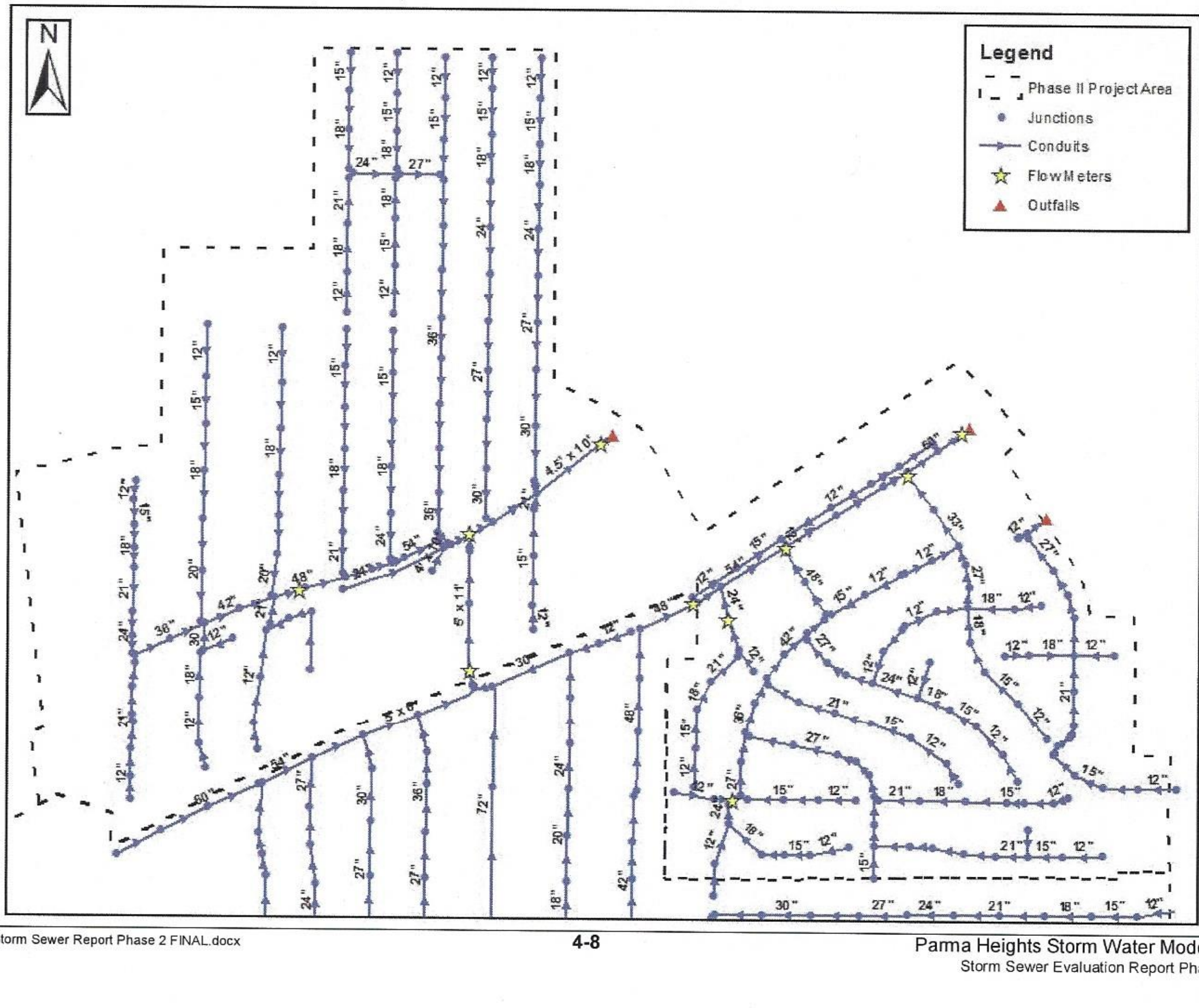


Figure 4-3: Phase II Modeled System Pipe Diameters



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

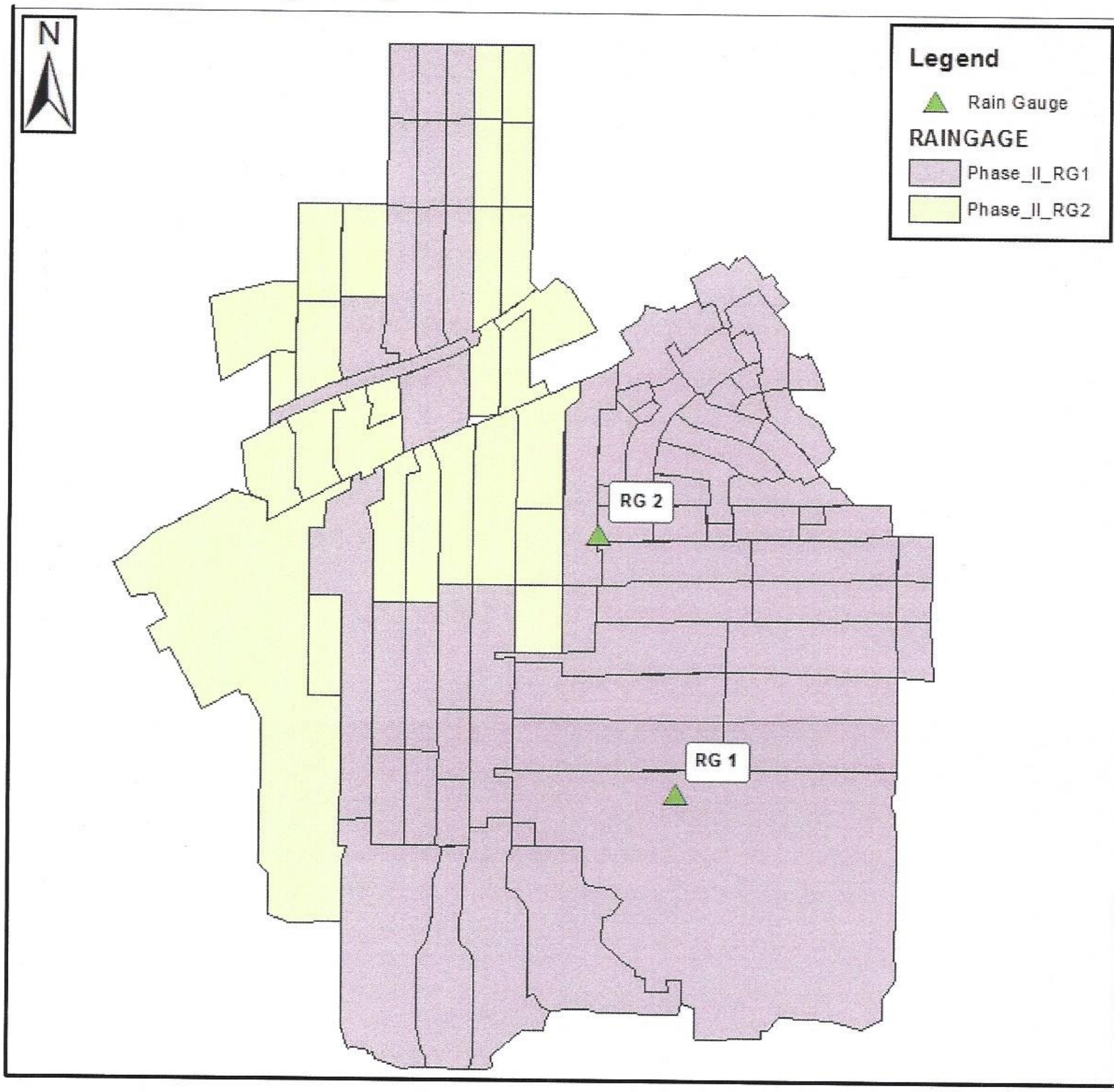
Observed rain events were simulated in PCSWMM by utilizing its time series function. During the monitoring period, two rain gauges were installed within the project area; so two different rain gauge time series were created. The storm events were reproduced based on the 5-min intensities recorded at 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.

