HISTORIC MILL DISTRICT WATER & SEWER MASTER PLAN

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EXECUTIVE SUMMARY

At the request of the Town of Andover's Department of Community Development & Planning, Woodard & Curran has prepared this Water & Sewer Master Plan for the Town of Andover's (the Town) Historic Mill District (HMD). The Water & Sewer Master Plan presents the assessment of the Town's existing sanitary sewer and water infrastructure within the HMD using the redevelopment guidelines developed by the Town.

The primary goal of the Water & Sewer Master Plan for the HMD is to document the current and proposed redevelopment water demand and sewer loading, locate infrastructure capacity deficiencies, and provide recommendations for water and sewer infrastructure improvements for the proposed redevelopment. Proposed developments may require a Peer Review by the Town's Consultant to be paid for by the developer.

Sanitary Sewer Capacity Assessment

Utilizing the Town's existing geographic information system (GIS) database and relevant record drawings of their existing wastewater infrastructure, a hydraulic model (SewerGEMS) of the collection system was developed. One day of field investigation was also completed by a Woodard & Curran engineer to inspect key junction manholes where record drawing information was conflicting or not existent. The model incorporated hydraulic elements such as ground surface elevations, manhole rim elevations, sewer pipe elevations, and sewer pipe diameters and slopes. Using this desktop model of the sewer system, existing and proposed wastewater flows were introduced based on flows measured during the Town's recent wastewater metering program. Woodard & Curran then performed an analysis that describes hydraulic performance of the sewer system during existing wastewater flow scenarios and proposed redevelopment flow scenarios. For the purposes of this report, hydraulic deficiencies are defined as "high risk of sanitary sewer overflow" or sewer overflowing out of the sanitary sewer collection system and into the surrounding area due to overloading of the system with excess flow. Sanitary sewer overflows should not be acceptable to the Town as they cause risk to public health, financial impacts for cleanup, and non-compliance with the Clean Water Act resulting in fines and the possibility of forced capital improvements.

Woodard & Curran's analysis of the existing sewer system during existing wet (high flow due to rainfall/groundwater entering the sanitary sewer system) and existing dry weather (only the influence of sanitary flow from residential/commercial customers) shows that there is minimal risk for sanitary sewer overflows in the HMD area.

Woodard & Curran's analysis of the existing sewer system's performance under proposed redevelopment shows that there is minimal risk for sanitary sewer overflows caused by the addition of redevelopment flows. High flows that increase risk of sanitary overflow in the sanitary sewer system are a function of wet weather flow entering the system, rather than the nominal additional flow from redevelopment (an additional 125,000 gallons per day or 86 gallons per minute). Specifically, the Town's interceptor sewers in the area (along the Shawsheen River) have adequate capacity. Smaller sewers, located on side streets currently servicing few users, should be considered for replacement due to capacity or future maintenance related challenges.

The following should be considered if the proposed redevelopment occurs in the HMD area:

1. Town should consider performing a detailed review of developers proposed utility plans indicating exact locations of sewer service connections as the proposed sewer connections may differ than what is considered in the hydraulic model.

2. The HMD area contains approximately 4,770 linear feet of 6-inch mainline vitrified clay sewers (see Appendix D for locations of sewer and record drawings). If developers plan to connect to the 6-inch mainline sewers, it is recommended they be removed and replaced with new SDR35 PVC no smaller than 8-inch in diameter for

the reasons listed below. In addition, Woodard & Curran recommends that CCTV be performed on the sewers located within the gas impacted area and conduct any necessary repairs.

a. The 6-inch mainline sewers have been in service for more than 100 years and may be in poor condition.

b. 6-inch sewers can be costly to clean, inspect, and maintain due to grease, debris, and root buildup.

c. 6-inch sewers are difficult to repair trenchlessly due to their small size which limits the ability of trenchless robots to navigate bends or intruding service connections. Therefore, replacement of these sewers should be considered. Replacement may be more cost-effective if coordinated with other utility replacement and prior to street paving.

d. A budgetary cost of approximately $$560-$600/LF to replace these sewers is estimated based on recent bid prices in the Merrimack Valley. If all of these sewers require upgrade, a total construction cost of approximately $2,900,000 is expected.

3. In order to increase reliability of the sewer service in the area performing a pipe CCTV inspection should be considered and surface-level manhole condition assessment that conforms to the National Association of Sewer Service Companies (NASSCO) standards. This will help to identify sewers that need to be updated because of structural failure even if they are adequate hydraulically. It is expected that the sewer system in the area is near the end of its useful life due to much of the system being constructed of vitrified clay pipe likely greater than 50-100 years old.

4. It may be prudent for the Town to reduce upstream infiltration and inflow which causes peak flows in the HMD area. The 2017 I/I Metering report should form the basis of the Town's I/I removal program, and basins upstream of the HMD (e.g. sewer basins 3, 14, 28) should be targeted to increase sewer capacity in the HMD area, and reduce flows to the Greater Lawrence Sanitary District. In addition, Woodard & Curran recommends that new developments within these basins be responsible for moving to 4:1 removal.

5. Per the November 27, 2018 field visit, manhole SMH-210, where GIS shows two existing outflow pipes, may only have one functioning outflow pipe. This was based on discussion with Town collection systems staff. If this sewer is truly non-functioning, then the 24-inch concrete relief sewer may not provide sufficient capacity. This non-function/blocked pipe should be inspected and rehabilitated via cleaning, chemical root treatment, or other method, prior to redevelopment of the area. Much of the sewer is located off-road which may cause inspection challenges.

6. Per the November 27, 2018 field visit, a potentially large source of inflow was located at 8 Essex Street. Upon inspection of MH1494, a large, clear, continuous discharge of water via unknown PVC connection was located. It is possible that roof drains from nearby buildings may be plumbed directly into the sanitary collection system. Prior to any redevelopment in the area, this connection should be confirmed via dye test and reconnected to nearby stormdrain.

7. Areas 1-5, as shown in Appendix C, should be investigated and repaired, if necessary, prior to redevelopment in the HMD.

Summary of estimated unit costs for each consideration are provided in the table below:

Table 1:Estimated Costs for Wastewater Recommendations

Summary of Recommendations			
Item	**Description**	**Unit**	**Unit Cost**
Review Developer Plans	Review developer utility plans.	Per Development	$5,000.00
Replace 6-inch Sewers	Provide 8-inch or larger sewers downstream of new connections.	Per Foot	$600.00
HMD Area Pipe CCTV Inspection	Perform condition assessment of sewers in HMD Area	Per Foot	$2.50
HMD Area Manhole Inspection	Perform condition assessment of manholes in HMD Area.	Per MH	$100.00
Perform SSES Upstream of HMD Area	Perform SSES upstream of HMD Area to Locate I/I.	See Appendix E - CDM Recommendations	
Rehabilitation of Basins Upstream of HMD Area	Remove upstream infiltration/inflow to increase capacity in HMD Area.	See Appendix E - CDM Recommendations	
Cleaning and Inspection Interceptor Sewer Downstream of SMH-210	Per collection system operator, eastern sewer may be blocked with roots/debris. Should be cleaned and inspected. Approximately 1,700 feet.	$10,000	
Perform Building Inspection at 8 Essex Street	Locate source of unknown clear water connection and recommend rehabilitation.	$10,000	

Drinking Water System Capacity Assessment

For the drinking water system capacity assessment, Woodard & Curran utilized the hydraulic model developed by CDM Smith in 2016. Element characteristics including pipe connectivity, pipe diameter, length, C-values, and node elevation as well as operational characteristics were utilized from the previous model, with the following exceptions. In 2017 Woodard & Curran performed a limited calibration of the hydraulic model and made minor modifications to the treatment plant operating schedule and pump data. In addition, Woodard & Curran conducted additional calibration work focused on the HMD, as described in this Water & Sewer Master Plan. No additional updates or calibrations of the hydraulic model have otherwise been conducted.

In November 2018, Woodard & Curran conducted hydrant flow tests to assess the capacity for fire suppression (fire flow) and to provide flow and head loss data for use in calibrating pipe C-Factors within the hydraulic model in the vicinity of the HMD. Following, a limited C-Factor calibration was performed to validate the information used in the hydraulic model for the HMD. Woodard & Curran used the calibrated model to create and analyze two scenarios, one for Average Day Demand and one for Maximum Day Demand. The scenarios were compared to the existing HMD area distribution system and demand conditions in order to assess impacts to the water distribution infrastructure as a result of the redevelopment of nine parcels within the HMD.

Based on the hydraulic modeling analysis it was determined that the additional estimated domestic demands of proposed redevelopment of the HMD do not appear to have a significant effect on the available pressure or flow of the existing water distribution system within the vicinity of the HMD. However, Woodard & Curran recommends the following should the proposed redevelopment occur in the HMD area:

- The water distribution infrastructure should be reanalyzed once demands are established and submitted to the Town by developers.

- The needed fire flow for each property within the HMD should be verified with the Town's Fire Department to determine that adequate AFF is provided at all locations.

In addition, the following recommendations for water main improvements within the vicinity of the HMD are presented below.

All junctions had a predicted Available Fire Flow (AFF) value at or above the Insurance Services Office (ISO) recommended 500 gpm. However, the ISO recommended 500 gpm is a minimum value for single-family spaced out development. Since the HMD is mainly commercial, Woodard & Curran assumed for the purposes of the analysis that a needed fire flow of 2,500 gpm would be required for properties within the HMD. However, the needed fire flow for each property within the HMD should be verified with the Town's Fire Department to determine that adequate AFF is provided at all locations. Based on an analysis of AFF under maximum day demand conditions, approximately one-third of the fire flow junctions in the vicinity of the HMD had a predicted AFF of less than 2,500 gpm. In general, locations with predicated AFF below 2,500 gpm correlated with water mains which had low C-Factors or were small diameter mains. In addition, the hydraulic model was updated with the recommended water main improvements in the HMD and the water main improvements that the Town will conduct in the gas affected area and Scenario 2B was reevaluated. In general, with the improvements mentioned the predicted AFF on public roadways in the vicinity of the HMD was at or above 2,500 gpm.

Therefore, Woodard & Curran recommends the Town consider developing a phased approach for the following water main improvements within the vicinity of the HMD. In addition, it is recommended that water main improvements on Lupine Road, Henderson Avenue, Marion Avenue, and Central Street including final paving be conducted.

- Upgrade of 6-inch unlined cast iron water main in Red Spring Road from Moraine Street to Essex Street
- Upgrade of unlined cast iron water main in Lupine Road.
- Abandon the 8-inch water main and tie-over services to the existing 16-inch parallel water main in Central Street from Red Spring Road to Torr Street.
- Upgrade 6-inch lined cast iron water main in Henderson Avenue and Marion Avenue.
- Replace 8-inch unlined cast iron water main in Red Spring Road off Essex Street.
- Replace 12-inch cast iron water main in Essex Street
- Upgrade 6-inch lined cast iron water main in Pearson Street.
- Upgrade 6-inch cast iron water main in Railroad Street.
- Upgrade 6-inch unlined cast iron water main in Amici Way.
- Tie-over water services to the existing 12-inch water main in Stevens Street from Shawsheen Road to approximately 27 Stevens Street.

The majority of the recommended water main improvements are in the area effected by the current gas main replacement work; therefore, the Town may want to consider accelerating the water main improvements in the HMD.

Based on the recommendations for water main improvements above, approximately 12,330 linear feet (LF) of 6-inch and 8-inch water main and approximately 3,560 LF of 12-inch water main within the HMD are recommended for upgrade and/or replacement. Additional improvements include abandonment of smaller water mains and connecting water services to larger mains. Woodard & Curran prepared a preliminary cost estimate for the water main improvements, as summarized in the table below. It was assumed that all 6-inch water mains would be upgraded with 8-inch cement line ductile iron (CLD) mains and all 8-inch and 12-inch water mains would be replaced with CLDI main of the same size. A budgetary cost of approximately $275/LF was assumed to upgrade 6-inch water main to new 8-inch water main and replace 8-inch water main. A budgetary cost of approximately $325/LF was assumed for

12-inch water main replacement. If all of water main improvements are conducted, a total design and construction cost of approximately $7,836,600.00 is estimated. The estimated cost assumes full-width mill and pave. An increase in cost is anticipated if the Town wanted to conduct reclamation instead of full width mill and pave.

Table 2: Preliminary Cost Estimate for Water Main Improvements

Parameter	Preliminary Cost
Construction Cost	$5,295,000.00
Design (10%)	$529,500.00
Police Detail Allowance (13%)	$688,350.00
Construction Administration (15%)	$794,250.00
Contingency (10%)	$529,500.00
Total	$7,836,600.00

1. PROJECT BACKGROUND

At the request of the Town of Andover's Department of Community Development & Planning, Woodard & Curran has prepared this Water & Sewer Master Plan for the Town of Andover's (the Town) Historic Mill District (HMD). The Water & Sewer Master Plan presents the assessment of the Town's existing sanitary sewer and water infrastructure within the HMD using the redevelopment guidelines developed by the Town.

1.1 Introduction

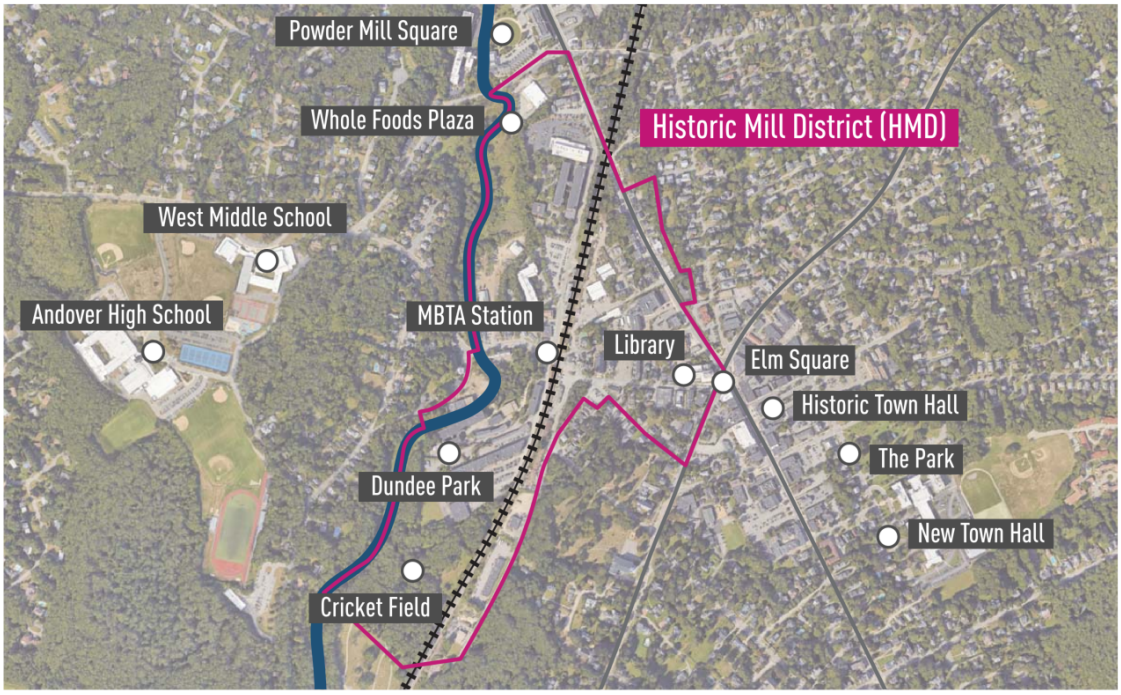
The Town of Andover is interested in redeveloping a portion of the Town known as the HMD, with a focus on controlled economic development and adaptive reuse. As part of the redevelopment process, Gamble Associates established guidelines, titled Historic Mill District Design Guidelines and dated February 6, 2018, to address the design of new buildings within the HMD. According to the HMD Guidelines, "the purpose of the Historic Mill District is to encourage smart growth development in proximity to Andover's regional transit station by fostering a range of housing opportunities within mixed-use development projects that promote compact design, preservation. These guidelines were adopted by the Andover Planning Board on February 13, 2018.

1.2 Site Location

The Historic Mill District is a zoning district established in 2015, which includes approximately 100 acres of developed and undeveloped land. As shown in Figure 1, the Historic Mills District is generally bounded by the following:

North: The intersection of Harding Street, North Main Street, and Stevens Street
Northeast: North Main Street
East: The intersection of Central Street, Elm Street, North Main Street, and Main Street
Southeast: Lupine Road
South: St. Augustine's Cemetery
West: Shawsheen River

Figure 1: Boundaries of the Historic Mill District



Source: Gamble Associates, Historic Mill Design Guidelines, February 6, 2018

1.3 Project Objective

As part of the redevelopment efforts within the HMD, the Town requested the preparation of a Water and Sewer Master Plan, which supports the proposed revitalization, and identifies improvements needed to meet Insurance Services Office (ISO) requirements and provide adequate sewer collection system capacity to the HMD area. The preparation of this Water and Sewer Master Plan included the development of future sanitary sewer loads and water demand projections based on the anticipated buildout of the HMD and an evaluation of the existing sanitary sewer and water infrastructure under redevelopment conditions. In addition, recommendations for improvements to remedy identified current and future deficiencies and to improve the reliability of the water and wastewater systems under future demand conditions are included where applicable.

1.4 Redevelopment of the Historic Mill District

The redevelopment of the HMD was depicted by Gamble Associates during development of the HMD Design Guidelines in February 2018. Appendix A is from the design guidelines and illustrates the nine properties within the HMD that are identified by the Town for development opportunities. Appendix A also identifies the anticipated use of each redeveloped property. Prior to redevelopment, proposed developments may require a Peer Review by the Town's Consultant to be paid for by the developer.

1.4.1 Sanitary Sewer Loads & Water Design Demands

In order to evaluate the potential impact that the redevelopment of the HMD area will have on the Town's sanitary sewer and water infrastructure, future sanitary sewer loads and water demands were developed for each property using Massachusetts *310 CMR 15.00 The State Environmental Code, Title 5 Standard Requirements for the Siting, Construction, Inspection, Upgrade and Expansion of On-Site Sewage Treatment and Disposal Systems and for the Transport and Disposal of Septage* (Title V).

The following assumptions were used in the development of the estimated sanitary sewer loads for the proposed developments:

- **Residential:** The total square footage presented in Table 3 below was assumed to be divided into bedrooms for each residential building. Space for walkways, common areas, elevators, etc. was not considered for the purposes of this calculation. Each residential building was assumed to contain approximately 25% one-bedroom apartments, 50% two-bedroom apartments, and 25% three-bedroom apartments. Each one-bedroom apartment was assumed to be 800 square feet, each two-bedroom apartment was assumed to be 1000 square feet, and each three-bedroom apartment was assumed to be 1300 square feet. These dimensions were obtained from information presented in the *Market Analysis for Reuse Planning of the Andover Town Yard a 40R Overlay District* prepared by RKG Associates, Inc. and dated February 10, 2010.

- **Mixed-Use**: Both properties (No. 4 and No. 6) proposed for mixed-use are in the rail corridor portion of the HMD. Per the HMD Design Guidelines created by Gamble Associates, the rail corridor buildings are to be developed as 4 to 5 story buildings. Therefore, for the purposes of this calculation each mixed-use building was assumed to be 4 stories high and assumed to contain retail or restaurant space on the ground floor and residential or office space on the remaining three floors. Under Title V, residential space has a higher design flow than office space. Therefore, residential space was assumed for each mixed-use building as a conservative estimate. Design flow for residential space was calculated in accordance with the residential buildings, as above. The ground floor was assumed to be 25% restaurant space and 75% retail space. Restaurant space was assumed to be 60% customer space with 15 square feet per seat.