Most subcatchments were assigned RG1 to match flow responses in the hydraulic model, since RG1 has higher rainfall intensity for some storms. Peak flows were undervalued in some areas when using data from RG2.

Table 5-1: Rain Gauge Characteristics

Rain Gauge	# of Subcatchments	Area of Coverage (acres)	Total Recorded Rainfall	Max Rainfall Intensity (in/hr)
RG1	66	990	6.723	0.63
RG2	35	596	6.296	0.58

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	73.13
2	39.37
3	34.54
4	33.29
5	21.96
6	33.49
7	37.16
8	33.08
9	28.05
10	30.44
11	28.05
12	34.98
13	-
14	6.01
15	-

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.

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 are issues that prolonged calibration efforts and can relate to inconsistencies in model results.

Unknown Factors

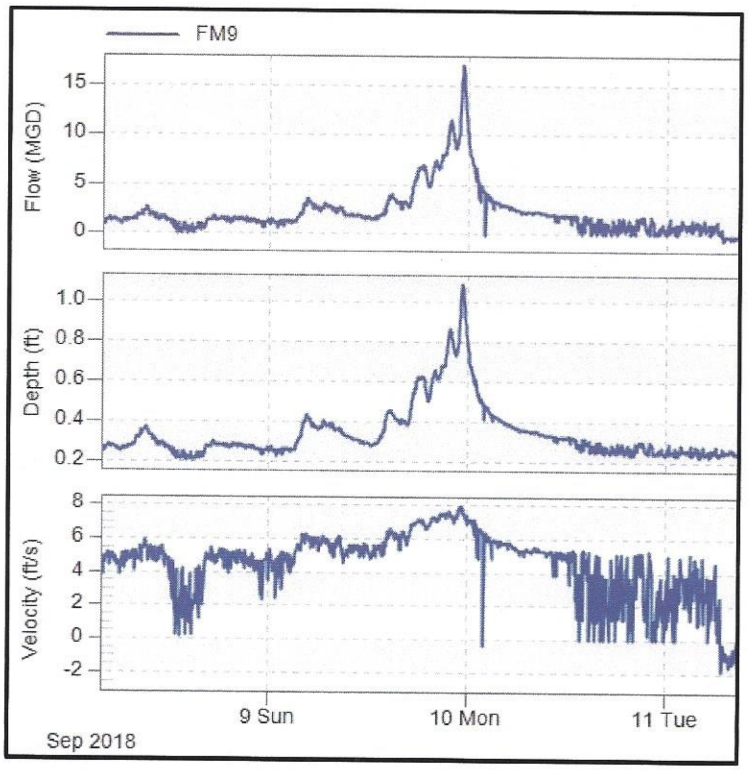
Other unknown factors that can affect calibration include blockages or other flow restrictions, relief pipes, undocumented overflows, silt build-up, inaccurate record drawings, or other sources of surface flooding that travel to the project area.

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.

Figure 5-4 is an illustration of questionable velocity for meter FM9, but shows that the flow data (peak flow, general hydraulic pattern) is not severely affected.

Figure 5-2: FM 9 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 6 various storm events throughout the monitoring period. This analysis includes the five calibration events along with 4 additional storms varying in intensity (Table 5-3). The model was also run outside the flow monitoring period, for the major event that occurred in August 2018.

Model verification results can be located in Appendix A.

Table 5-3: Model Calibration Storm Events

Statistical Summary of Storm Events					
#	Start Date/Time	End Date/Time	Duration (hr)	Peak Hourly Intensity (in/hr)	Total Rain (in)
1	8/21/2018 16:25	8/21/2018 21:30	5.08	0.24	0.32
2	9/6/2018 8:10	9/6/2018 9:15	1.08	0.15	0.16
3	9/8/2018 1:10	9/11/2018 11:25	82.25	0.2	2
4	9/24/2018 16:05	9/25/2018 4:00	11.92	0.23	0.68
5	9/26/2018 2:20	9/26/2018 10:15	7.92	0.54	1.17
6	10/6/2018 4:30	10/7/2018 6:05	25.58	0.63	1.33
	8/6/2018 23:40	8/7/2018 5:20	5.67	1.87	2.26

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.

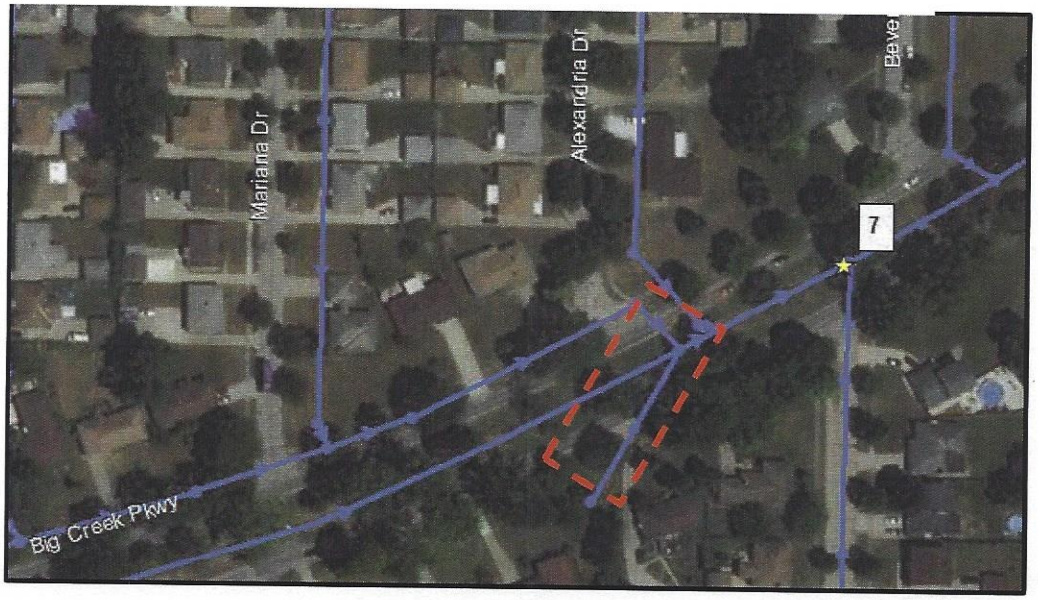
Structural health of the system - Structural health of the system was accounted for when calibrating groundwater effect into the system. Areas with high infiltration rates are considered less structurally stable than areas with low infiltration rates. .

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 for Area 7 includes a 4' x 10' storm sewer that travels from the southwest and connects to the Big Creek Parkway storm sewer. The mapping for this storm sewer is limited but it is believed to capture flows from Liberty Ford (6600 Pearl Rd).In general, the flow values for Area 7 were higher than expected and resulted in raising the overall impervious percent amount by approximately 233% from a mean average of 37.16% to 86.87%. This was done to increase the modeled flows to meet recorded meter data for calibration. It is likely that the 4' x 10' storm sewer is providing most of this additional flow but without an accurate travel path the subcatchment area is limited. Distributing these additional flows throughout Area 7 creates misleading results when simulating the 6-hour design storms such as surcharging, flooding, and capacity issues.

Figure 6-1: 4'x10' Storm Sewer Location



A stream gauge (FM15) was installed upstream of FM 14 to capture levels of a tributary stream to estimate the streams response during storm events and to assist in predicting overland flow. See ***Appendix C*** for more details.

Unfortunately, during the monitoring period there were no storm events that created a significant response to the stream. The average depth was approximately 1.53 feet and the max depth during the monitoring period was approximately 2.36 feet, a difference of 0.83 feet or ~10 inches. Local evidence, flooding reports, and previous meter data suggest this stream creates overland flow to Meadowbrook Drive, but with no significant data recorded the impacts cannot be accurately predicted.

6.1.3 *FUTURE IMPROVEMENTS*

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

7.0 HYDRAULIC ANALYSIS

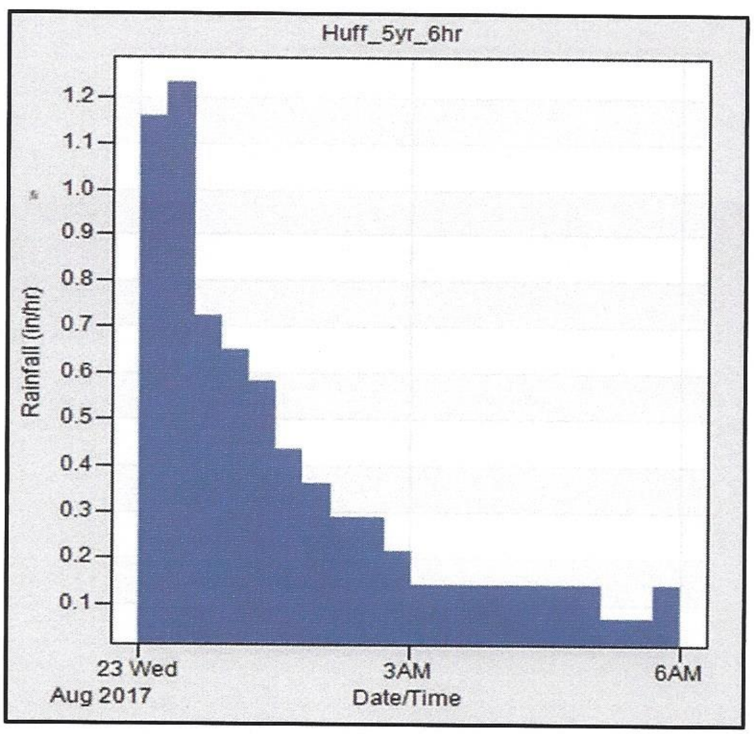
7.1 DESIGN STORMS

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 1586.50 acres or about 2.48 square miles.

Table 7-1: Design Storm Values

Design Storms				
Storm	ARI (years)	Duration (hrs)	Depth (in)	Peak Intensity (in/hr)
#1	5	6	2.17	1.23
#2	10	6	2.56	1.451
#3	25	6	3.29	1.794

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 or yellow.
- 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