- Per Title V, the design flow for retail space is 15 gpd per 1000 square feet and the design flow for restaurant space is 200 gpd per seat.

- **Public Use**: Building No. 1 was assumed to be similar to an office building with lower use of restroom facilities. Therefore, the unit flow was reduced from 75 gpd per 1000 square feet to 50 gpd per 1000 square feet.

Table 3 summarizes the estimated sanitary sewer development loads for each property in Appendix A.

Table 3: Estimated Sanitary Sewer Loads for Proposed Development

No.	Current Use	Future Development	Development Type	Units	Quantity	Unit Flow Factor (gpd/Unit)	Estimated Flow (gpd)
1	Auto Service Station	Public Use: 53,100 sq ft	Public Use	per 1000 sq ft	53.1	50	2,655
2	Bank	Office: 72,000 sq ft	Office	per 1000 sq ft	72	75	5,400
3	Funeral home, houses, and Town Yard Parcel 1	Residential: 75,000 sq ft	Family Dwelling Multiple	Bedroom	142	110	15,620
4	Town Yard Parcel 2 and nearby lots	Mixed-Use: 80,000 sq ft	Mixed-Use	-	-	-	24,250
5	Town Yard Parcel 3	Residential: 35,000 sq ft	Family Dwelling Multiple	Bedroom	66	110	7,260
6	MBTA Parking Lot	Mixed Use: 79400 sq ft	Mixed-Use	-	-	-	19,994
7	Automotive Service Yard	Residential: 90,000 sq ft	Family Dwelling Multiple	Bedroom	169	110	18,590
8	Corner market, dry cleaners, and multifamily housing	Residential: 22,000 sq ft	Family Dwelling Multiple	Bedroom	41	110	4,510
9	Dundee Park	Residential:90,000 sq ft & Office 122,000 sq ft	Family Dwelling Multiple	Bedroom	169	110	18,590
			Office	per 1000 sq ft	122	75	5,400
Total Estimated Flow (gpd)							122,269

Title V design flows represent maximum daily flow values and are considered to be equivalent to 200% of metered average daily water use. Therefore, Title V flows were assumed to be the maximum daily water demand for each property and for the average daily demand the Title V flows were divided in half to convert the flow to average day. In addition, water demands were converted to gallons per minute (gpm) for use in the drinking water system capacity analysis. Table 4 summarizes the estimated water development demands for each property in Appendix A.

Table 4: Estimated Water Development Demands

No.	Current Use	Future Development	Average Daily Demand (gpm)	Maximum Daily Demand (gpm)
1	Library Extension	Public Use: 53,100 sq ft	0.92	1.84
2	Arts Center	Office: 72,000 sq ft	1.88	3.75
3	Town Yard Parcel 1	Residential: 75,000 sq ft	5.42	10.85
4	Town Yard Parcel 2	Mixed-Use: 80,000 sq ft	8.42	16.84
5	Town Yard Parcel 3	Residential: 35,000 sq ft	2.52	5.04
6	MBTA Parking Lot	Mixed Use: 79400 sq ft	6.94	13.88
7	Riverview Development	Residential: 90,000 sq ft	6.45	12.91
8	Railroad St. Development	Residential: 22,000 sq ft	1.57	3.13
9	Dundee Park Extension	Residential:90,000 sq ft & Office 122,000 sq ft	6.45	12.91
			1.88	3.75
Total Estimated Flow (gpm)			42.45	84.90

2. SANITARY SEWER CAPACITY ASSESSMENT

2.1 Introduction

The following sections include a summary of the sanitary sewer model development and model verification as well as an evaluation of the potential impact of the proposed HMD development on the existing sewer capacity.

Flow in the Andover Historic Mill District is conveyed from the south to the north. The Commuter Rail railroad tracks and the Shawsheen River cause distinct divides in the sewer system. There are two railroad crossings along Essex Street near Dundee Park. On the west side of Dundee Park, these two sewer collectors cross each other and then cross the Shawsheen River in aerial crossings. At this junction, various sewers combine and change direction to the north, conveying flow along the west side of the Shawsheen River. Near Powder Mill Square, a double-barreled siphon is used to cross the River from west to east. Two additional railroad crossings occur between Essex Street and North Main Street. All of this flow, from both sides of the river and railroad, combines near 209 N Main Street at another siphon that crosses the Shawsheen River. See Appendix B for a figure of the system.

In December 2017, CDM Smith provided the Town with an infiltration/inflow analysis referred to as *Infiltration/Inflow Flow Monitoring Summary Report (CDM, 2017).* In this report, the Town's sewer collection system was divided into 29 sub-areas for flow monitoring, see Appendix B for a flow chart of the sub-areas. The area of interest for this report, the Historic Mill District, is primarily located in sub-area 2 which is tributary to sub-areas 3, 4, 5, and 16. Further sub-areas are located upstream of these. This report indicates the presence of significant I/I upstream of the HMD, where sanitary flows can more than double due to wet weather. This is a significant impact to the sanitary sewer system's capacity to sewer flows. The addition of sanitary flow due to redevelopment will provide additional loading to the system, however at a lower flow rate. For this reason, a model was developed to identify areas of possible hydraulic deficiently that the Town can target improvements that will increase reliability and capacity of the sanitary sewer system. If requested by the Town, I/I investigations and removal should be conducted by developers in sub-areas 3,4,5 and 16 prior to redevelopment.

2.2 Wastewater Model Development

The wastewater model was developed by Woodard & Curran, utilizing the Town's GIS database of gravity sewers and manholes as well as a review of the Town's record drawings. This information was brought into the Bentley SewerGEMS collection system modeling software, and a new model was created. Based on the Town's GIS, most manhole rim elevations were unknown and therefore, the manhole rim elevations were set to match the ground elevation, which was based on digital terrain elevation information from MassGIS (Bureau of Geographic Information), Commonwealth of Massachusetts Executive Office of Technology Services and Security (EOTSS). This data was also supplemented with record drawings provided by the Town. Ground elevations and invert elevations were modified where necessary to match the record drawings and reasonably correlate with adjacent modeled sewer sections.

2.2.1 Significant Physical Model Assumptions

The Town's GIS database served as the basis for the hydraulic model and in many cases provided information on elevations (invert and ground), material, diameter, and length. However, in a few cases the GIS database did not provide reliable information and assumptions were made by Woodard & Curran. This is typical of most municipal GIS systems as many do not have completed elevation data as it is costly to acquire.

As stated above, most ground elevations were derived from a digital elevation model. Further data gaps were filled in with the use of record drawings. Where sewer record drawings were conflicting or where record drawings were not available, Woodard & Curran estimated sewer capacity by assuming minimum slopes and assumed pipe elevations that would connect to known upstream of downstream elevations. Therefore, the slope (and ultimately sewer capacity)

calculated from the nearest known upstream invert elevation to the nearest known downstream invert elevation, was applied along the full length of unknown invert elevations. In locations where Woodard & Curran could not locate record drawing information and where manholes were deemed critical to the model, a one-day field investigation was performed. The results of the field investigation on November 27, 2018 are documented in Appendix C. Areas 1-5, as shown in Appendix C, should be investigated and repaired, if necessary, prior to redevelopment in the HMD.

2.2.2 Existing Model Flow Conditions

Existing wastewater flows were estimated using recent wastewater metering data from 2017. These wastewater flows were incorporated into the model to represent dry weather flows and wet weather flows. The sections below describe Woodard & Curran's process for flow estimation.

Baseline Sanitary Flows

The existing model flows were based on two sources, the parcel database from MassGIS and the recorded meter data contained in the *Infiltration/Inflow Flow Monitoring Summary Report (CDM, 2017)*. The parcels within the HMD area of interest were exported to a spreadsheet based on MassGIS records. These parcels were sorted by Use Codes which indicated the primary use of the building. Finally, existing flows were assigned based on Title V guidelines and reasonable assumptions about each property. Once flows were assigned per parcel, the flows for each parcel were assigned to the nearest manhole. Several Title V and reasonable assumption examples are as follows:

Single family residences with five or less rooms were assumed to have two bedrooms, six to eight rooms had three bedrooms, and nine to eleven rooms had four bedrooms.

Apartments – four to eight units were based on both the number of rooms as well as the number of units. If the ratio of rooms to units was less than three, then one bedroom per unit was assumed. If the ratio of rooms to units was between three and five, then two bedrooms per unit were assumed; greater than five resulted in three bedrooms per unit.

Office buildings are assigned 75 gallons per day per 1,000 sq ft, according to Title V, and this parameter was used for offices as well as other difficult to quantify public spaces.

Vacant lots and parking lots were assumed to contribute no flow to the sanitary sewer.

The average weekday flows were presented the *Infiltration/Inflow Flow Monitoring Summary Report (CDM, 2017*), refer to Appendix B. These flows and the diurnal pattern associated with them were directly entered into the model for sub-areas 4, 5, and 16. These three locations were specifically chosen when determining the bounds of the model to provide the incoming flows from sub-areas upstream of the HMD.

After all of these baseline sanitary flows were entered into the model, the modeled flows at meter locations 2-1, 3-1, and 3-2 were compared to the average weekday flow reported for those meters. Additional base flow was added to the model to represent sub-areas 14, 15, and 28 which are tributary to sub-area 3. This additional flow was added to match the flow at the meter to the average weekday flow for Meter 3-1. This allowed for an appropriate level of flow correlation at Meter 3-1 between the actual metered flows and the model output.

It was indicated in the CDM report that flows recorded at Meter 3-2 and Meter 2-1 were defective and less than anticipated. Meter 3-2 measured flow on the sewer downstream of a diversion of flow to another sewer system area. Therefore, there were no reasonable locations to introduce flow that could be coming from beyond the modeled system. The *Infiltration/Inflow Flow Monitoring Summary Report (CDM, 2017)* indicated that this meter experienced poor depth/velocity correlation and was modified to align with Manning's equation for capacity. Therefore, no changes to existing flow values were made to calibrate the modeled flow to match the recorded flows at Meter 3-2. The average weekday flow at Meter 2-1 was lower than the modeled flow which echoes the report's findings of "negative" infiltration

for this sub-area. Due to the unusual flow data recorded at these two meters, Woodard & Curran applied a conservative approach, and the flow was not adjusted for sub-area 2.

Wet Weather Flow

Wet weather flow is defined as the measured flow in the sanitary sewer system during and after a precipitation event. The collection system in the HMD area is designed to be a separate sanitary sewer system, meaning that it should not be influenced by rain or groundwater. However, due to defects in the aging system (cracks, fractures, holes, leaking joints) flow in the separated system will increase during precipitation events. Peak flows in the sewer system are typically a result of wet weather flow and usually represent the flows that have the highest likelihood for inducing a sanitary sewer system overflow.

To incorporate wet weather flow into the hydraulic model, Woodard & Curran utilized The *Infiltration/Inflow Flow Monitoring Summary Report (CDM, 2017)*. The metering report describes the elevated flows in the sewer system resulting wet weather precipitation event that occurred on April 25, 2017. This April 25, 2017 storm was used as the basis for wet weather flow response in the hydraulic model. The total precipitation for the event was 0.94 inches with a peak intensity of 0.15 inches per hour. This storm and its subsequent dry weather days were charted and included in Appendix C. The resulting hydrographs (graphs that plot flow vs. time) were used to determine wet weather peaking factors for the metered sub-areas in W&C's model. Each dry weather flow in the model was assigned to a metered region and multiplied by the peaking factor calculated for that region. Table 5 shows the peaking factors for each meter.

Table 5: Wet Weather Peaking Factors

PEAKING FACTORS	Measured Dry Peak (MGD)	Measured Wet Peak (MGD)	Calculated Peaking Factor
METER 2-1	3.4	4.6	1.35
METER 3-1	1	1.7	1.7
METER 3-2	0.4	0.75	1.88
METER 4-1	0.5	0.6	1.2
METER 5-1	1.1	1.45	1.32
METER 16-1	1.25	1.78	1.42

Note that the magnitude of the wet weather flows (4.6 million gallons per day) is much larger than the magnitude of the additional redevelopment sanitary flows (approximately 125,000 gallons per day). Peak flows in the area appear to be due to wet weather events rather than from strictly sanitary loading.

2.2.3 Wastewater Model Redevelopment Flow

The comparison of the existing flows to the proposed redevelopment scenario for the HMD is primarily focused on nine properties as described in Section 1.4

Table 6 below shows the comparison between the estimated existing flows and potential redevelopment flows. If all nine of the proposed redevelopments occur as planned, approximately 106,200 gallons per day of additional peak sanitary flow will be generated within the HMD. The portion of additional peak sanitary flow for each development is outlined in Table 6 below.

Table 6: Comparison of Redevelopment Flows to Existing Flows

No.	Current Use	Current Estimated Flow (gpd)	Future Development	Development Type	Redevelop-ment Flow (gpd)	Increase in Flow (gpd)
1	Auto Service Station	750	Library Extension Public Use: 53,100 sq ft	Public Use	2,655	1,905
2	Bank	862	Office: 72,000 sq ft	Office	5,400	4,538
3	Funeral home, houses, and Town Yard Parcel 1	1,007	Residential: 75,000 sq ft	Family Dwelling Multiple	15,620	14,613
4	Town Yard Parcel 2 and nearby lots	1,630	Mixed-Use: 80,000 sq ft	Mixed-Use	24,250	22,620
5	Town Yard Parcel 3	0	Residential: 35,000 sq ft	Family Dwelling Multiple	7,260	7,260
6	MBTA Parking Lot	0	Mixed Use: 79400 sq ft	Mixed-Use	19,994	19,994
7	Automotive Service Yard	900	Riverview Development Residential: 90,000 sq ft	Family Dwelling Multiple	18,590	17,690
8	Corner market, dry cleaners, and multifamily housing	840	Railroad St. Development Residential: 22,000 sq ft	Family Dwelling Multiple	4,510	3,670
9	Dundee Park	10,118	Dundee Park Extension Residential:90,000 sq ft & Office: 122,000 sq ft	Family Dwelling Multiple	18,590	13,872
				Office	5,400	
				Total Estimated Flow (gpd)	122,269	106,162

2.3 Wastewater Model Evaluation & Findings of Existing Infrastructure

In order to determine the potential impact of additional sanitary loading due to redevelopment, four scenarios were analyzed. The scenarios focused on the hydraulic performance of existing infrastructure under dry weather (sanitary loading), wet weather (sanitary loading multiplied by wet weather peaking factors), and the corresponding redevelopment dry weather and wet weather scenarios. These four scenarios are summarized below:

Table 7: Modeled Scenarios

Scenario Number	Baseline Scenario	Weather Impact	Scenario Notes
1	Existing	Dry Weather	Match metered flows
2	Existing	Wet Weather	Apply wet weather peaking factors
3	Redevelopment	Dry Weather	Revise flows for redevelopment properties
4	Redevelopment	Wet Weather	Revise flows for redevelopment properties, then apply wet weather peaking factors

2.3.1 Evaluation Criteria

Each flow scenario was evaluated against the existing collection system. Hydraulic grade lines (elevations of the sewer system) were reviewed to determine hydraulic performance during each flow scenario.

Two criteria were used to perform this evaluation:

1. Freeboard Depth: The height from the ground elevation to the level of the sewer in the manhole. A minimum freeboard depth of 4-feet is a conservative measure that will provides an appropriate factor of safety against sanitary

sewer overflow. This means that if the sewer is within 4-feet of the ground surface it was flagged as hydraulically deficient during each scenario.

2. Depth of Sewer In Pipe (d/D or depth divided by diameter): The height of the wastewater in the sewer pipe. This describes if the sewer pipe is surcharged and reduces sewer service capacity. For example, if an 8-inch pipe has 8-inches of flow, d/D would be equal to 100%.

Together these indicate whether the Town of Andover describe potential locations for sewer capacity deficiencies in the system.

2.3.2 Analysis of Existing Flow Condition on Sewer System Capacity

Woodard & Curran's analysis of the existing system under current flow conditions on the sewer system capacity, neither dry, nor wet weather result in sanitary sewer overflow. Results of the analysis showing the hydraulic performance of the existing system as described in the section above are included in Appendix D. These figures show locations in the system where the freeboard is less than four feet or between four and eight feet. The HMD area contains sewers that are in some instances only 5-feet deep and are inherently have minimal freeboard. For example, Figure 2 shows the profile of the water level along the Dundee Park area which shows sewers less than 4-feet in depth and result in risk of sanitary sewer overflow. The depth of these sewers should be field verified and may be deeper in actuality that depicted in the model. Shallow manholes are marked in Appendix D by a green circle.

Dry Weather: Dry weather flow through the HMD area is estimated to be approximately 4.25 MGD including upstream sewer basins. Based on modeling, there are no hydraulic deficiencies under current dry weather flows.

Wet Weather: The model results indicate that peak flow occurs during wet weather events. Specifically, the storm on April 25, 2017 produced approximately 6.4 MGD of flow through the area. This is approximately 1.5 times greater than dry weather flow. Based on modeling, there appear to be minimal hydraulic deficiencies under current wet weather scenarios. The Town's interceptor sewers in the area run near 80% full during the storm above, however no overflow is expected.

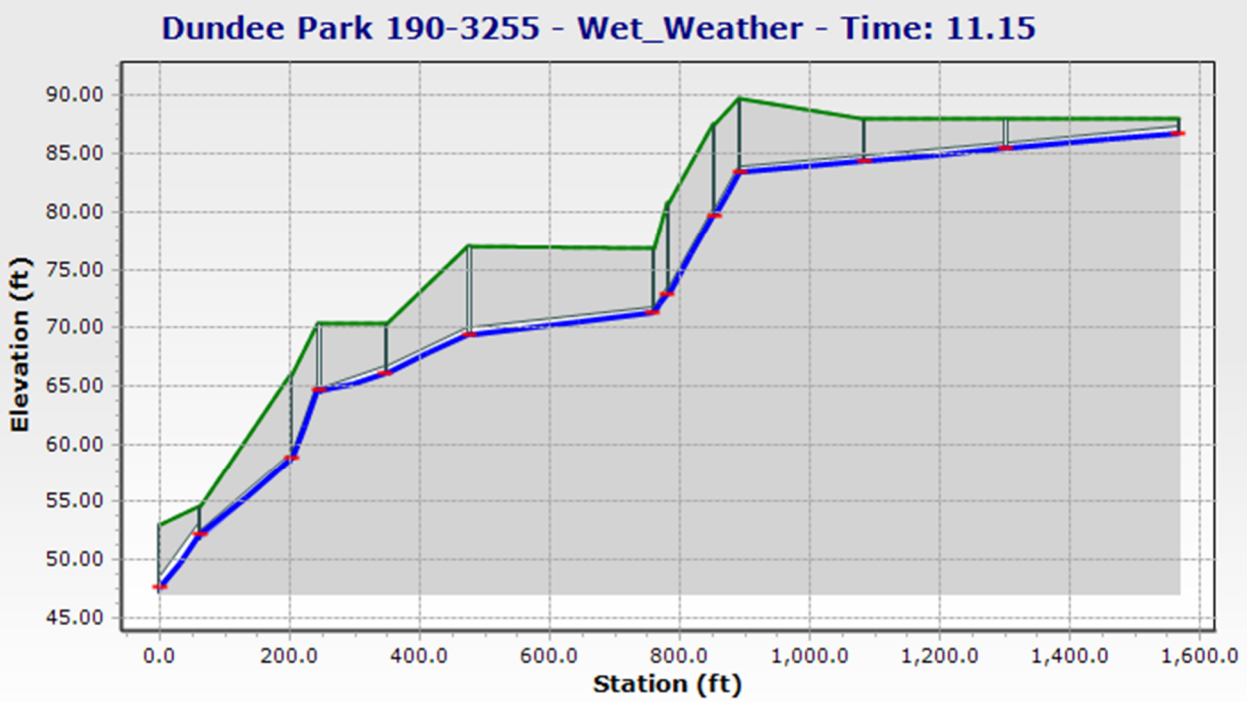
2.3.3 Analysis of Redevelopment on Sewer System Capacity

Woodard & Curran's analysis of the existing system under proposed redevelopment flow conditions on the sewer system capacity, neither dry, nor wet weather result in sanitary sewer overflow.