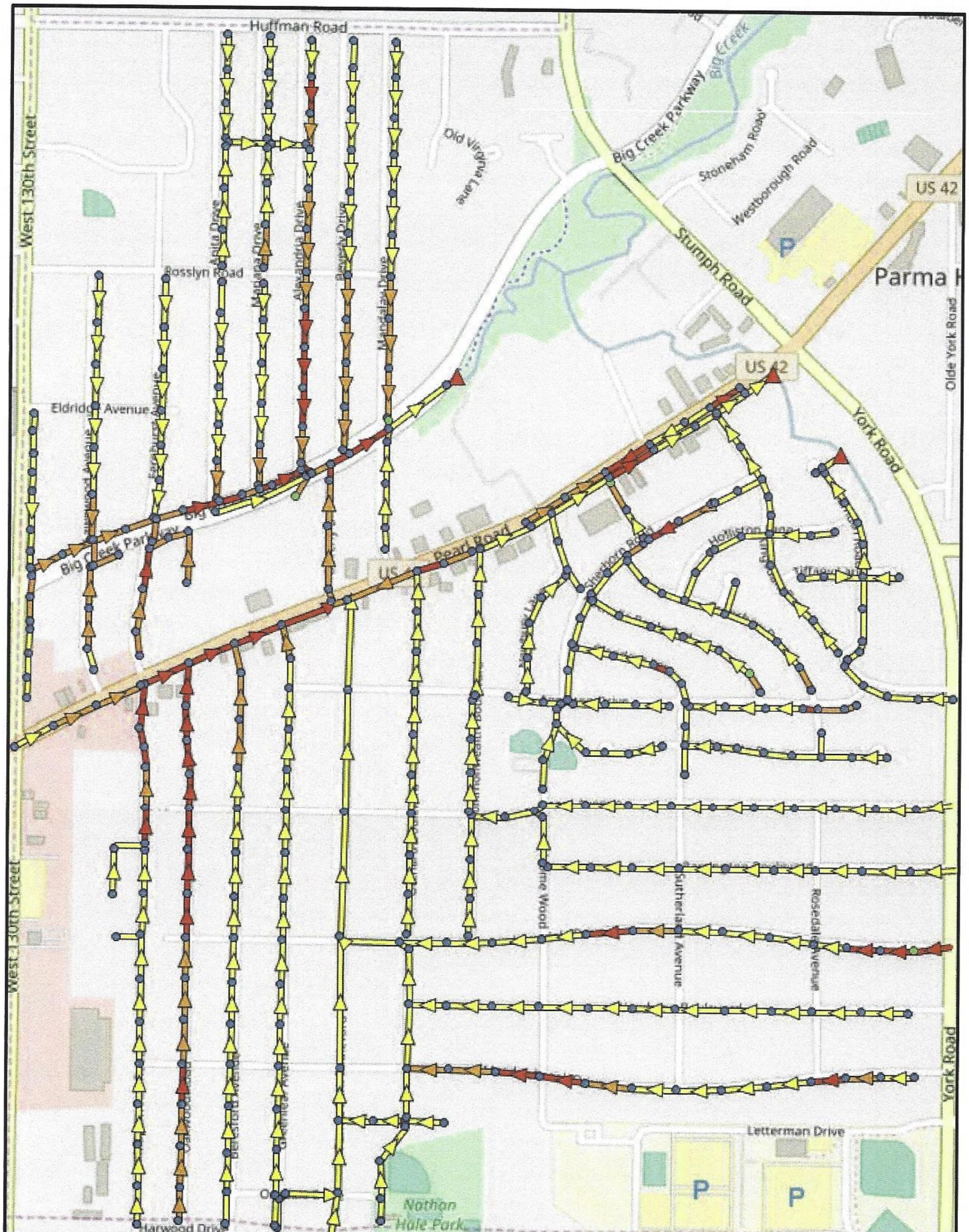


Figure 7-3: 10-Year 6-Hour Storm Results

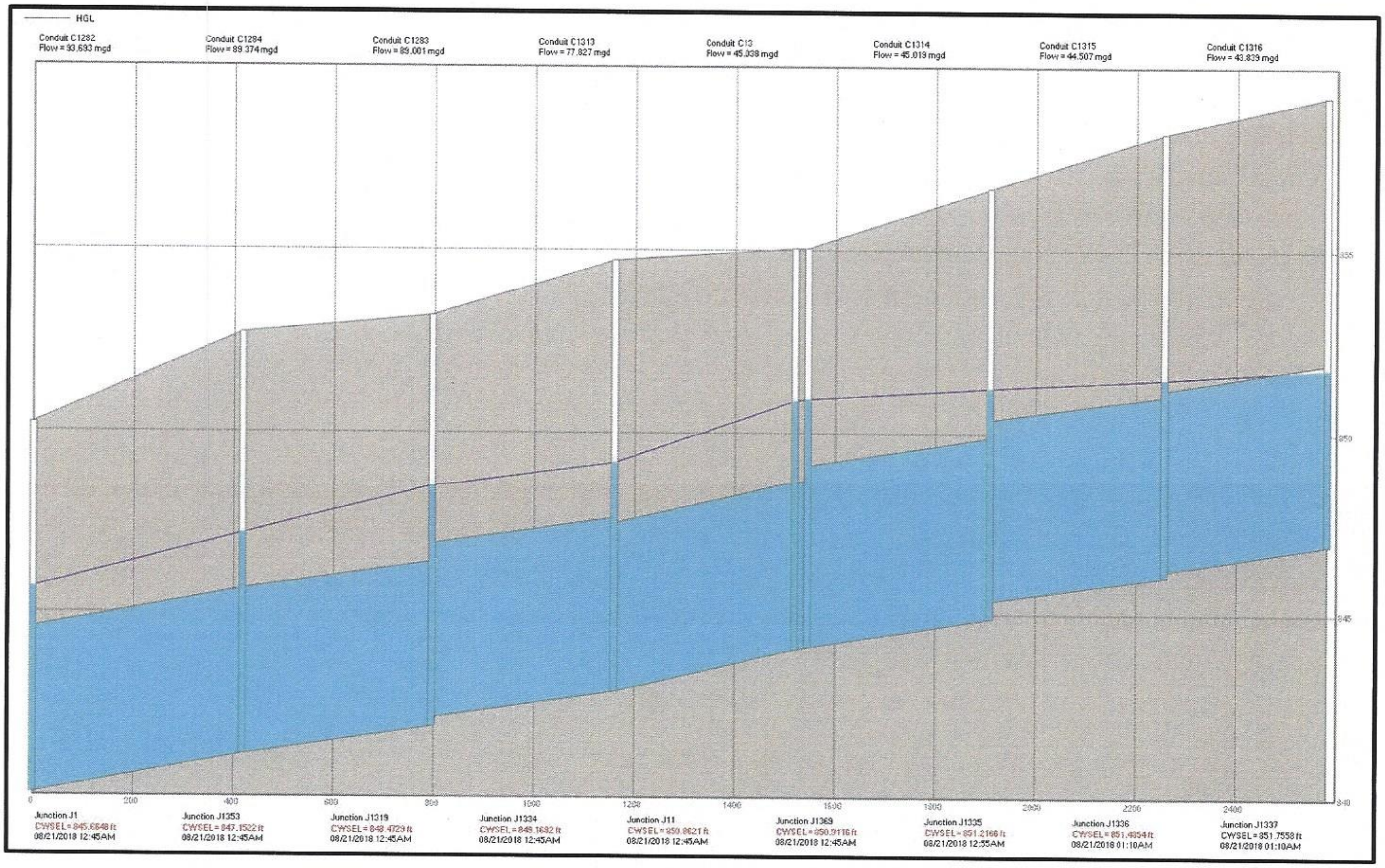
Figure 7-4: 25-Year 6-Hour Storm Results

Table 7-2: Design Storm System Summary

Design Strom	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	138	85	53	4	0.35
10 year 6 hour	233	118	115	15	2.75
25 year 6 hour	320	161	159	37	9.04

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 Peak HGL on Pearl Rd



7.3 5 YEAR DESIGN STORM ANALYSIS

The target level of service for this project is the 5-year 6-hour design storm. After model calibration, the capacity results and hydraulic grade lines (HGL) were compared to determine areas of possible surface flooding.

Similarly to results found in the ***Phase I Storm Sewer Evaluation Report*** submitted on December 29, 2017, the main capacity issues area caused by the main storm sewers along Pearl Rd, Beverly Drive, and Big Creek Parkway. This is mainly due to large inflows along Pearl Rd travelling east, from flows traveling north along Parma Park Boulevard, and from flows traveling east along Big Creek Parkway to Beverly Drive. Peak flows during the 5 year design storm reached 44.75 MGD along Pearl Road, 100.5 MGD along Parma Park Boulevard, and 81.84 MGD along Big Creek Parkway.

Other sewer lines experiencing pipe capacity limitations during the 5 year storm are Maplewood Rd, Oakwood Rd, Beresford Ave, Greenleaf Ave, Meadowbrook Dr., and on most streets north of Big Creek Parkway. 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 20.6% of the total flow entering the system and has a peak flow of 34.76 MGD. Overall it is ranked 4th for flow contribution to the Beverly Drive storm sewer behind flows from Pearl Avenue, Maplewood Rd, and Lawndale Dr.