Dry Weather: Dry weather flow through the HMD area is estimated to be approximately 4.36 MGD (an increase of 2%) including upstream sewer basins. Based on modeling, there are no hydraulic deficiencies under proposed dry weather flows. However, based upon actual locations of the proposed sewer service connections, individual pipe sections may require upgrade. Particularly, 6-inch sewers located on Lewis Street, Railroad Street, Pearson Street, Essex Street, School, and Ridge Street, should be considered for replacement if proposed redevelopment occurs (See Appendix D for location of 6-inch Sewers and record drawings). In addition, CCTV should be performed on the 6-inch sewer lines in the streets listed above if they are within the gas impacted area.

Wet Weather: The model results indicate that peak flows in the sewer system will occur due to wet weather, rather than the addition of new flows from redevelopment. Wet weather flow through the HMD area is estimated to be approximately 6.6 MGD (an increase of 1% from pre-redevelopment wet weather flows). The Town's interceptor sewers in the area run near 80% full during the storm above, however no overflow is expected.

Figure 2: Profile of Dundee Park Shallow Sewers



2.4 Conclusions & Recommendations

The planning level model indicates no major overflow locations if the redevelopment within the HMD occurs as planned. Under dry weather conditions, the additional flows due to redevelopment of the HMD area result in an increase of approximately 106,000 gallons per day or 2% of the average flow passing through the area. Peak sewer flows in the area are due to wet weather induced sewer flows. The sewer interceptors located in the HMD area (along the Shawsheen River) appear to have adequate capacity Smaller sewers, located on side streets currently servicing few users, may be considered for replacement due to capacity or future maintenance related challenges.

The following should be considered if the proposed redevelopment occurs in the HMD area:

1. Town should consider performing a detailed review of developers proposed utility plans indicating exact locations of sewer service connections as the proposed sewer connections may differ than what is considered in the hydraulic model.

2. The HMD area contains approximately 4,770 linear feet of 6-inch mainline vitrified clay sewers (see Appendix D for locations of sewer and record drawings). If developers plan to connect to the 6-inch mainline sewers, it is recommended they be removed and replaced with new SDR35 PVC no smaller than 8-inch in diameter for the reasons listed below. In addition, Woodard & Curran recommends that CCTV be performed on the sewers located within the gas impacted area and conduct any necessary repairs.

    a. The 6-inch mainline sewers have been in service for more than 100 years and may be in poor condition.

    b. 6-inch sewers can be costly to clean, inspect, and maintain due to grease, debris, and root buildup.

c. 6-inch sewers are difficult to repair trenchlessly due to their small size which limits the ability of trenchless robots to navigate bends or intruding service connections. Therefore, replacement of these sewers should be considered. Replacement may be more cost-effective if coordinated with other utility replacement and prior to street paving.

d. A budgetary cost of approximately $560-$600/LF to replace these sewers is estimated based on recent bid prices in the Merrimack Valley. If all of these sewers require upgrade, a total construction cost of approximately $2,100,000 is expected.

3. In order to increase reliability of the sewer service in the area, a pipe CCTV inspection should be performed and surface-level manhole condition assessment that conforms to the National Association of Sewer Service Companies (NASSCO) standards of all sewer lines within the HMD and repair deficiencies as needed. This will help to identify sewers that need to be updated because of structural failure even if they are adequate hydraulically. It is expected that the sewer system in the area is near the end of its useful life due to much of the system being constructed of vitrified clay pipe likely greater than 50-100 years old.

4. It may be prudent for the Town to reduce upstream infiltration and inflow which causes peak flows in the HMD area. The 2017 I/I Metering report should form the basis of the Town's I/I removal program, and basins upstream of the HMD (e.g. sewer basins 3, 14, 28) should be targeted to increase sewer capacity in the HMD area and reduce flows to the Greater Lawrence Sanitary District. In addition, Woodard & Curran recommends that new developments within these basins be responsible for moving to 4:1 removal.

5. Per the November 27, 2018 field visit, manhole SMH-210, where GIS shows two existing outflow pipes, may only have one functioning outflow pipe. This was based on discussion with Town collection systems staff. If this sewer is truly non-functioning, then the 24-inch concrete relief sewer may not provide sufficient capacity. This non-function/blocked pipe should be inspected and rehabilitated via cleaning, chemical root treatment, or other method prior to the redevelopment of the area. Much of the sewer is located off-road which may cause inspection challenges.

6. Per the November 27, 2018 field visit, a potentially large source of inflow was located at 8 Essex Street. Upon inspection of MH1494, a large, clear, continuous discharge of water via unknown PVC connection was located. It is possible that roof drains from nearby buildings may be plumbed directly into the sanitary collection system. Prior to the redevelopment of this area, this connection should be confirmed via dye test.

7. According CDM report, "Andover MA Sanitary Sewer Master Plan, June 2016", Dascomb Road Development future tributary flow is estimated 778,00 gpd (including possible I/I) to the collection system. CDM conducted a hydraulic model based on dry and wet weather scenario incorporating the new development flows. The dry weather results show the Shawsheen Interceptor has the capacity. But wet weather results show a significant capacity issue upstream and downstream from the HMD connection point. The report states to allow any new future development flows, the sanitary collections system would require 1.5 MGD I/I flow removal. Refer to figures 4-5 and 4-6 in Attachment I from the CDM report showing the scenarios of future peak weather flow and 40% reduction area inflow.

Summary of estimated unit costs for each consideration are provided in the table below:

Table 8:Estimated Costs for Wastewater Recommendations

Summary of Recommendations			
Item	**Description**	**Unit**	**Unit Cost**
Review Developer Plans	Review developer utility plans.	Per Development	$5,000.00
Replace 6-inch Sewers	Provide 8-inch or larger sewers downstream of new connections.	Per Foot	$600.00
HMD Area Pipe CCTV Inspection	Perform condition assessment of sewers in HMD Area	Per Foot	$2.50
HMD Area Manhole Inspection	Perform condition assessment of manholes in HMD Area.	Per MH	$100.00
Perform SSES Upstream of HMD Area	Perform SSES upstream of HMD Area to Locate I/I.	See Appendix E - CDM Recommendations	
Rehabilitation of Basins Upstream of HMD Area	Remove upstream infiltration/inflow to increase capacity in HMD Area.	See Appendix E - CDM Recommendations	
Cleaning and Inspection Interceptor Sewer Downstream of SMH-210	Per collection system operator, eastern sewer may be blocked with roots/debris. Should be cleaned and inspected. Approximately 1,700 feet.	$10,000	
Perform Building Inspection at 8 Essex Street	Locate source of unknown clear water connection and recommend rehabilitation.	$10,000	

3. DRINKING WATER SYSTEM CAPACITY ASSESSMENT

3.1 Introduction

The following sections include a summary of the water distribution hydraulic model calibration in the HMD area and an evaluation of the potential impact of the proposed development on the distribution system.

3.2 Existing Conditions

3.2.1 System Overview

The Town of Andover's existing distribution system is comprised of the following:

- Three pressure zones:
  - West High-Pressure Zone (Wood Hill)
  - Bancroft Low-Pressure Zone (Prospect Hill)
  - East High-Pressure Zone
- Water Treatment Plant with separate pumped service to the West High-Pressure Zone and Bancroft Low-Pressure Zone
- One pump station
  - Bancroft Pump Station, located adjacent to the Bancroft Reservoir and supplying the East High-Pressure Zone
- Approximately 250 miles of water mains ranging from 4-inches in diameter to 24-inches in diameter.
- Five storage tanks with total storage capacity of 14 MG, and
- One active interconnection:
  - Supply to North Reading, located in the East High-Pressure Zone.
  - No other interconnections were modeled for this study

The Town provides potable water by treating water from Haggetts Pond (supplemented by the Merrimack River and Fish Brook) at the drinking water treatment plant (WTP) located at 397 Lowell Street. Finished water is pumped to the West High-Pressure Zone and the Bancroft Low-Pressure Zone by respective pumping systems. The Bancroft Pump Station pumps water from the Bancroft Low-Pressure Zone to the East High-Pressure Zone.

As shown in Appendix F, the HMD is located in the northeastern portion of the Bancroft Low Pressure Zone along the Shawsheen River.

3.2.2 Hydraulic Model

For this drinking water system capacity assessment, Woodard & Curran utilized the hydraulic model developed by CDM Smith in 2016. The Town provided this model to Woodard & Curran in 2017 to conduct a limited calibration of the model. Woodard & Curran used SCADA information and field pressure data loggers to record distribution system flow and pressure conditions and determine hydraulic grade lines for two transmission mains, a 16-inch diameter main that takes a northern route through the Town and a transmission main varying in size from 16-inch to 24-inch diameter located to the south.

Woodard & Curran made minor modifications to the hydraulic model to correlate the predicted and observed distribution system conditions, including changes to the WTP operating schedule and pump data. In order to further validate the hydraulic model, Woodard & Curran conducted additional calibration work focused on the vicinity of the HMD as described in this report section. No additional updates or calibrations of the hydraulic model have otherwise been conducted.

The portion of the HMD represented by the hydraulic model consists of approximately 110 junction nodes and 145 pipe segments ranging from 6-inch to 12-inch in diameter, making up approximately 3.3 miles of distribution piping. The water mains within the HMD generally consist of the lined and unlined cast iron pipe (CIP) and ductile iron pipe (DIP). A figure which depicts the water main diameters within the vicinity of the HMD is included in Appendix G. Table 9 provides the approximate installation year and the pipe materials that are located in the HMD and Appendix G depicts the pipe materials that are located in the HMD.

Table 9: HMD Water Main Material and Installation Year

Pipe Material	Year of Installation
Unlined CIP	Unknown, late 40's, early 50's, early 70's
Lined CIP	Unknown, late 40's, early 50's
Lined DIP	Unknown, early 2000's

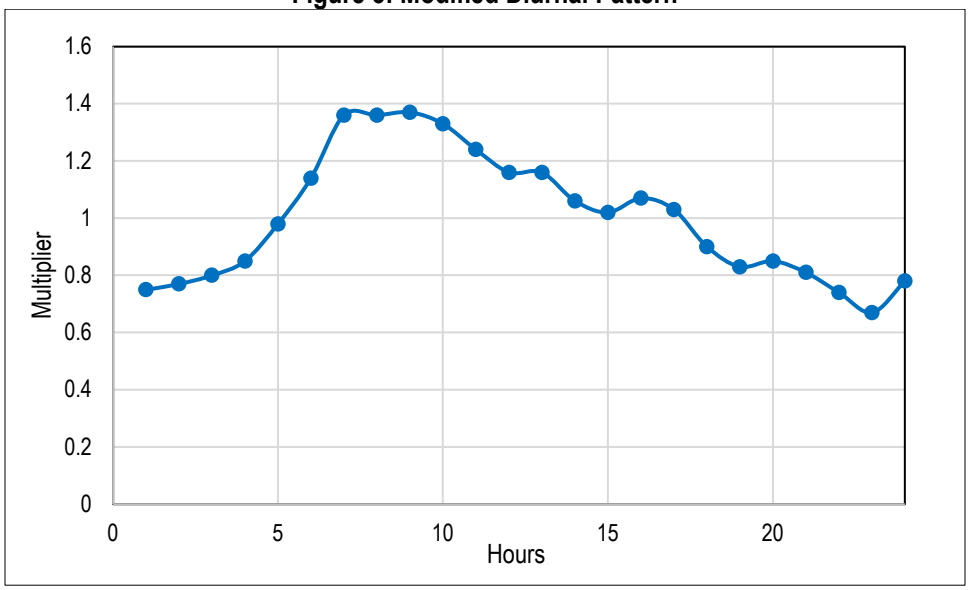
3.2.2.1 Existing Demands

The hydraulic model is configured to use Average Day Demand (ADD) and Maximum Day Demand (MDD).
In the model, Andover's Average Day Demand (ADD) is 5.4 million gallons per day (MGD). Woodard & Curran reviewed this demand against SCADA logs in 2017 and found this to be representative of existing conditions.

Based on the Town of Andover's Annual Statistical Reports in the past 5 years (2013 through 2017), the recorded MDD was 1.8 to 2.1 times greater than the recorded ADD. A new MDD was developed for this model using a multiplier of 2.1 applied to the ADD for each node to calculate the existing MDD (11.3 MGD) for this analysis.

A typical residential diurnal pattern was incorporated into the hydraulic model received by Woodard & Curran. Woodard & Curran developed the diurnal pattern shown in Figure 3, which is a modified residential diurnal pattern developed for Andover, MA. The modified diurnal pattern was applied to demands to represent the variance of water usage throughout the day for use in extended period simulation modeling.

Figure 3: Modified Diurnal Pattern



3.3 Hydraulic Model Calibration

A limited C-Factor calibration was performed to validate the information used in the hydraulic model for the vicinity of the HMD. Data for model calibration was collected from hydrant flow testing and pressure data logger within the vicinity of the HMD.

Corrosion and the deposition of sediment (tuberculation) on the interior of unlined cast iron water mains is the chief cause of reduced carrying capacity. Corrosion and tuberculation greatly increase the roughness of the pipe's interior wall, resulting in increased friction coefficient and a corresponding resistance to flow, which reduces the hydraulic capacity of the water main.

The Hazen-Williams friction coefficient, or C-Factor, is used to define the relative hydraulic condition of the water mains. According to the Hazen-Williams formula, the lower the C-Factor, the lower the carrying capacity of a pipe. Typical C-Factors for new cement-lined iron pipes are in the range of 120-140. Older, unlined cast iron pipes can have C-values lower than 40. Pipes with a C-value of 60 have a flow capacity of only half of a newer or lined pipe with a C-value of 120.

3.3.1 Fire Flow Testing

Hydrant flow tests were conducted to assess the capacity for fire suppression (fire flow) and to provide flow and head loss data for use in calibrating pipe C-Factors within the vicinity of the HMD.

On November 8, 2018, Woodard & Curran performed seven hydrant flow tests. Two hydrants were selected for each hydrant test: one flow hydrant and one gauge hydrant, located in the vicinity of the flow hydrant. Static pressures were recorded at both hydrants. Residual pressures were recorded at the gauge hydrant, while a pitot flow measuring device was used to measure flow at the flow hydrant. In addition, the Town provided results for a fire flow test conducted on October 20, 2018 at Dundee Park, which is a private road within the vicinity of the HMD, recorded as Fire Flow Test No. 8 for the purposes of this study. Table 10 lists the measurements obtained from the hydrant tests. Appendix H shows the location of the fire flow tests.

Table 10: Fire Flow Test Data – October & November 2018

Test No.	Location	Flow Hydrant		Residual Hydrant		Model Nodes
		Flow (gpm)	Static (psi)	Static (psi)	Residual (psi)	
1	Buxton Court	1427	100	90	76	HYD-2006, HYD-1787
2	Across from Public Safety & 96 N Main Street	1330	80	85	74	HYD-1788, HYD-1786
3	30 & 40 Railroad Street	1185	105	104	82	HYD-2001, HYD-1773
4	20 & 53 Essex Street	1351	82.5	100	94	HYD-2001, HYD-1773
5	Intersection of Ridge Street and School Street; Between 11 & 19 Lupine Road	-[1]	96	103	70	HYD-1598, HYD-2004
6	30 Whittier Court; Youth Center	1092	67.5	66	57	HYD-1903, HYD-1904

Test No.	Location	Flow Hydrant		Residual Hydrant		Model Nodes
		Flow (gpm)	Static (psi)	Static (psi)	Residual (psi)	
7	200 Lowell Street; Intersection of Lowell St and W. Parish Drive	1185	75	74	64	HYD-1354, HYD-1355
8[2]	Dundee Park	730	-[3]	103	98	HYD-2030, HYD-2029

Notes:
psi – pounds per square inch
gpm – gallons per minute
1. Flow hydrant pitot reading was less than 100 gpm.
2. Test No. 8 was conducted by the Town on October 20, 2018.
3. Static pressure data at the flow hydrant was not provided to Woodard & Curran.

3.3.2 Datalogger Pressure Recording

In addition to the hydrant flow testing data, pressure data loggers were installed on hydrants at two locations to monitor pressures during flow testing conducted by Woodard & Curran on November 8, 2018. Table 11 summarizes the pressure data collected in the field by the data loggers.

Table 11: Pressure Data Collected November 2018

Location	Pressure Range (psi)	Average Pressure (psi)
AND-1: N Main Street and Railroad	96.2 – 109.5	105.1
WC-A: N Main Street and Sweeney Court	74.3 – 95.7	89.4

Notes:
psi – pounds per square inch

The dataloggers recorded pressures in the distribution system that were fairly stable during normal system operation. However, as expected, each of the loggers recorded varying degrees of pressure drops when hydrant flow tests were being conducted at nearby locations. The pressure readings obtained from these dataloggers helped to validate the calibrated model results.

3.3.3 C-Factor Calibration

To obtain an accurate representation of the current condition of the interior of the distribution system mains within the HMD, it was necessary to adjust model parameters, specifically C-Factors, to match results of field testing. The Innovyze Infowater software extension Calibrator was used to run an initial calibration and determine Hazen-Williams C-Factors for various pipe materials separated by calibration group. The water distribution system within the HMD was divided into five calibration groups based on pipe material and diameter, as shown in Table 12.

Hydrant flow tests were imported into the Calibrator module for calculation, with the following exceptions. Flow tests No. 4 and No. 6 were not use during calibration because there was not a minimum 10 psi pressure drop (static pressure minus residual pressure) at the residual hydrant per American Water Works Association (AWWA) M32. Flow Test No. 5 was not used during calibration because the pitot reading at the flowing hydrant was recorded as 0 psi. Flow Test No. 7 was not input into the calibrator module because it was not within the HMD. The data from flow Test No. 7 was used as supplemental data to verify the calibrated C-factor results.

Each calibration group was assigned a range of C-Factors depending on the pipe material and expected results. Table 12 lists the C-Factor range restrictions that were entered into the model for each pipe material and pipe size. Refer to Appendix G for a depiction of C-Factors for the water mains within the vicinity of the HMD.

The correlation between the calculated C-Factor and the input data is expressed by the fitness value. The results below had a fitness value of 0.0004, which indicates a strong correlation between the model C-Factors and the field-collected fire flow data.

Table 12: HMD C-Factor Calibration Results

Calibration Group by Pipe Material	Diameter (inches)	C-Factor Range	Calibrated C-Factor
Unlined Cast Iron	6	30 - 70	36
Lined Cast Iron and Ductile Iron	6	60 - 110	93
Lined Cast Iron and Ductile Iron	8 & 10[1]	60 - 120	67
Unlined Cast Iron	10 & 12[2]	30 - 90	38
Lined Cast Iron and Ductile Iron	12 & 16[3]	110 - 130	121

Notes:
1. There were two 10-inch lined Cast Iron pipes within the HMD; therefore, these pipes were incorporated with the 8-inch lined pipes.
2. There were two 10-inch unlined Cast Iron pipes within the HMD; therefore, these pipes were incorporated with the 12-inch unlined pipes.
3. Due to the small number or 12-inch and 16-inch lined pipes (approximately 21 pipes total), these two diameters were combined in the model calibrator program.