A flow ranking of the top 10 flow contributors can be found in Table 7-3 and a map of showing peak flows at each location can be seen in Figure 7-6. 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 predictions.

Table 7-3: Top 10 Flow Contributors

Location	Peak Flow (MGD)
Pearl Road	45.0
Maplewood Road	44.88
Nathan Hale Park	35.73
Alexandria Drive	32.92
Crenshaw Drive	29.97
N. Commonwealth Blvrd.	23.94
Lawndale Drive	17.15
Meadowbrook Drive	16.63
S. Parma Park Blvrd.	12.85
4' x 10' Storm	12.68

Figure 7-6: 5 Year 6 Hour Storm Max Peak Flows

7.4 NATHAN HALE STORAGE BASIN

A proposed Storage Basin is located in Nathan Hall Park in the southeastern section of the project area. Flow Monitor 14 recorded flows from this area during Phase II. Incoming flows are mainly transported through a swale whose tributary area is about 210 Acres. 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 for the 5 year 6 hour design storm. This conduit is downstream of Nathan Hale Park and is in a location where hydraulic issues are present. This location can be seen in Figure 7-7.

The proposed location has positive effect on the hydraulic issues in the system. Total flow is reduced by over 4 million gallons and allows for flow along Pearl Road to travel down Beverly Drive. There are still hydraulic issues downstream of Beverly Drive and at the intersection of Pearl Rd and Maplewood Rd but the overall system surcharging is reduced. Figure 7-8 displays the storage results for the 5 year 6 hour design storm

Figure 7-7: Project Overview for the 5 year 6 hour Design Storm

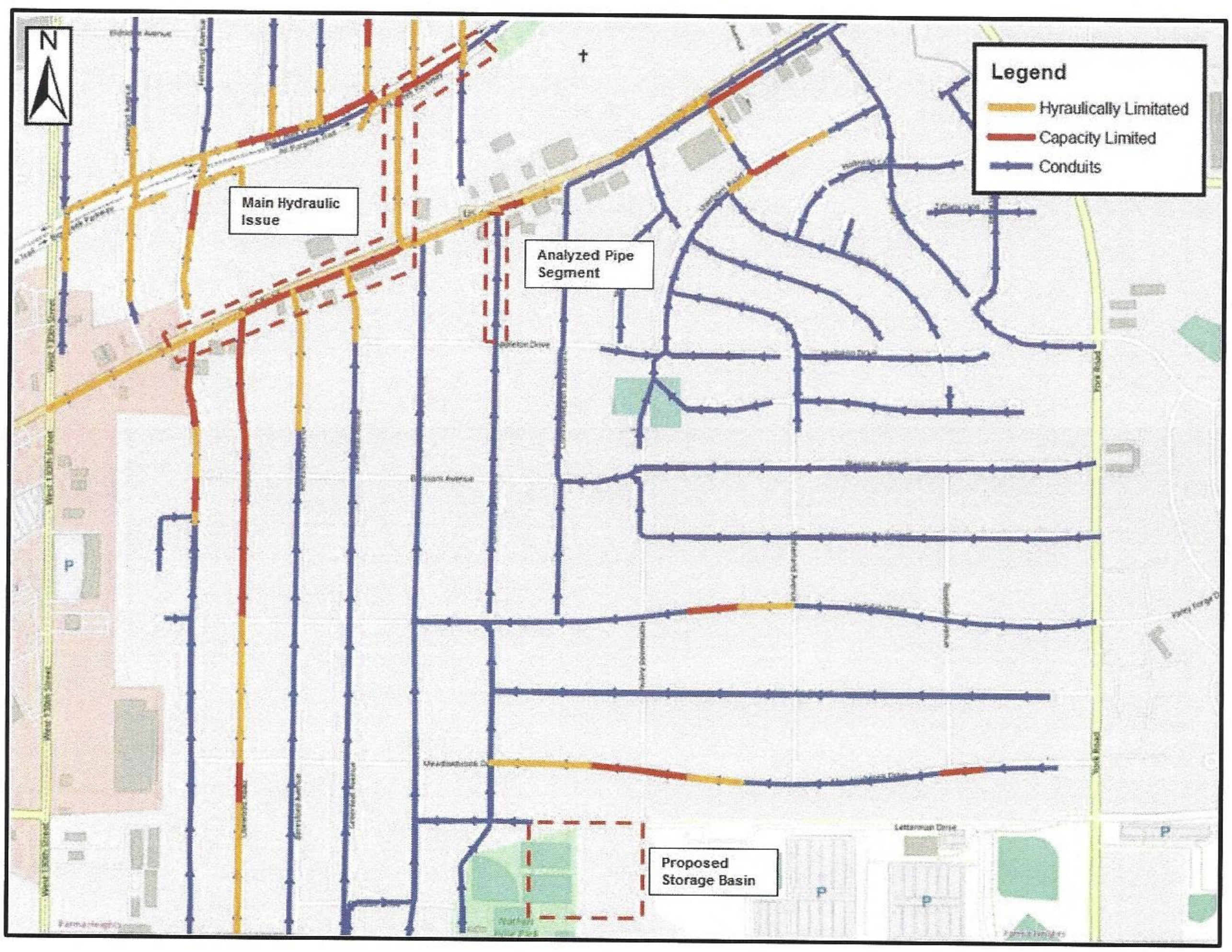


Figure 7-8: 5 Year 6 Hour Model Results with Nathan Hale Storage

8.0 RECOMMENDATIONS

8.1 ALTERNATIVE 1: UPSIZE HYDRAULICALLY LIMITED PIPES

The model predicted that the Nathan Hale Park storage provided relief to the system but that there were still hydraulic bottlenecks in 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. This process excluded the Nathan Hale Park storage.

8.1.1 5 Year Design Storm

Overall, forty-two (42) pipes needed to be upsized to completely eliminate surcharging during the 5 year 6 hour design storm. A summary of results 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. There are pipes that still experienced hydraulic limitations on the arterial streets but this is due to invert locations on the main storm sewer and not due to pipe size. There streets include Oakwood Road, Beresford Ave, Fernhurst Ave, the storm sewer from Crenshaw Drive, Beverly Drive (north of Big Creek Pkwy), and Mandalay Drive.