The C-Factors for the 6-inch, 10-inch, and 12-inch diameter unlined cast iron water mains were modeled to be very low following calibration, which could be a result of pipe condition or a potentially closed valve in the distribution system.

3.3.4 Model Validation

Following C-Factor calibration, Woodard & Curran performed a model validation using the Town's distribution system operational records for the day of the testing, hydrant flow test data, and pressure logger data. The Town stated that two of the WTP Finished Water pumps were in operation during the testing period, producing 8.72 MGD.

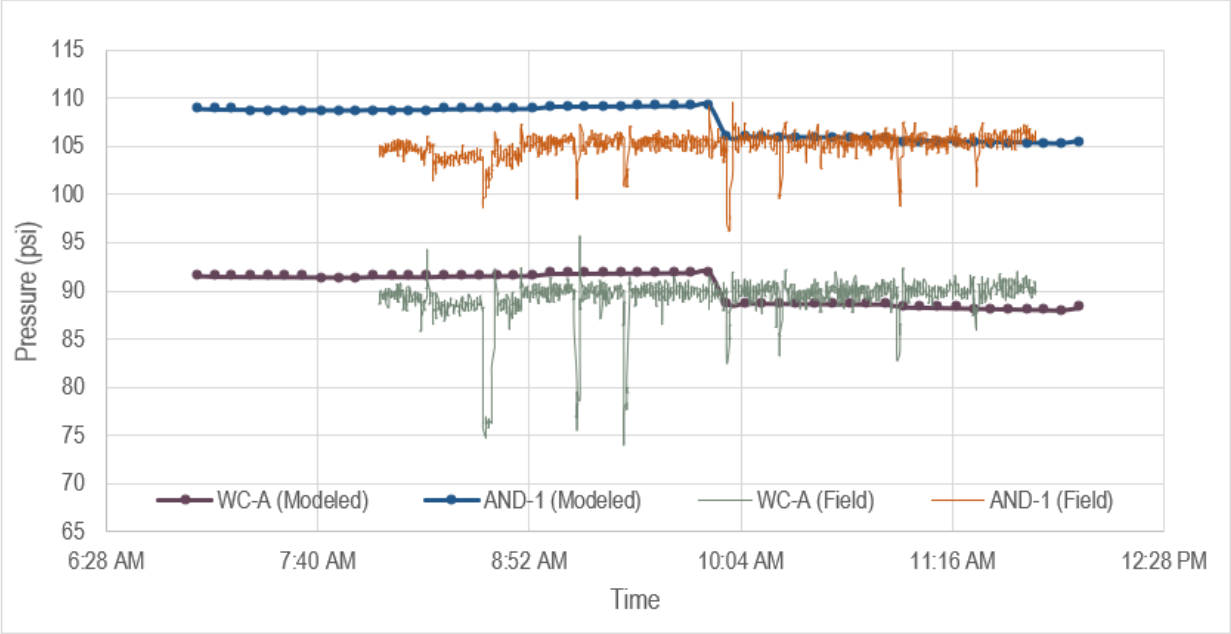
A steady state scenario in the hydraulic model was created with these parameters, using the calibrated C-Factors discussed in the previous section. The scenario was run separately for each of the fire flow tests utilized for model calibration and the pressures at locations AND-1 and WC-A were compared to the pressure logger data. Table 13 summarizes the model results.

Table 13: Pressure Logger Data vs Model Results Comparison

Test No.	AND-1: N Main Street and Railroad	WC-A: N Main Street and Sweeney Ct
Average Field Pressure (psi)	105.1	89.4
Modeled Pressure during Test 1 (psi)	**106.5**	**85.3**
Pressure Difference (psi)	1.4	4.1
Percentage Difference (%)	1.4%	4.7%
Modeled Pressure during Test 2 (psi)	**107.1**	**86.4**
Pressure Difference (psi)	2.0	3.0
Percentage Difference (%)	1.9%	3.4%
Modeled Pressure during Test 3 (psi)	**106.4**	**89.4**
Pressure Difference (psi)	1.3	0.0
Percentage Difference (%)	1.3%	0%
Modeled Pressure during Test 8 (psi)	**109.3**	**92.0**
Pressure Difference (psi)	4.2	2.6
Percentage Difference (%)	3.9%	2.8%

In addition, Figure 4 depicts the pressure readings obtained in the field and the modeled dataloggers pressure readings under ADD conditions.

Figure 4: Pressure Logger Data Field vs Model Results



Based on the model validation, there was a satisfactory correlation between the model results under the steady state scenario described above and the field data collected.

3.4 Evaluation & Findings

3.4.1 Evaluation Criteria

Typical and recommended operating standards, such as those provided by the American Water Works Association (AWWA), are used to ensure that an adequate level of service is provided to all customers at all times. These criteria were used to evaluate the results obtained from the hydraulic model. The evaluation criteria include fire protection, system pressure, pipe velocity, and head loss, each of which are discussed below.

Fire Protection
The ability to provide required sufficient and sustained water flows and pressures during a fire emergency is a key goal for all water providers. The AWWA recommends that fire flows be evaluated for MDD conditions instead of ADD or peak hour demand conditions. This ensures that sufficient flow is available during high usage periods and systems are not excessively oversized. Guidance from the Insurance Services Office (ISO) stipulates that needed fire flow shall not be less than 500 gpm, with increasing values based on the type of building being assessed. Therefore, available fire flow was evaluated at junctions in the system while maintaining a pressure of 20 pounds per square inch (psi) throughout the rest of the distribution system.

System Pressure
The AWWA recommends maintaining a minimum pressure above 20 psi under all operating conditions. Pressures above 20 psi should be available during all types of demand conditions and scenarios.

In addition, the Massachusetts Department of Environmental Protection Bureau of Resource Protection Drinking Water Program states the following pressure requirements in Part 9.3 of "Guidelines for Public Water Systems":

1. *The normal working pressure in the distribution system should be approximately 60 – 80 psi and not less than 35 psi.*

    a. *Pressure - All water mains, including those not designed to provide fire protection, shall be sized after a hydraulic analysis based on flow demands and pressure requirements. All service connections shall have a minimum residual water pressure at street level of at least 20 pounds per square inch under all design conditions of flow.*

Pipe Velocities
Establishing a recommended range for velocities within the piping network is related to maintaining system water quality. Pipes with inadequate velocities may result in decreased water quality, low chlorine residual, and/or bacterial growth. Excessive velocities can result in high head loss due to pipe friction and subsequent loss of system pressure and high pumping energy requirements. The AWWA recommends a maximum velocity of 5 feet per second (fps) and a minimum of 0.1 fps at ADD condition. These standards are based on sustained velocities for long stretches of pipe.

Headloss
Pipes with high head loss generally cannot deliver optimum pressures to downstream customers. These pipes may be undersized for the amount of water flowing through them, have excessive velocities, or excessive frictional headloss caused by interior surface roughness (low C-Factor). The AWWA recommends a range of 2 to 5 feet of headloss per 1,000 feet of pipe (ft/k-ft) for transmission mains and a maximum head loss of 10 ft/k-ft for distribution pipes.

3.4.2 Evaluation Scenarios

Two evaluation scenarios (Scenarios 2A and 2B) were created to assess impacts to water distribution infrastructure as a result of the redevelopment of nine parcels within the HMD. These scenarios were compared to the existing HMD

area distribution system conditions (Scenarios 1A and 1B). Table 14 describes each scenario and its respective distribution system and demand conditions.

Table 14: Model Scenarios

Scenario Number	Scenario Description	Distribution System Condition	Demand
1A	No Redevelopment (Baseline)	Existing HMD	Average Day Demand (5.4 MGD)
1B	No Redevelopment (Baseline)	Existing HMD	Maximum Day Demand (11.3 MGD)
2A	Redeveloped HMD	Estimated demands were added to the model junction located closest to each of the nine proposed redeveloped properties within the vicinity of the HMD	Average Day Demand (5.46 MGD)
2B	Redeveloped HMD	Estimated demands were added to the model junction located closest to each of the nine proposed redeveloped properties within the vicinity of the HMD	Maximum Day Demand (11.42 MGD)

The hydraulic model utilized an Extended Period Simulation (EPS) analysis, in order to evaluate variations in demand, pumping rate, tank levels, or pressures that occur during each scenario, with the following exception. The fire flow analysis under maximum day conditions utilized steady state. The hydraulic model requires additional system calibration in order to allow the model to run system-wide in EPS for fire flow analysis under MDD. Woodard & Curran evaluated the model results for impacts to pressures, velocity, head loss, and available fire flow within the HMD. This evaluation was limited to an assessment of impacts to the water distribution infrastructure within the HMD and select water distribution infrastructure in close proximity to the HMD. The results are summarized for each scenario in the following section.

3.4.1 Results Discussion

Scenario 1A No Redevelopment (Baseline ADD)

Scenario 1A conditions consisted of the existing distribution system under ADD. Two of the WTP Finished Water Pumps serving the Bancroft Low-Pressure Zone were simulated as operating for approximately 12 hours per day, turning on each day at 6:00AM and off at 6:00PM. In addition, a third WTP Finished Water Pump was simulated as operating 2 hours each day between 8:00AM and 10:00AM to meet peak daily demand. Under ADD conditions, approximately 0.45 MGD is transported through the interconnection on Central Street to North Reading.

Under Scenario 1A, modeled pressures ranged from approximately 70 to 120 psi across all junctions in the HMD during various times of the day. Velocities in the vicinity of the HMD ranged from 0 ft/sec to approximately 3.3 ft/sec. In addition, head loss in the vicinity of the HMD ranged from 0 ft/k-ft to approximately 3.2 ft/k-ft, with two exceptions. Head loss in the 6-inch water main on the southern portion of Stevens Street near Shawsheen Road was predicted to be approximately 17 ft/k-ft. Head loss in the 8-inch water main on Central Street between Red Springs Road and Lupine

Road was predicted to be approximately 6.9 ft/k-ft. Both mains, the 6-inch on Stevens Street and the 8-inch on Central Street, are modeled as unlined cast iron water mains with a calibrated C-Factor of 36.

Scenario 1B *No Redevelopment (Baseline MDD)*
Scenario 1B conditions consisted of the existing distribution system under estimated MDD. Two of the WTP Finished Water Pumps serving the Bancroft Low-Pressure Zone were simulated as operating 24 hours per day. In addition, the third Finished Water Pump, serving the same zone, was simulated as operating for 1 hour around midnight each day. Under MDD conditions, approximately 1.38 MGD is transported through the interconnection on Central Street to North Reading.

Under MDD conditions, modeled pressures across all junctions in the vicinity of the HMD ranged from approximately 72 to 129 psi during various times of the day. Velocities in the vicinity of the HMD ranged from 0 ft/sec to approximately 2.6 ft/sec. In addition, head loss in the vicinity of the HMD ranged from 0 ft/k-ft to approximately 4.5 ft/k-ft, with one exception. Head loss in the 6-inch water main on the southern portion of Stevens Street near Shawsheen Road was predicted to be approximately 11.1 ft/k-ft.

Available Fire Flow (AFF) within the HMD was analyzed under MDD conditions at peak demand and under MDD conditions at the base maximum day demand (no diurnal curve multiplier). Under both analyses the Bancroft Pump Station was offline and the range of AFF across all junctions in the vicinity of the HMD was approximately 500 gpm to 10,000 gpm.

Scenario 2A – Redeveloped HMD under ADD
Scenario 2A conditions were the same as Scenario 1A, except the estimated development demands, refer to Section 1.4.1, were added to the hydraulic model at the junctions closest to the locations of the proposed developments.

Under Scenario 2A, predicted pressures within the HMD were noted to decrease slightly when compared to existing conditions and all junctions within the HMD remained above 35 psi. Under Scenario 2A, a slight increase was predicted to head loss and velocity in the water mains in the vicinity of the HMD as a result of the simulated redevelopment. However, head loss and velocity in water mains which had predicted values below guidelines under baseline conditions remained below guideline values under Scenario 2A.

Scenario 2B – Redeveloped HMD under MDD
Scenario 2B conditions were the same as Scenario 1B, except the estimated development demands, refer to Section 1.4.1, were added to the hydraulic model at the junctions closest to the locations of the proposed developments.

Under Scenario 2B, predicted pressures within the HMD were noted to decrease slightly when compared to existing conditions and all junctions within the HMD remained above 35 psi. Under MDD, a slight increase was predicted to head loss and velocity in the water mains within the HMD as a result of the simulated redevelopment. However, head loss and velocity in water mains which had predicted values below guidelines under baseline conditions remained below guideline values under Scenario 2A.

Under Scenario 2B, predicted AFF was projected to be reduced by 300 gpm or less at junctions in the vicinity of the HMD with the simulated redevelopment compared to existing conditions. All junctions, which were projected to be reduced by 100 gpm or more had a predicted AFF of greater than 5,000 gpm under baseline conditions. In addition, all junctions had a predicted AFF value at or above the ISO recommended 500 gpm. However, the ISO recommended 500 gpm is a minimum value for single-family spaced-out development. The needed fire flow for multi-family, mixed-use, commercial, and public space developments would be greater than 500 gpm. Woodard & Curran was not provided with the needed fire flow requirements for the proposed redeveloped properties or properties within the HMD. Therefore, Woodard & Curran assumed a needed fire flow of 2,500 gpm for properties within the HMD. However, the

needed fire flow for each property within the HMD should be verified with the Town's Fire Department to determine that adequate AFF is provided at all locations. Woodard & Curran analyzed AFF within the HMD under MDD conditions at peak demand and under MDD conditions at the base maximum day demand (no diurnal curve multiplier). Approximately one-third of the fire flow junctions in the vicinity of the HMD had a predicted AFF of less than 2,500 gpm, refer to figures in Appendix G. In general, locations with predicated AFF below 2,500 gpm correlated with water mains which had low C-Factors or were small diameter mains. In addition, the hydraulic model was updated with the recommended water main improvements in the HMD and the water main improvements that the Town will conduct in the gas affected area and Scenario 2B was reevaluated. In general, with the improvements mentioned the predicted AFF on public roadways in the vicinity of the HMD was at or above 2,500 gpm.

3.4.2 Findings & Recommendations

Based on the hydraulic modeling analysis comparing Scenarios 2A and 2B to existing conditions (Scenarios 1A and 1B), it was determined that the additional estimated domestic demands of proposed redevelopment of the HMD do not appear to have a significant effect on the available pressure or flow of the existing water distribution system within the vicinity of the HMD, under the conditions described in the report above. In addition, the Town has planned water main improvements in the vicinity of the HMD as part of the work in the Gas Affected Area. However, Woodard & Curran recommends the following should the proposed redevelopment occur in the HMD area:

- The water distribution infrastructure should be reanalyzed once demands are established and submitted to the Town by developers.
- The needed fire flow for each property within the HMD should be verified with the Town's Fire Department to determine that adequate AFF is provided at all locations.

In addition, the following recommendations for water main improvements within the vicinity of the HMD based on specific evaluation criteria are presented below.

Available Fire Flow

As discussed in Section 3.4.1, under MDD conditions AFF in the vicinity of the HMD was predicted to be below 2,500 gpm in several locations. In general, locations with predicated AFF below 2,500 gpm correlated with water mains which had low C-Factors or were small diameter mains. Table 15 summarizes the public streets which had cast iron water mains with a junction that had a predicted AFF of 2,500 gpm or less and summarizes potential factors limiting AFF in the vicinity of the location. Additional locations, beyond those listed in Table 15, were predicted to have AFF below 2,500 gpm. However, in general those locations were private roadways and/or were identified in the model as having lined ductile iron water mains.

Table 15: Areas of Low AFF under Scenario 2B

No.	Street Name	Water Main Diameter (inches)	Material	Approximate Length (feet)	Potential Limiting Factors	Potential Way to Increase AFF
1	Red Spring Road from Moraine Street to Essex Street	6	unlined cast iron	1,500	low roughness factor, small diameter	Upgrade main to 8" CLDI or greater
2	Lupine Road	6	unlined cast iron	3,720	low roughness factor, small diameter	Upgrade main to 8" CLDI or greater

No.	Street Name	Water Main Diameter (inches)	Material	Approximate Length (feet)	Potential Limiting Factors	Potential Way to Increase AFF
3	Central Street from Red Spring Road to Main Street	6/8[1]	unlined/lined cast iron	3,000/1,630	low roughness factor, small diameter	Abandon 8-inch water main and tie-over services to the existing 16-inch CLCI parallel main [2]
4	Henderson Avenue and Marion Avenue	6	lined cast iron	1,660	small diameter	Upgrade main to 8" CLDI or greater
5	Red Spring Road off Essex Street	8	unlined cast iron	250	low roughness factor	Replace main with new 8-inch or greater CLDI
6	Essex Street	12	cast iron	3,000	low roughness factor	Replace main with new 12-inch or greater CLDI.
7	Pearson Street	6	lined cast iron	950	low roughness factor, small diameter	Upgrade main to 8" CLDI or greater
8	Railroad Street	6	cast iron	1,700	low roughness factor, small diameter	Upgrade main to 8" CLDI or greater
9	Amici Way	6	unlined cast iron	150	low roughness factor, small diameter	Upgrade main to 8" CLDI or greater
10	Stevens Street	6	unlined cast iron	1,100	low roughness factor	Tie-over services to existing 12-inch CI water main[3]

Notes:

1. The water main is an 8-inch diameter main from Red Spring Road to Phipps Street and a 6-inch diameter main from Phipps Street to Main Street.
2. There is a 16-inch lined cast iron main located parallel to a smaller diameter (8-inch) water main in Central Street from Red Spring Road to Main Street. Unless two parallel mains are necessary at this location, the Town should consider abandonment of the smaller diameter main.
3. There is a 12-inch unlined cast iron water main in Stevens Street that runs parallel to the 6-inch water main from Shawsheen Road to approximately 27 Stevens Street.

In addition to being identified as having available fire flows below 2,500 gpm, portions of Stevens and Central Streets (in smaller diameter water mains) were also predicted to have high head loss as discussed in Section 3.4.1.

It is recommended that all the improvements listed above be implemented prior to redevelopment of the HMD. As part of a phased approach for water main improvements, the Town plans conduct the upgrades on Red Spring Road, Essex Street, Railroad Street, and Stevens Street. It is recommended that water main improvements on Lupine Road, Henderson Avenue, Marion Avenue, and Central Street including final paving be conducted by the developer if requested by the Town.

3.4.2.1 Water Main Replacement Cost Estimate

Based on the recommendations for water main improvements above, approximately 12,330 linear feet (LF) of 6-inch and 8-inch water main and approximately 3,560 LF of 12-inch water main within the HMD are recommended for upgrade and/or replacement based on limiting factors such as low roughness factor and pipe size. Additional improvements include abandonment of smaller water mains and connecting water services to larger mains. Woodard & Curran prepared a preliminary cost estimate for the water main improvements, as summarized in the table below. It was assumed that all 6-inch water mains would be upgraded with 8-inch CLDI mains and all 8-inch and 12-inch water mains would be replaced with CLDI main of the same size. A budgetary cost of approximately $275/LF was assumed to upgrade 6-inch water main to new 8-inch water main and replace 8-inch water main. A budgetary cost of approximately $325/LF was assumed for 12-inch water main replacement. If all the water main improvements are conducted, a total design and construction cost of approximately $7,836,600.00 is estimated. The estimated cost assumes full-width mill and pave. An increase in cost is anticipated if the Town wanted to conduct reclamation instead of full width mill and pave.