Figure 8-1: 5 Year Storm Upsized Pipes

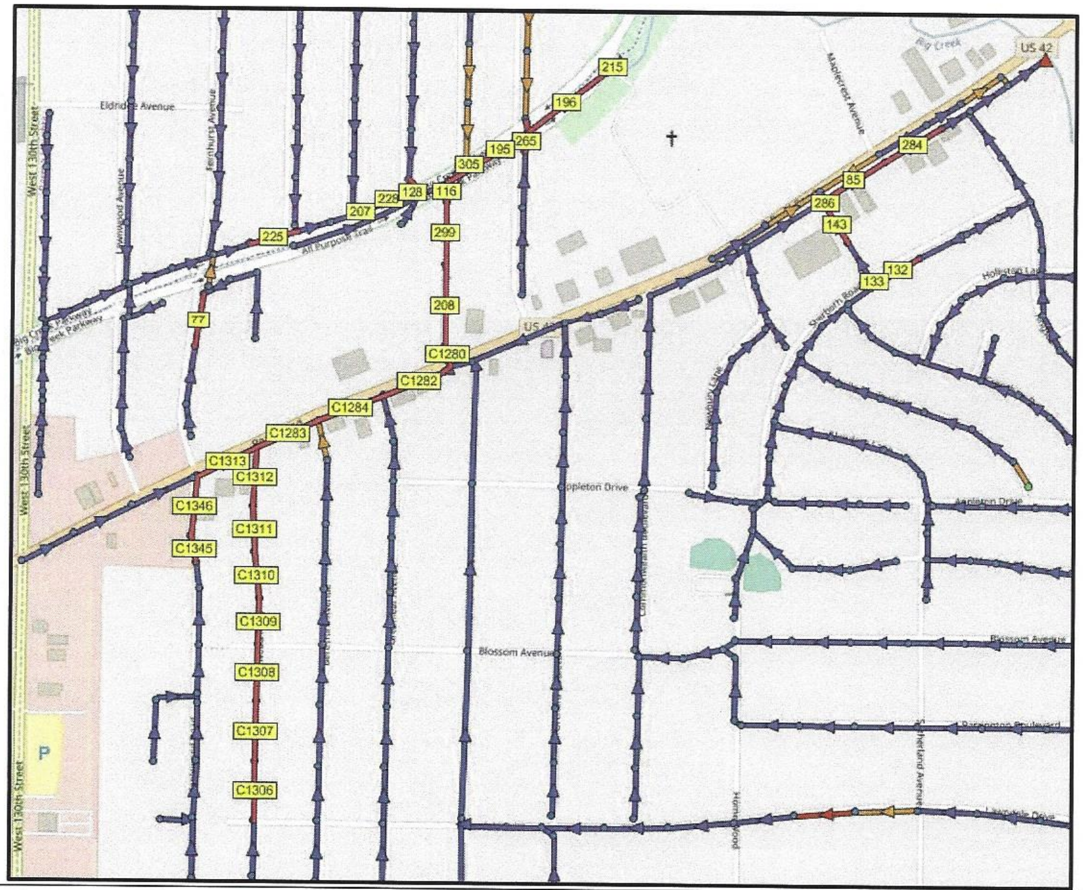


Table 8-1: 5 Year Storm Pipe Capacity Summary

Name	Shape	Original Size (ft)	Upgraded Size (ft)	Slope	Mannings	Original Full Flow (MGD)	New Full Flow (MGD)	Capacity Increase (MGD)
116	Rectangular	5x11	6x11	0.0009	0.020	109.7	142.8	33.1
117	Rectangular	4.5x10	6x11	0.0041	0.020	183.1	311.8	128.7
195	Rectangular	4.5x10	6x11	0.0052	0.020	205.4	349.8	144.4
196	Rectangular	4.5x10	6x11	0.0016	0.020	116.0	197.5	81.5
208	Rectangular	5x11	6x11	0.0033	0.020	213.6	278.0	64.4
215	Rectangular	4.5x10	6x11	0.0020	0.020	127.0	216.2	89.3
299	Rectangular	5x11	6x11	0.0045	0.020	250.6	326.1	75.5
C1280	Rectangular	5x11	6x11	0.0038	0.010	461.0	599.9	138.9
C1282	Rectangular	5x6	6x6	0.0026	0.019	72.1	119.3	47.1
C1283	Rectangular	5x6	6x6	0.0020	0.018	76.2	110.5	34.3
C1284	Rectangular	5x6	6x6	0.0020	0.020	60.6	100.2	39.6
77	CIRCULAR	1	1.5	0.0042	0.010	1.9	5.6	3.7
85	CIRCULAR	4.5	5	0.0021	0.020	37.4	49.6	12.1
128	CIRCULAR	4.5	5	0.0020	0.023	31.4	41.5	10.2
132	CIRCULAR	1.25	1.5	0.0041	0.020	1.7	2.8	1.1
133	CIRCULAR	1.25	1.5	0.0103	0.020	2.7	4.4	1.7
134	CIRCULAR	3.5	4	0.0025	0.020	20.9	29.8	8.9
143	CIRCULAR	4	4.5	0.0036	0.020	35.8	49.1	13.2
156	CIRCULAR	3	4	0.0031	0.010	30.9	66.5	35.6
183	CIRCULAR	3	4	0.0030	0.010	30.5	65.7	35.2
207	CIRCULAR	4.5	5	0.0022	0.022	35.1	46.6	11.4
225	CIRCULAR	4	4.25	0.0019	0.023	22.6	26.6	4.0
228	CIRCULAR	4.5	5	0.0021	0.023	32.6	43.2	10.6
265	CIRCULAR	1.75	2	0.0038	0.010	8.1	11.6	3.5
267	CIRCULAR	2.5	2.75	0.0026	0.010	17.4	22.5	5.0
284	CIRCULAR	4.5	5	0.0025	0.020	41.1	54.5	13.3
286	CIRCULAR	4	4.5	0.0176	0.010	158.2	216.6	58.4
305	CIRCULAR	2.5	2.75	0.0046	0.010	23.1	29.8	6.7
C1306	CIRCULAR	1.75	2	0.0031	0.015	4.8	6.9	2.1
C1307	CIRCULAR	1.75	2	0.0030	0.015	4.8	6.8	2.0
C1308	CIRCULAR	2	2.25	0.0029	0.015	6.8	9.3	2.5
C1309	CIRCULAR	2	2.25	0.0030	0.015	6.9	9.4	2.5
C1310	CIRCULAR	2.25	2.75	0.0030	0.015	9.3	15.9	6.6
C1311	CIRCULAR	2.25	2.75	0.0030	0.015	9.4	16.1	6.7
C1312	CIRCULAR	2.25	2.75	0.0040	0.015	10.9	18.6	7.7
C1313	CIRCULAR	4.5	5	0.0032	0.018	54.6	68.2	13.6
C1344	CIRCULAR	4.5	5	0.0008	0.015	30.1	39.9	9.8
C1345	CIRCULAR	4.5	5	0.0013	0.015	39.8	52.7	12.9
C1346	CIRCULAR	4.5	5	0.0009	0.015	32.4	43.0	10.5
C527	CIRCULAR	2.75	3	0.0046	0.018	16.6	20.9	4.3
C528	CIRCULAR	2.75	3	0.0050	0.018	17.2	21.7	4.5
C574	CIRCULAR	2.75	3	0.0050	0.018	17.3	21.8	4.5

8.1.2 10 Year Design Storm

Overall, sixty-four (64) pipes needed to be upsized to completely eliminate surcharging during the 10 year 6 hour design storm. A summary of results listed in Table 8-2 for rectangular pipes and Table 8-3 for circular pipes. Each table includes the initial capacity for each pipe and a new capacity of the upsized pipe. Figure 8-2 illustrates which pipe segments were increased. There are pipes that still experienced hydraulic limitations on the arterial streets but this is due to invert locations on the main storm sewer and not due to pipe size. There streets include Oakwood Road, Beresford Ave, Greenleaf Ave, the storm sewer from Crenshaw Drive, Lawnwood Ave, Fernhurst Ave, Beverly Drive (north of Big Creek Pkwy), Alexandria Dr, Mandalay Drive.

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 58.9 MGD and greatly improved hydraulic issues along the two intersecting streets.

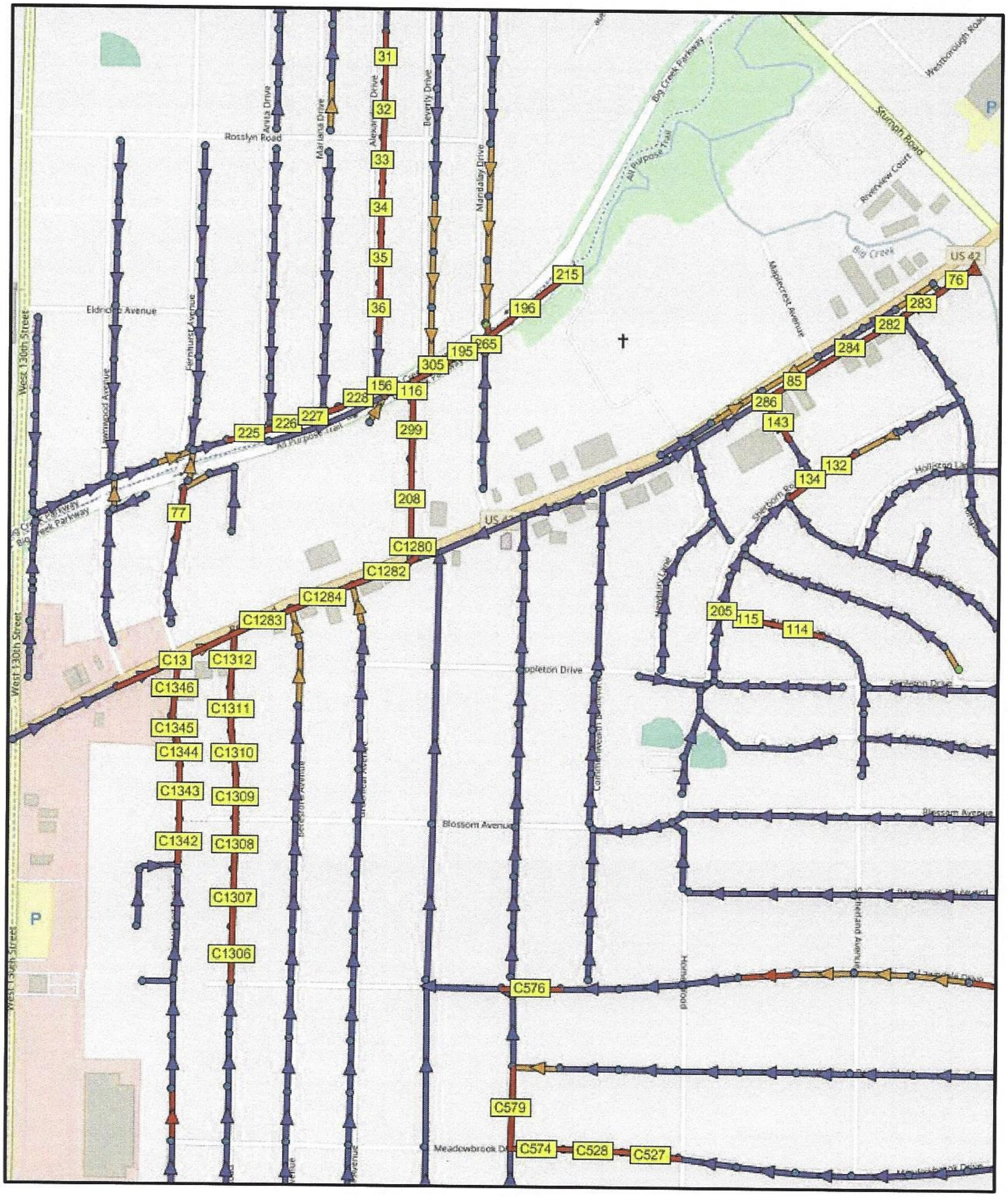
Table 8-2: 10 Year Storm Pipe Capacity Summary

Name	Shape	Original Size (ft)	Upgraded Size (ft)	Slope	Mannings	Original Full Flow (MGD)	New Full Flow (MGD)	Capacity Increase (MGD)
116	Rectangular	5x11	6x11	0.0009	0.020	109.7	142.8	33.1
117	Rectangular	4.5x10	6x11	0.0041	0.020	183.1	311.8	128.7
195	Rectangular	4.5x10	6x11	0.0052	0.020	205.4	349.8	144.4
196	Rectangular	4.5x10	6x11	0.0016	0.020	116.0	197.5	81.5
208	Rectangular	5x11	6x11	0.0033	0.020	213.6	278.0	64.4
215	Rectangular	4.5x10	6x11	0.0020	0.020	127.0	216.2	89.3
299	Rectangular	5x11	6x11	0.0045	0.020	250.6	326.1	75.5
C1280	Rectangular	5x11	6x11	0.0038	0.010	461.0	599.9	138.9
C1282	Rectangular	5x6	6x6	0.0026	0.019	72.1	119.3	47.1
C1283	Rectangular	5x6	6x6	0.0020	0.018	86.4	110.5	24.1
C1284	Rectangular	5x6	6x6	0.0020	0.020	60.6	100.2	39.6
C1313	Rectangular	4.5	5x6	0.0032	0.0	51.5	110.4	58.9