Table 16: Preliminary Cost Estimate for Water Main Improvements

Parameter	Preliminary Cost
Construction Cost	$5,295,000.00
Design (10%)	$529,500.00
Police Detail Allowance (13%)	$688,350.00
Construction Administration (15%)	$794,250.00
Contingency (10%)	$529,500.00
Total	$7,836,600.00

4. REFERENCES

1. American Water Works Association (AWWA), *M32 Computer Modeling of Water Distribution Systems, Third Edition*, 2012.

2. Gamble Associates, *Historic Mill District Andover, Design Guidelines, Andover, MA*, February 6, 2018.

3. Massachusetts Department of Environmental Protection (MassDEP), *310 CMR 15.00 The State Environmental Code, Title 5 Standard Requirements for the Siting, Construction, Inspection, Upgrade and Expansion of On-Site Sewage Treatment and Disposal Systems and for the Transport and Disposal of Septage*, September 9, 2016

4. MassDEP Bureau of Resource Protection Drinking Water Program, *Guidelines for Public Water Systems Chapter 9: Distribution System Piping & Appurtenances*, August 27, 2017.

5. RKG Associates, Inc., *Market Analysis for Reuse Planning of the Andover Town Yard a 40R Overlay District, Andover, Massachusetts,* February 10, 2010

APPENDIX A: HISTORIC MILL DISTRICT DEVELOPMENT OPPORTUNITIES

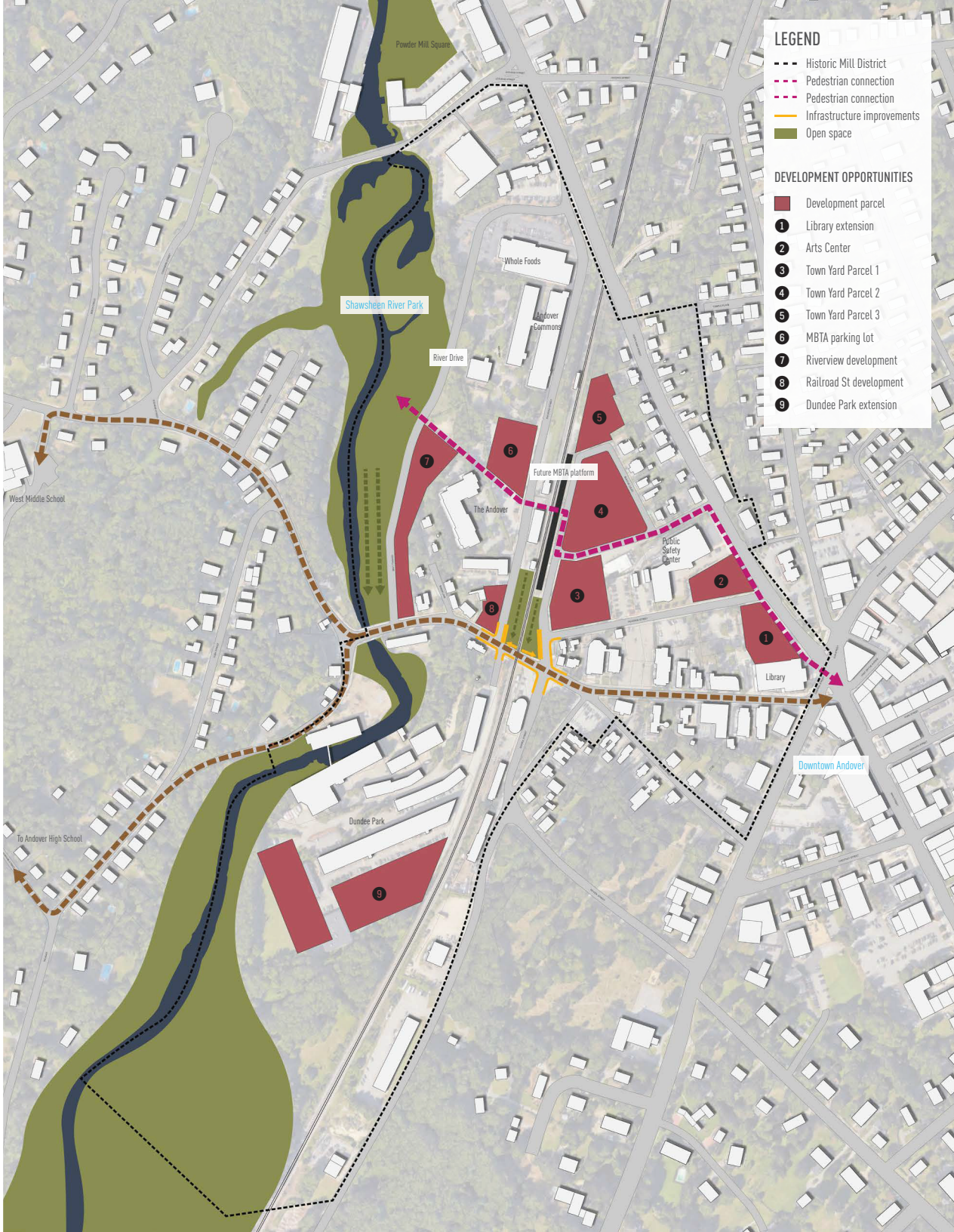
Bringing the downtown together

Conceptual approach

- Historic Mill District
- Pedestrian connection
- Pedestrian connection
- Infrastructure improvements
- Open space

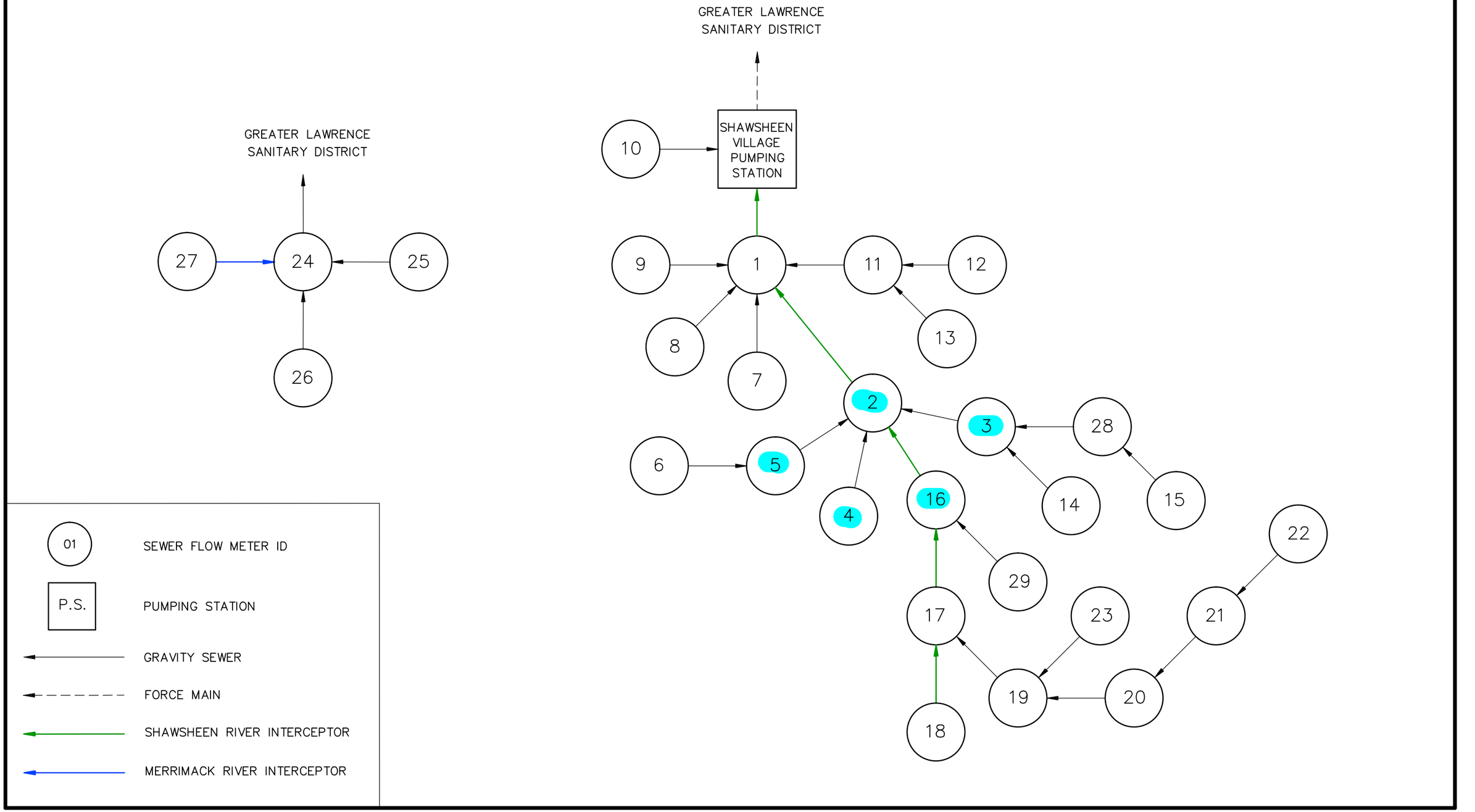
DEVELOPMENT OPPORTUNITIES

- Potential development parcel
- 1 Library extension
- 2 Arts Center
- 3 Town Yard Parcel 1
- 4 Town Yard Parcel 2
- 5 Town Yard Parcel 3
- 6 MBTA parking lot
- 7 Riverview development
- 8 Railroad St development
- 9 Dundee Park expansion.



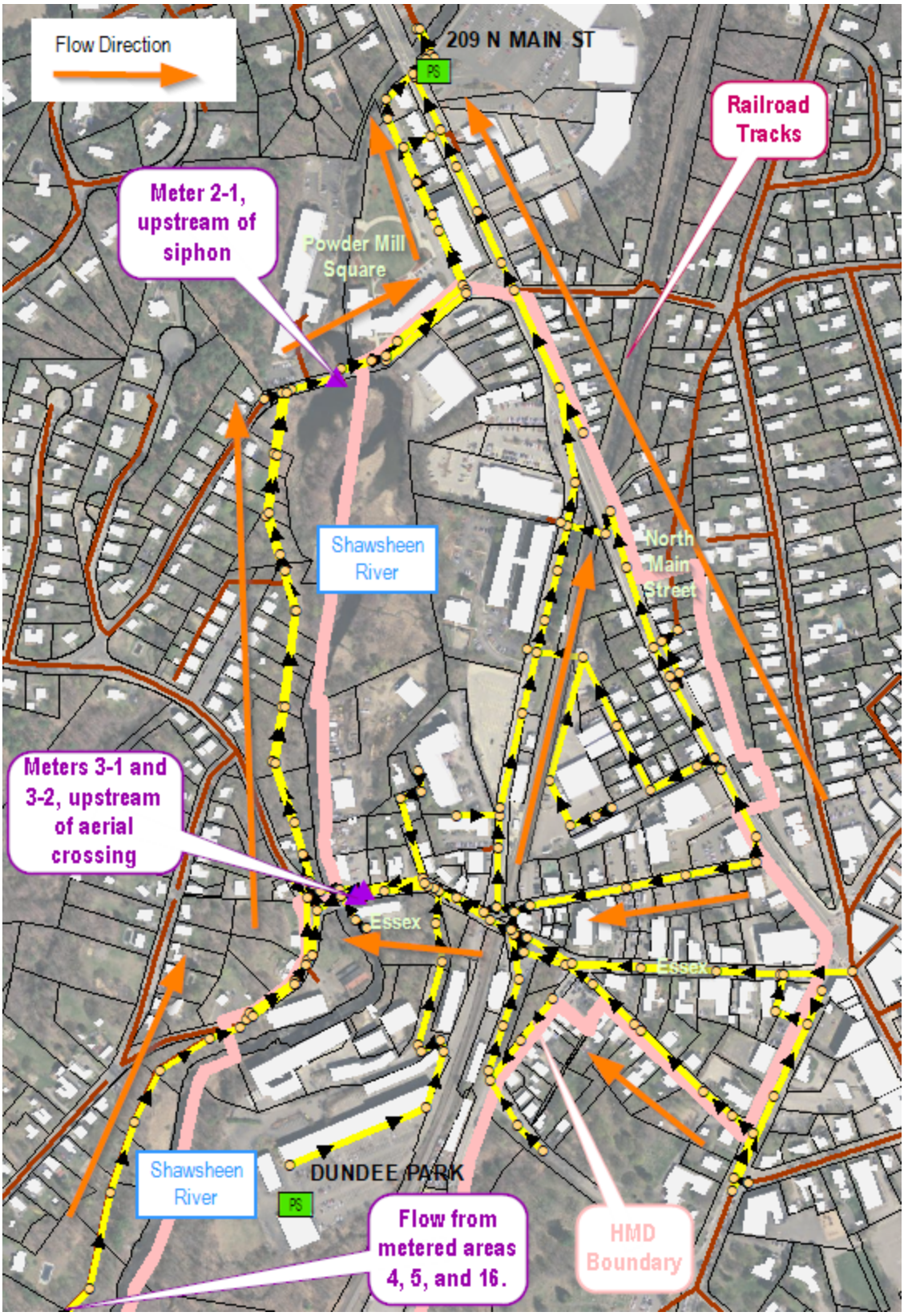
APPENDIX B: WASTEWATER SYSTEM OVERVIEW

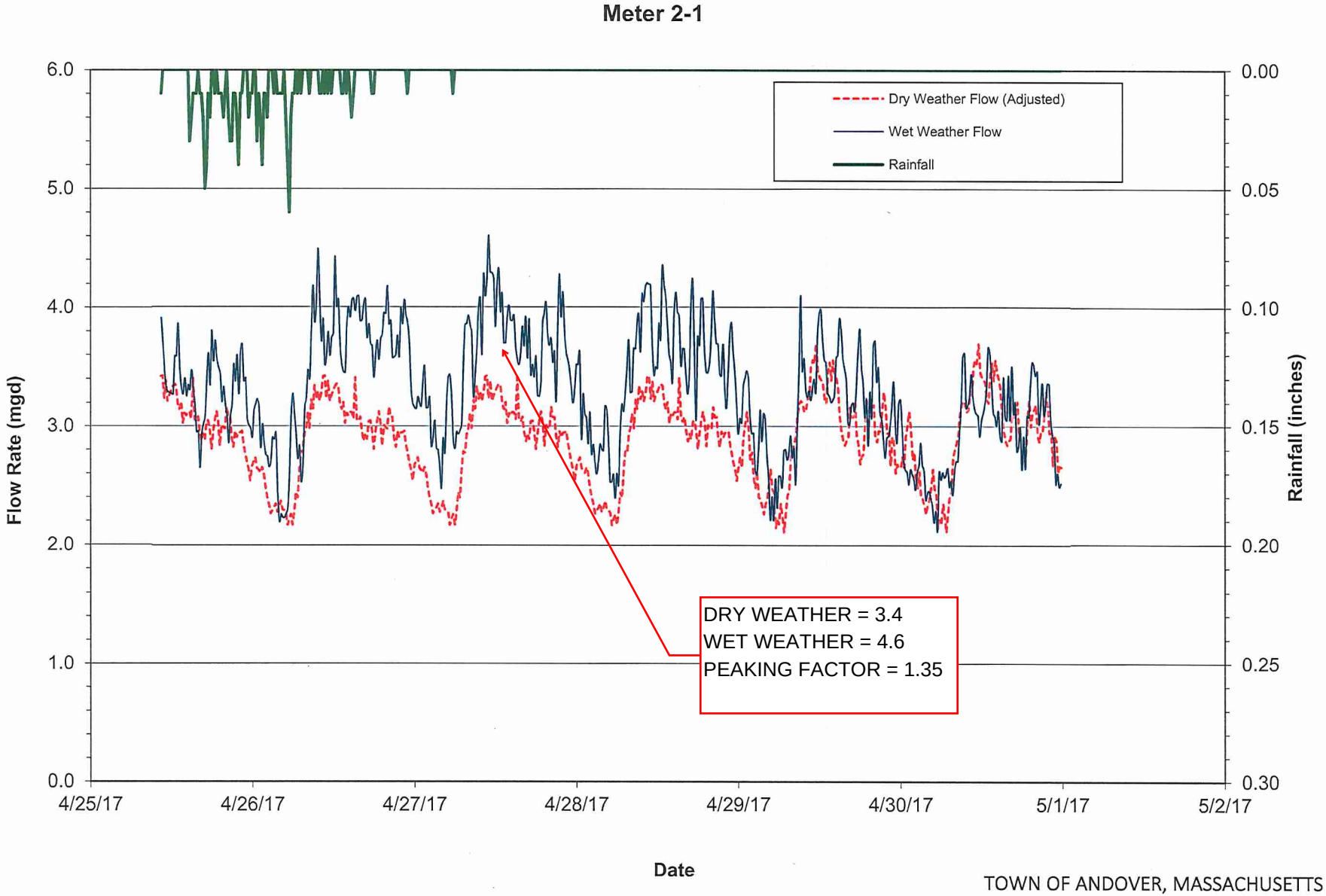
- Wastewater System Overview Figure
- Excerpts from *Infiltration/Inflow Flow Monitoring Summary Report (CDM, 2017)*