Table 8-3:10 Year Storm Pipe Capacity Summary

Name	Shape	Original Size (ft)	Upgraded Size (ft)	Slope	Mannings	Original Full Flow (MGD)	New Full Flow (MGD)	Capacity Increase (MGD)
31	CIRCULAR	3	3.5	0.0047	0.0	37.9	57.1	19.3
32	CIRCULAR	3	3.5	0.0051	0.0	39.5	59.6	20.1
33	CIRCULAR	3	3.5	0.2279	0.0	264.1	398.4	134.3
34	CIRCULAR	3	3.5	0.0036	0.0	33.3	50.2	16.9
35	CIRCULAR	3	3.5	0.0037	0.0	33.6	50.8	17.1
36	CIRCULAR	3	3.5	0.0083	0.0	50.3	75.9	25.6
37	CIRCULAR	3	3.5	0.0149	0.0	67.5	101.8	34.3
76	CIRCULAR	5	5.5	0.0023	0.0	51.3	66.2	14.9
77	CIRCULAR	1	1.5	0.0232	0.01	4.5	13.3	8.8
85	CIRCULAR	4.5	5.5	0.0054	0.0	59.9	102.4	42.4
114	CIRCULAR	2.25	2.5	0.0030	0.02	7.0	9.3	2.3
115	CIRCULAR	2.25	2.5	0.0025	0.02	6.5	8.6	2.1
128	CIRCULAR	4.5	5	0.0040	0.0	45.0	59.6	14.6
132	CIRCULAR	1.25	1.5	0.0030	0.02	1.5	2.4	0.9
133	CIRCULAR	1.25	1.5	0.0030	0.02	1.5	2.4	0.9
134	CIRCULAR	3.5	4	0.0029	0.02	22.6	32.3	9.7
137	CIRCULAR	3.5	4	0.0031	0.02	23.1	33.0	9.9
143	CIRCULAR	4	4.5	0.0042	0.0	38.6	52.8	14.2
156	CIRCULAR	3	4	0.0039	0.0	34.4	74.0	39.7
183	CIRCULAR	3	4	0.0030	0.0	30.1	64.9	34.8
205	CIRCULAR	2.5	3	0.0138	0.0	20.0	32.5	12.5
207	CIRCULAR	4.5	5	0.0074	0.0	63.8	84.5	20.7
225	CIRCULAR	4	4.5	0.0070	0.0	43.3	59.3	16.0
226	CIRCULAR	4	5	0.0028	0.0	27.3	49.4	22.2
227	CIRCULAR	4.5	5	0.0067	0.0	58.3	77.2	18.9
228	CIRCULAR	4.5	5	0.0068	0.0	58.6	77.6	19.0
265	CIRCULAR	1.75	2	0.0049	0.01	9.2	13.1	3.9
267	CIRCULAR	2.5	2.75	0.0031	0.01	19.0	24.5	5.5
282	CIRCULAR	5	5.5	0.0035	0.0	63.9	82.4	18.5
283	CIRCULAR	5	5.5	0.0034	0.0	63.0	81.2	18.2
284	CIRCULAR	4.5	5.5	0.0036	0.0	49.1	83.8	34.7
286	CIRCULAR	4	4.5	0.0056	0.0	88.8	121.5	32.8
305	CIRCULAR	2.5	2.75	0.0034	0.01	19.9	25.6	5.7
C13	CIRCULAR	4.6	5.5	0.0035	0.0	56.7	91.3	34.6
C1306	CIRCULAR	1.75	2.25	0.0130	0.015	10.0	19.6	9.5
C1307	CIRCULAR	1.75	2.25	0.0030	0.015	4.8	9.4	4.6
C1308	CIRCULAR	2	2.5	0.0030	0.015	6.8	12.4	5.6
C1309	CIRCULAR	2	2.5	0.0030	0.015	6.9	12.4	5.6
C1310	CIRCULAR	2.25	2.75	0.0019	0.015	7.4	12.6	5.2
C1311	CIRCULAR	2.25	2.75	0.0022	0.015	7.9	13.6	5.6
C1312	CIRCULAR	2.25	2.75	0.0038	0.015	10.6	18.1	7.5
C1314	CIRCULAR	5	5.5	0.0038	0.0	74.0	95.4	21.4
C1342	CIRCULAR	4.5	5	0.0181	0.0	156.7	207.6	50.8
C1343	CIRCULAR	4.5	5	0.0148	0.0	141.7	187.7	46.0
C1344	CIRCULAR	4.5	5.5	0.0044	0.0	72.2	123.3	51.1
C1345	CIRCULAR	4.5	5.5	0.0060	0.0	84.4	144.1	59.7
C1346	CIRCULAR	4.5	5.5	0.0049	0.0	76.0	129.8	53.8
C527	CIRCULAR	2.75	3	0.0077	0.018	21.3	26.9	5.6
C528	CIRCULAR	2.75	3	0.0079	0.018	21.6	27.3	5.6
C574	CIRCULAR	2.75	3	0.0028	0.018	12.8	16.1	3.3
C576	CIRCULAR	2.75	3	0.0009	0.015	8.7	11.0	2.3
C579	CIRCULAR	3	4	0.0495	0.0	68.4	147.3	78.9

Figure 8-2: 10 Year Storm Upsized Pipes



8.2 ALTERNATIVE 2: SECONDARY STORAGE BASIN SITE

Instead of increasing the storm sewer pipe size, a second alternative to address the hydraulic bottleneck at Pearl Rd and Meadowbrook Rd would be to construct a second storage basin in conjunction with the Nathan Hale Park storage basin. 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. Modeling results display that this second storage location, along with the storage basin in Nathan Hale Park, allow the storm system south of Big Creek Parkway to operate normally. There are still capacity issues along Big Creek Parkway. The legalities and ownership of this land is not currently know and would need to be investigated.

Figure 8-3: Secondary Location for Storage Basin

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 October 2018.

The model was used to analyze the existing sewer system's performance under various wet weather conditions, including three design storms (5, 10, and 25 year recurrence, 6 hour duration) and one historical event that occurred on the August 6, 2018. Main pipe capacity limitations during the 5 year storm were predicted along Pearl Rd and Big Creek Parkway. On Pearl Rd limitations were mainly at the intersections of Maplewood Rd, Oakwood Rd, and Parma Park due to large inflows along Pearl Rd travelling west and from flows traveling north along Maplewood Rd. On Big Creek Parkway, limitations were mainly due to large inflows along Big Creek Parkway travelling west and from flows traveling north along Beverly Drive.

Other bottlenecks include Maplewood Rd, Oakwood Rd, Alexandria Drive, Orchard Blvd, and Beverly Drive. These restrictions create pipe surcharge in the upstream sections. There were four (4) manholes flooding predicted for the 5 year storm. For the 10 and the 25 year storms, hydraulic conditions were worse, with more pipe surcharge and respectively 15 and 37 flooding locations.

The proposed Nathan Hale Park storage was modeled, showing a positive 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 and local swale, but these flows account for approximately 10.6% of the total storm sewer flow for that system.

The main alternative to eliminate pipe surcharge during the 5 year 6 hour design storm is to upsize the storm sewer along Pearl Rd, Beverly Drive, and Big Creek Parkway. 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 in conjunction with the Nathan Hale 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. Increasing the size or number of catch basins in flood reported areas should also alleviate issues.