CDM Smith

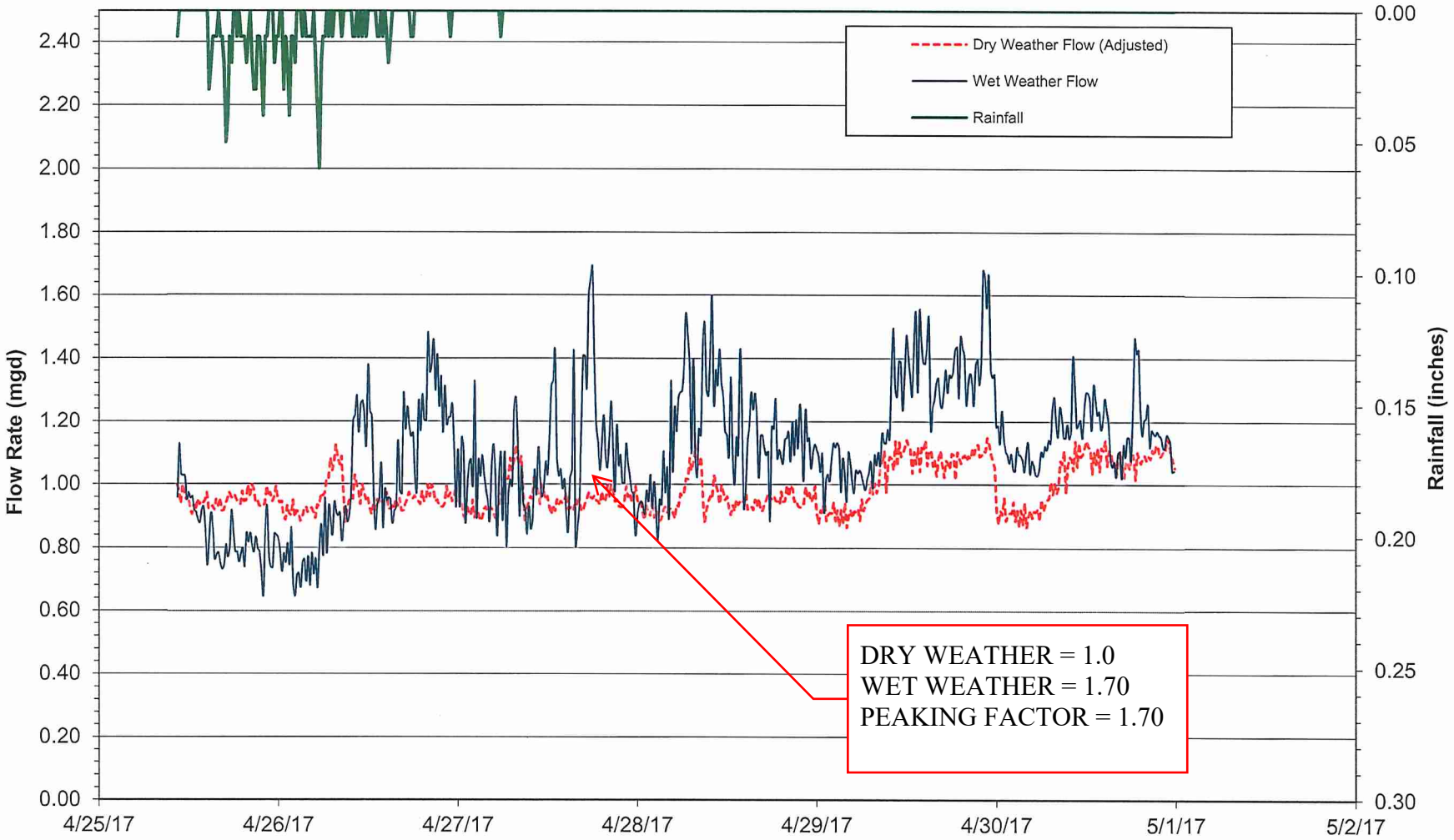
Figure 3-2
TOWN OF ANDOVER, MASSACHUSETTS
SPRING 2017 FLOW MONITORING REPORT
METERING SUB-AREA SCHEMATIC


Flow Direction
209 N MAIN ST
PS
Railroad Tracks
Meter 2-1, upstream of siphon
Powder Mill Square
Shawsheen River
North Main Street
Meters 3-1 and 3-2, upstream of aerial crossing
Essex
Essex
Shawsheen River
DUNDEE PARK
PS
Flow from metered areas 4, 5, and 16.
HMD Boundary



TOWN OF ANDOVER, MASSACHUSETTS
FLOW MONITORING SUMMARY REPORT
FLOW VS. TIME GRAPH - APRIL 25-26, 2017 STORM
METER 2-1

Meter 3-1



Date

TOWN OF ANDOVER, MASSACHUSETTS
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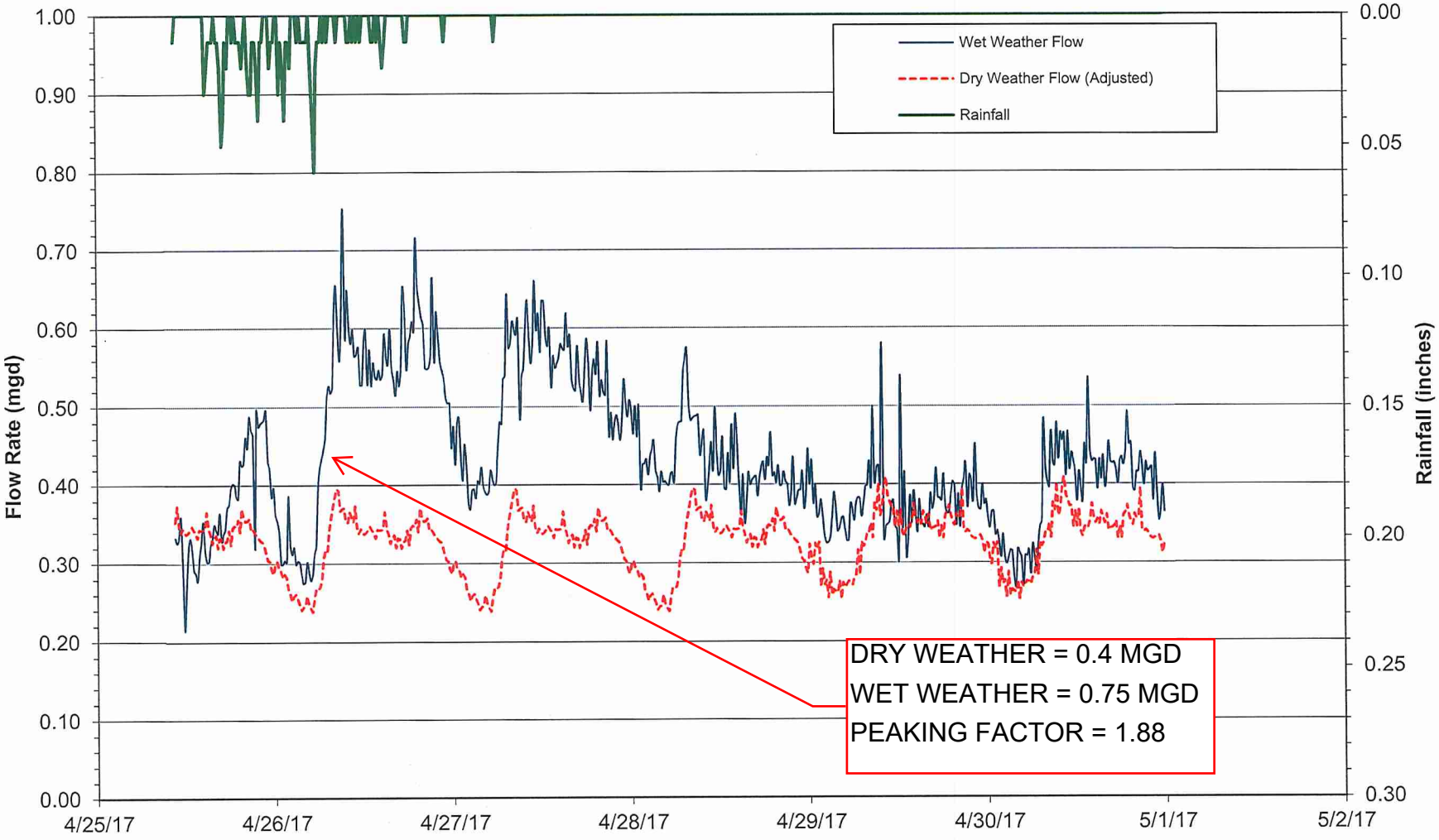
FLOW VS. TIME GRAPH - APRIL 25-26, 2017 STORM
METER 3-1

CDM Smith

Meter 3-2



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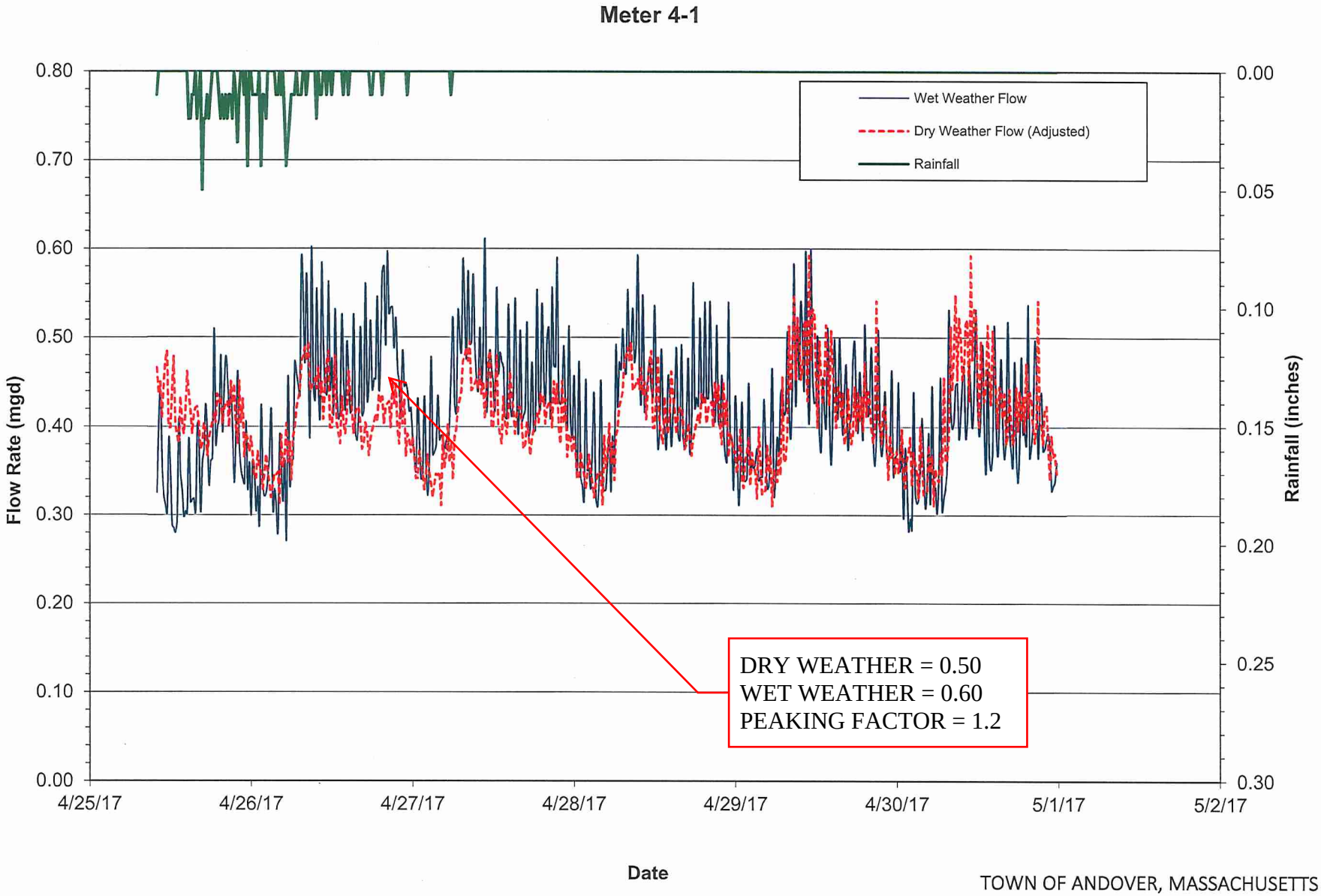
FLOW MONITORING SUMMARY REPORT
FLOW VS. TIME GRAPH - APRIL 25-26, 2017 STORM
METER 3-2

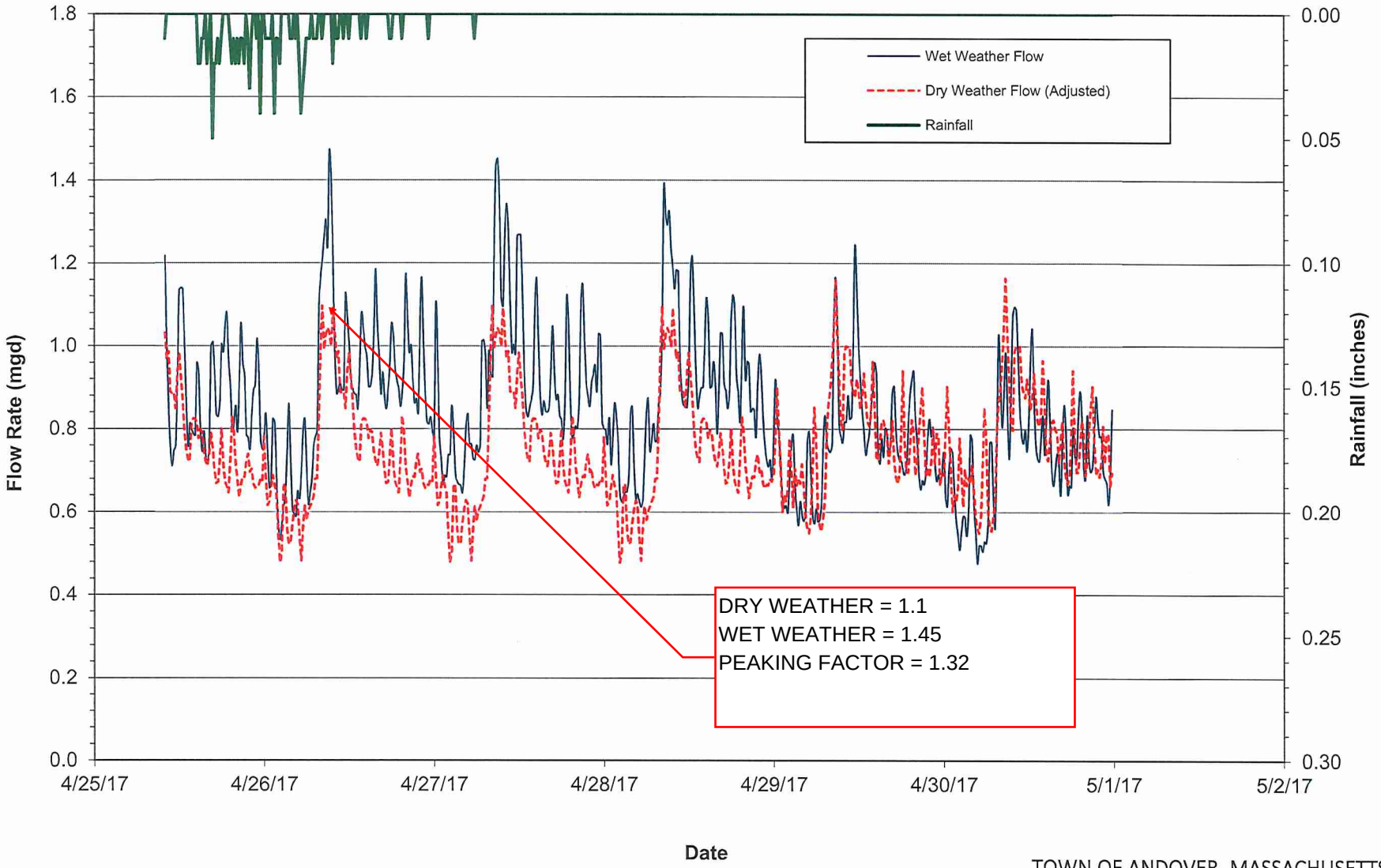
CDM Smith



TOWN OF ANDOVER, MASSACHUSETTS
FLOW MONITORING SUMMARY REPORT
FLOW VS. TIME GRAPH - APRIL 25-26, 2017 STORM
METER 4-1

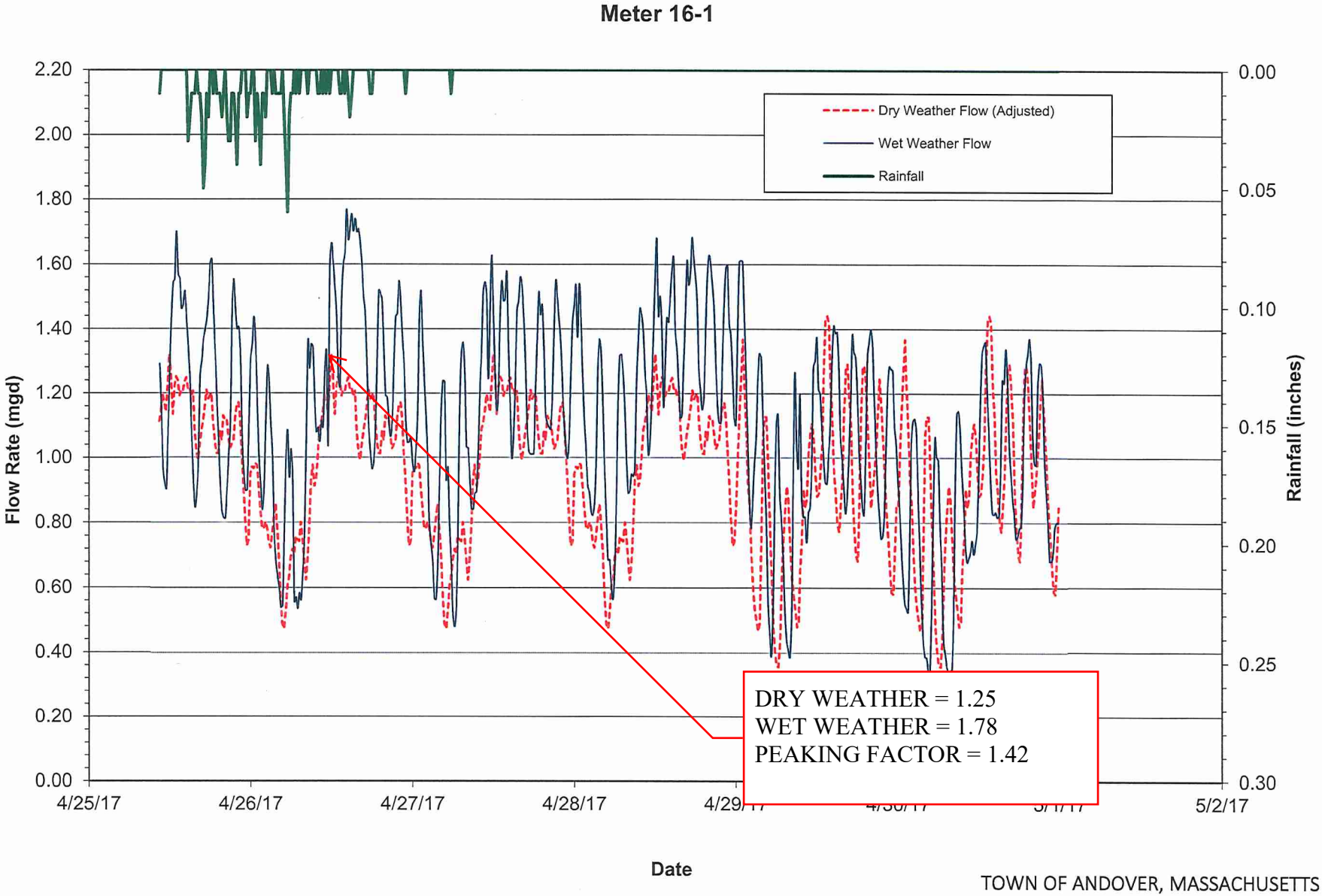
CDM Smith

Meter 5-1



CDM Smith

TOWN OF ANDOVER, MASSACHUSETTS
FLOW MONITORING SUMMARY REPORT
FLOW VS. TIME GRAPH - APRIL 25-26, 2017 STORM
METER 5-1



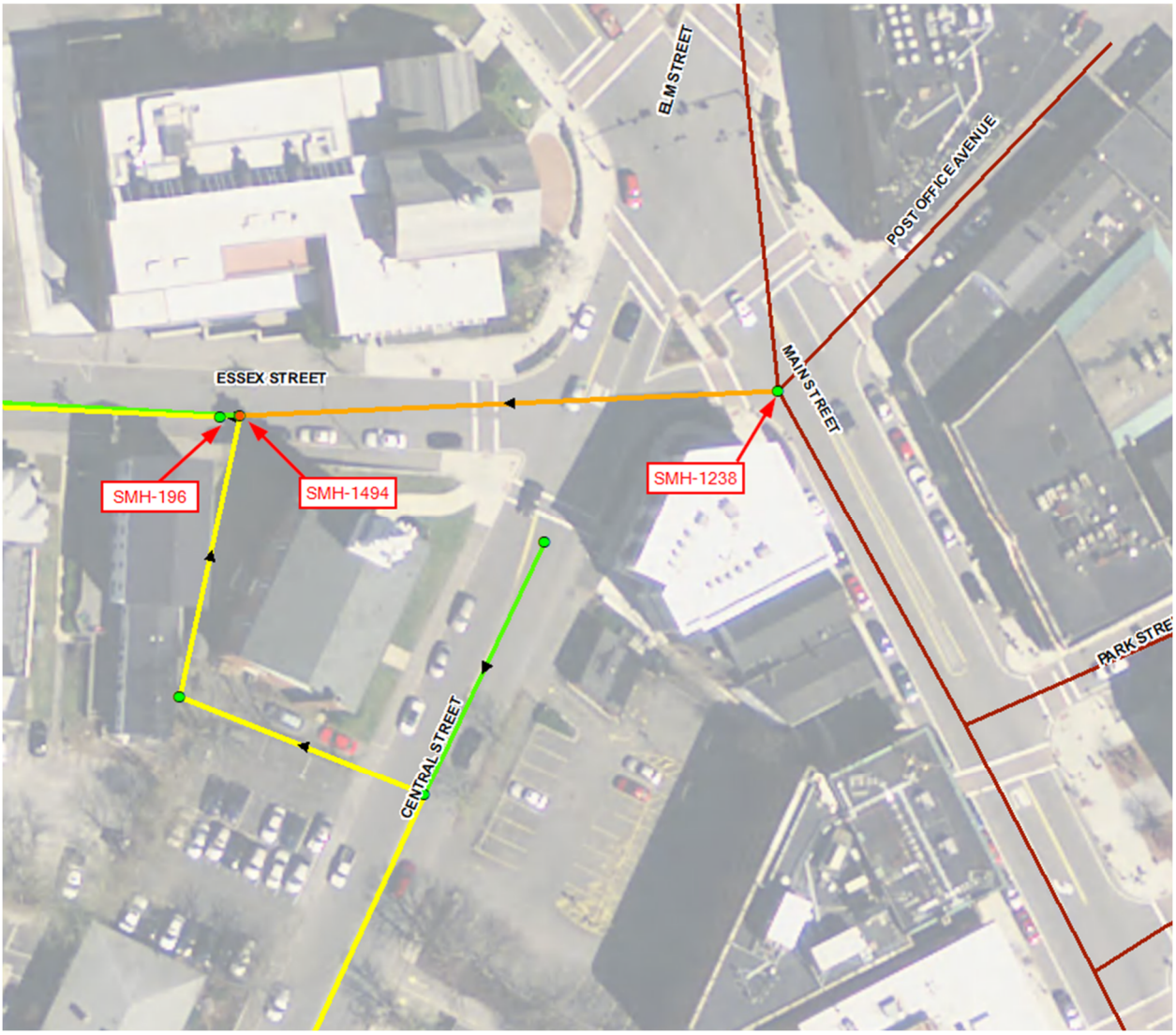
TOWN OF ANDOVER, MASSACHUSETTS
FLOW MONITORING SUMMARY REPORT
FLOW VS. TIME GRAPH - APRIL 25-26, 2017 STORM
METER 16-1

APPENDIX C: FIELD INVESTIGATION DATA

Area 1: SMH-1494

At Sanitary Manhole (SMH) 1494, there exists an original sewer and a relief sewer. The invert of this manhole is unknown as well as the difference in elevation of the outgoing pipes. The upstream and downstream manholes are requested to compare to the record drawings and orient the gathered data elevations to the rest of the system. Flow direction is marked via black arrows pointing in the downstream direction. Data can be entered on the next page.

Orientation Map



Area 2: SMH-210

At SMH-210, the elevation of the invert is critical and unknown. It should be lower than SMH-272, but higher than known downstream elevations, such as SMH-81 (Area 3). The flow split at SMH-210 also influences the percent of flow that is conveyed in the old smaller sewer (orange) or the relief sewer (red).

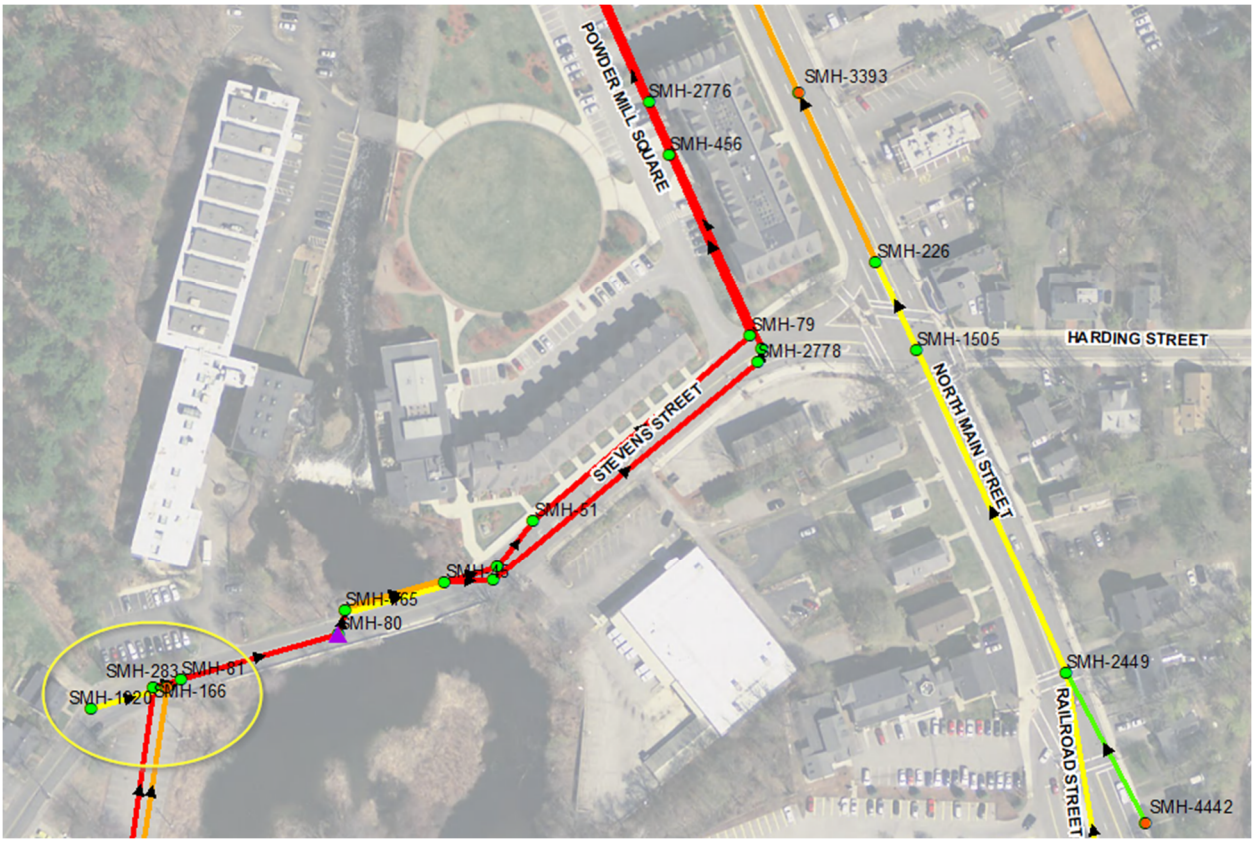
Orientation Map



Area 3: SMH-283

At SMH-283, the elevation of the invert is unknown. It should be higher than SMH-81, but lower than upstream elevations, such as SMH-210 (Area 2). SMH-81 is confirmed within a foot of the record drawings. Confirm direction of flow and depth of MH.

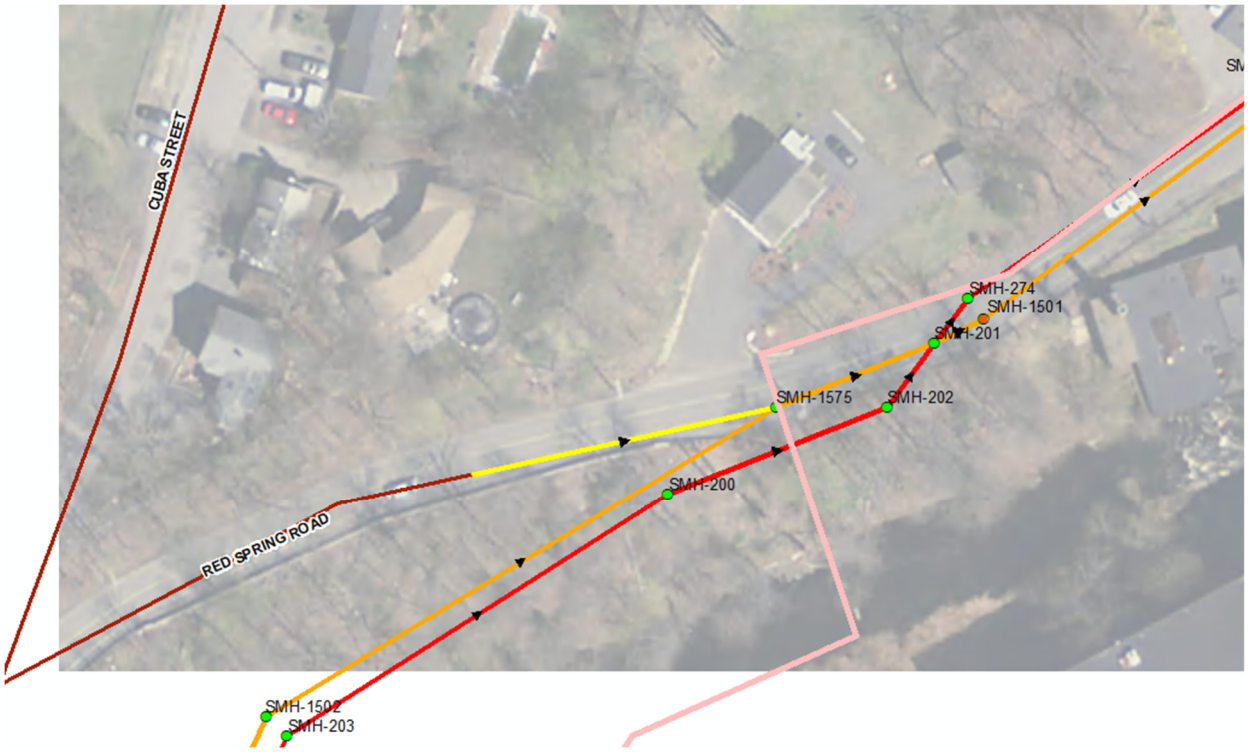
Orientation Map



Area 4: SMH-201

Record drawings confirm that these two sewers join and cross at SMH-201, but it is not clear if one sewer is higher than the other. The flow split at SMH-201 will determine which sewer line handles the base flow and which one handles increased flow due to infiltration.

Orientation Map



Area 5: PS-46

What is the downstream condition of the model? (This may not be answerable from the field, but record any observations.)

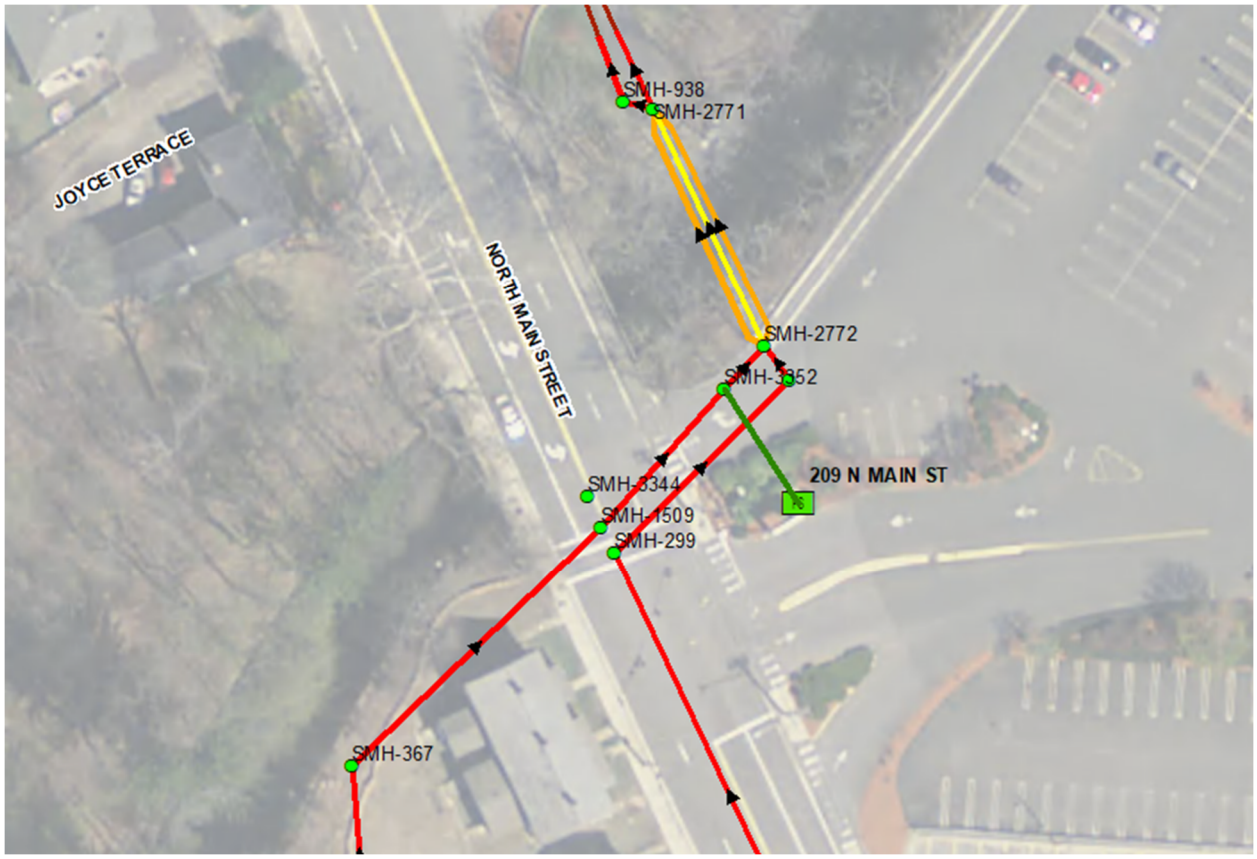
Does all of the flow go to a wet well at the PS?

If not, what flow comes into the pump station?

Found Record Drawings. I think pump station is designed to lift flow from nearby shopping area to sanitary sewer. \\wc\shared\Projects\0231642.00 Andover MA Historic Mill District Master Plan\wip\Info From Town\Referenced Record Drawings

Is the pump station's primary purpose to push flow through the siphon?

Orientation Map



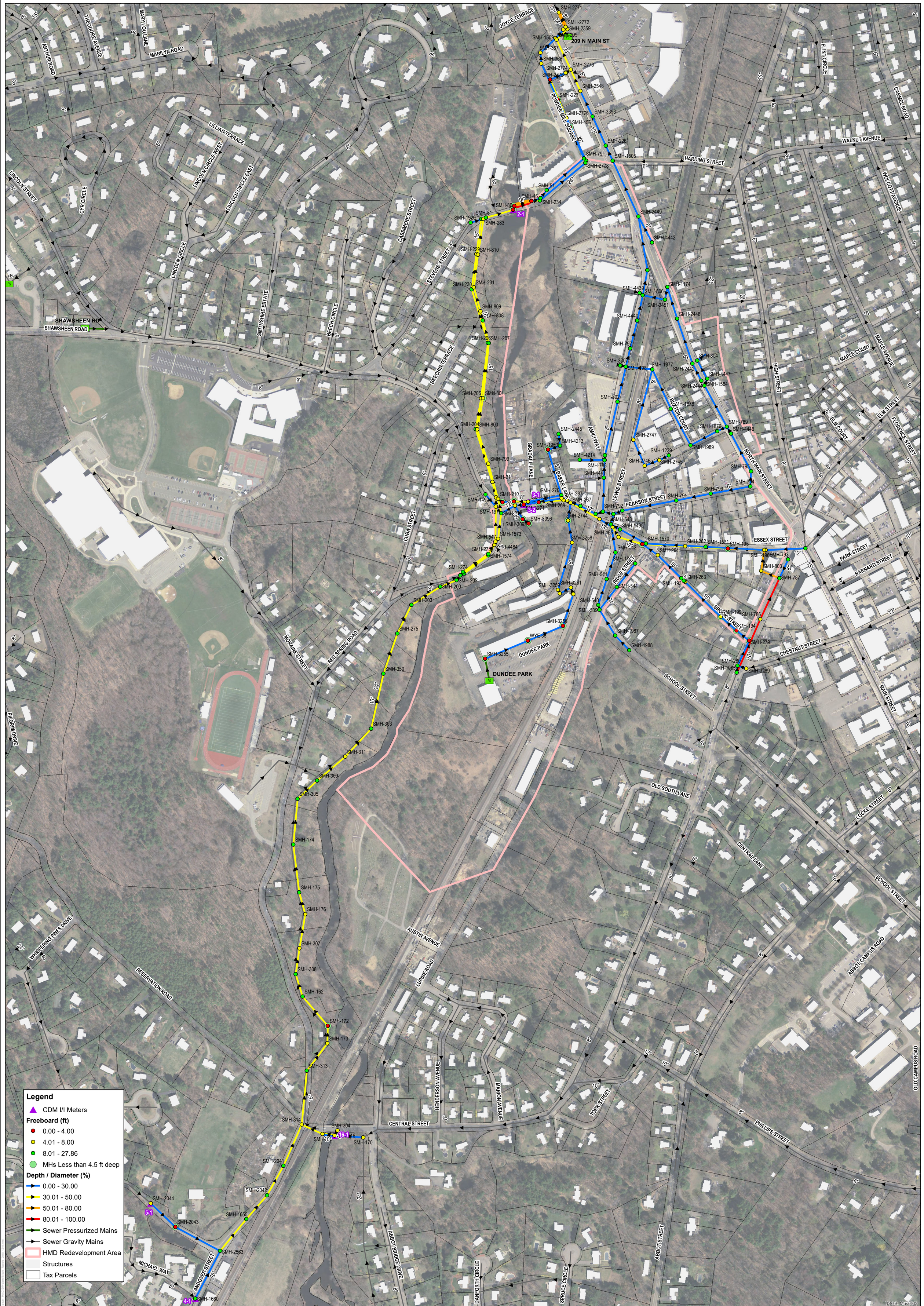
APPENDIX D: MODEL RESULTS MAPS

Historic Mill District

Town of Andover MA
Master Plan Study

Appendix D

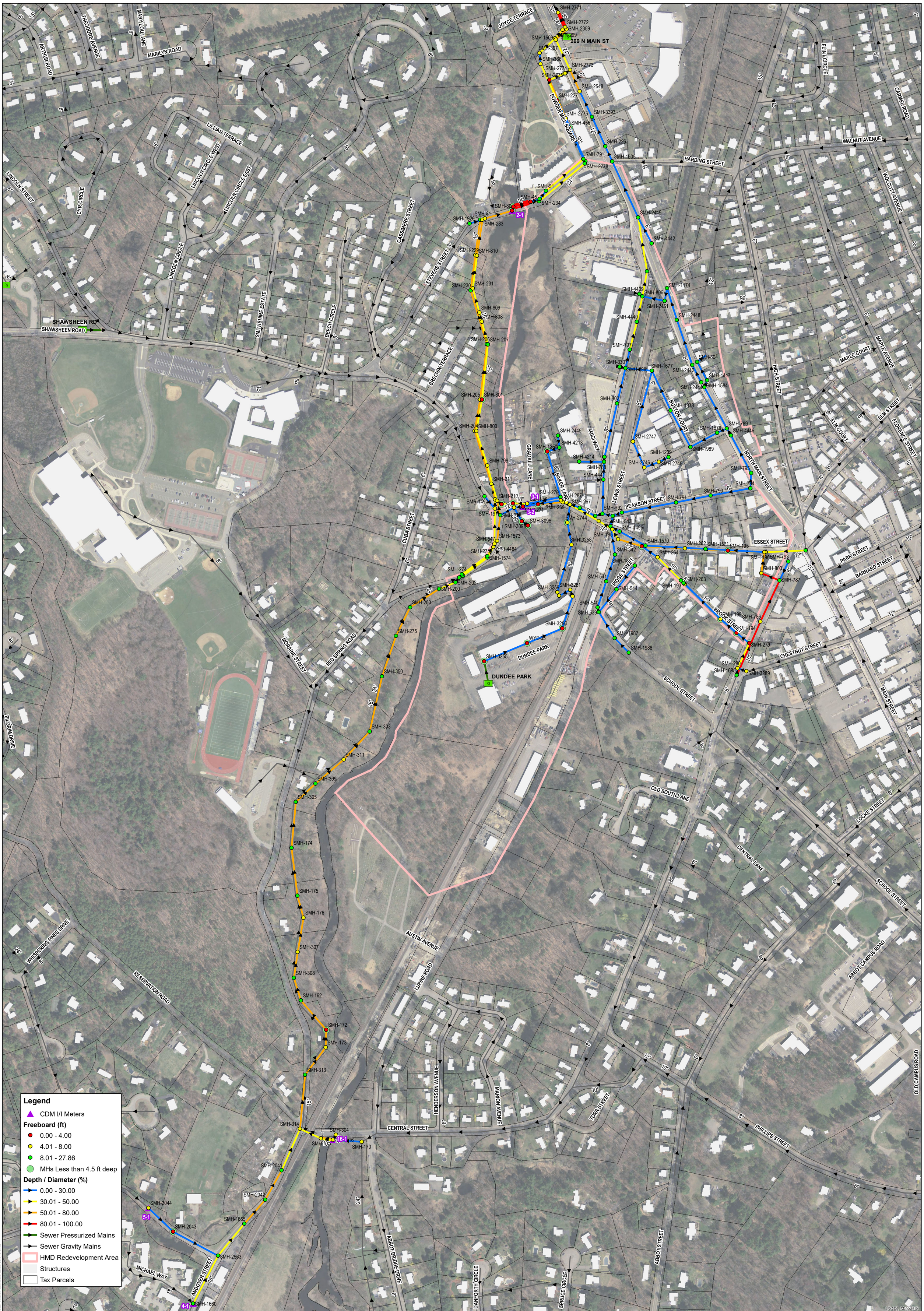
Scenario 1: Existing Flow Conditions in Dry Weather



WOODARD &CURRAN

Project #: 0231642.00
Map Created: December 2018

Third Party GIS Disclaimer: This map is for reference and graphical purposes only and should not be relied upon by third parties for any legal decisions.
Any reliance upon the map or data contained herein shall be at the users' sole risk. **Data Sources:**



Historic Mill District

Town of Andover MA
Master Plan Study

Appendix D

Scenario 2: Existing Flow Conditions in Wet Weather

N

0 0.02 0.04 0.08 Miles

WOODARD &CURRAN

Project #: 0231642.00
Map Created: December 2018

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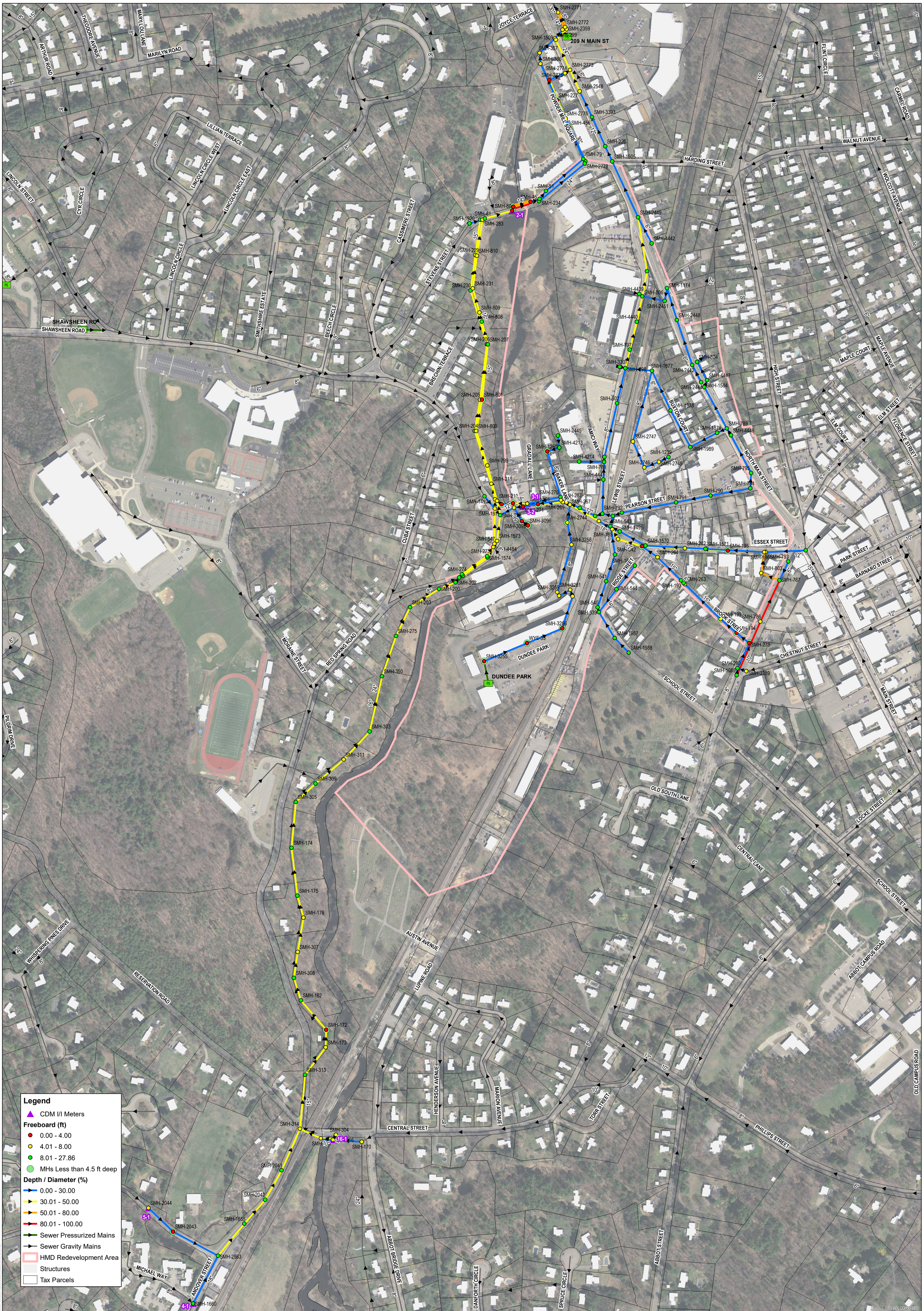
209 N MAIN ST
SHAWSHEEN RD
SHAWSHEEN ROAD
DUNDEE PARK
ARTHUR ROAD
THEODORE AVENUE
MARY LOU LANE
MARILYN ROAD
LILLIAN TERRACE
LINCOLN CIRCLE WEST
LINCOLN CIRCLE EAST
LINCOLN CIRCLE
LINCOLN STREET
CYR CIRCLE
SMITHSHIRE ESTATE
BEECH CIRCLE
BRECHIN TERRACE
CASSIMERE STREET
STEVENS STREET
JOYCE TERRACE
POWDER MILL SQUARE
FLINT CIRCLE
CARMEL ROAD
WALNUT AVENUE
HARDING STREET
WOLCOTT AVENUE
CUBA STREET
GRADALL LANE
BAKER LANE
AMICI WAY
LEWIS STREET
BUXTON COURT
NORTH MAIN STREET
PEARSON STREET
ESSEX STREET
RIDGE STREET
BROOK STREET
HIGH STREET
MAPLE COURT
MAPLE AVENUE
ELM COURT
ELM STREET
FLORENCE STREET
PARK STREET
BARNARD STREET
CHESTNUT STREET
SCHOOL STREET
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MORAINE STREET
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RESERVATION ROAD
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LOCKE STREET
AUSTIN AVENUE
LUPINE ROAD
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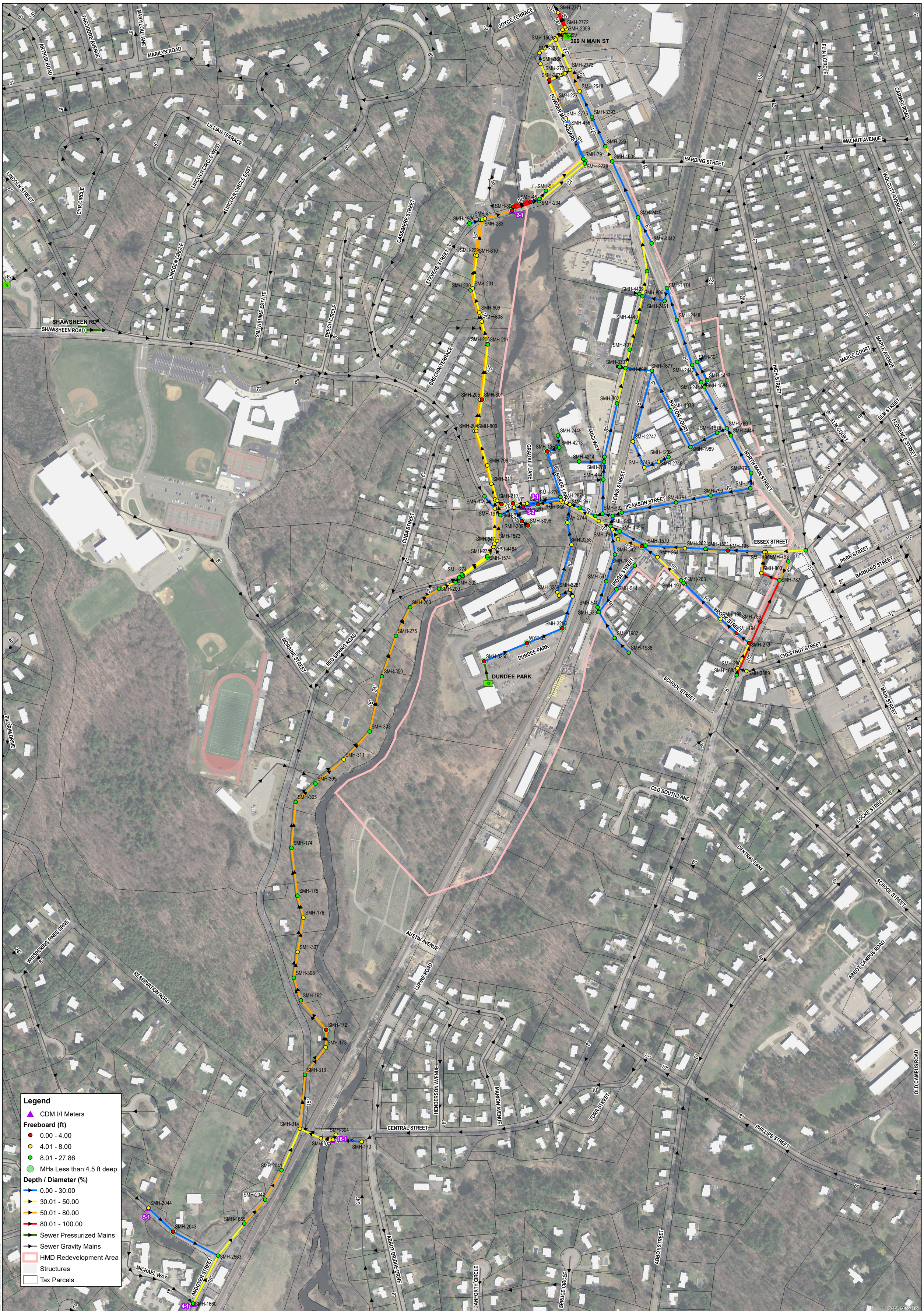
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ARTHUR ROAD
THEODORE AVENUE
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MARILYN ROAD
LILLIAN TERRACE
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Historic Mill District

Town of Andover MA
Master Plan Study

Appendix D

Locations of 6-inch Sewers

Legend

- 6-Inch Gravity Sewer Main
- Sewer Gravity Mains
- Sewer Pressurized Mains
- HMD Redevelopment Area
- Structures
- Tax Parcels

N

0 0.02 0.04 0.08 Miles

WOODARD &CURRAN

Project #: 0231642.00
Map Created: December 2018

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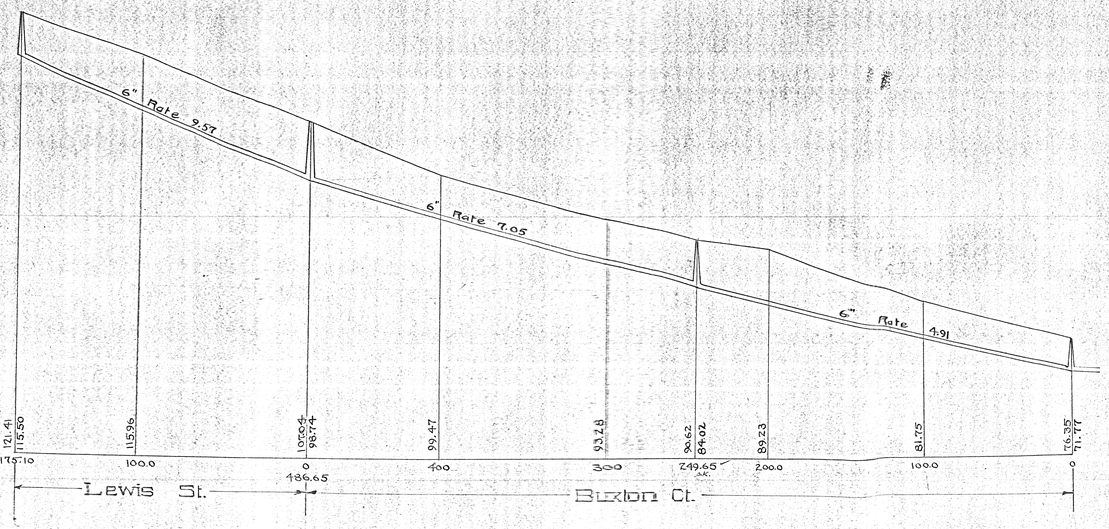
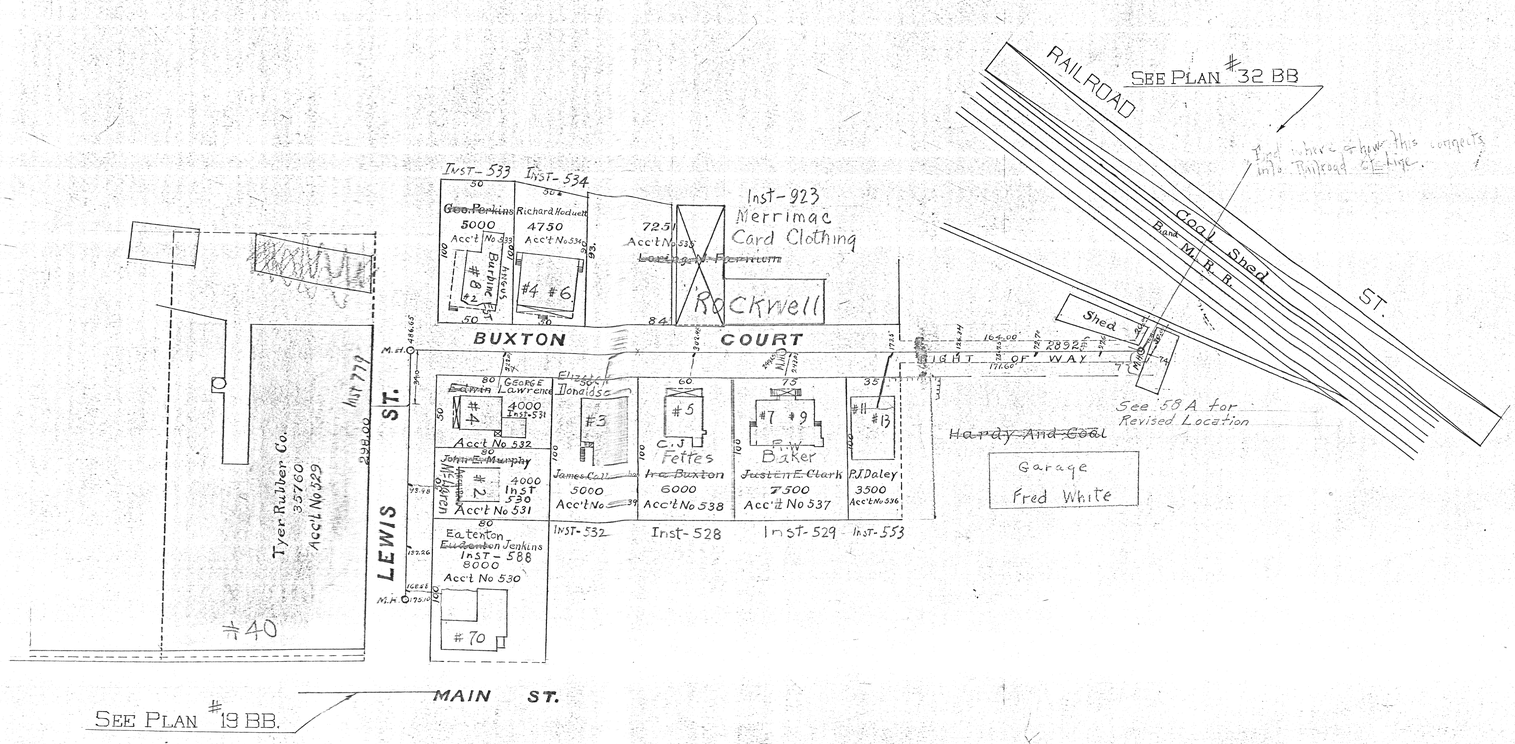
Any reliance upon the map or data contained herein shall be at the users' sole risk. **Data Sources:**

USGS, MassGIS





LEWIS ST. & BUXTON CT.

SCALE 50 FT. HOR. 10 FT. VER = 1 IN.

– 1909 –

BOARD OF PUBLIC WORKS, ANDOVER MASS.

JOHN E. SMITH, ENGINEER

ANDOVER STREET

HALL AVENUE

BRADLEY ROAD

ESSEX STREET

BROOK STREET

160 l.f. of 18" RCP
2-DMH

TEST PITS

Connect New 18" RCP to existing 18" RCP Stub & remove existing brick and mortar plug in manhole.

300 l.f. of 15" RCP
2 DMH

10 l.f. of 12" RCP
1 DMH
Break out existing 12" RCP, connect inlet and plug outlet

Connect New 15" RCP to existing CB and plug existing 12" RCP inlet.

Plug Existing 18" RCP Outlet

20' UTILITY EASEMENT

SCALE 1"=40'

132 l.f. of 15" RCP
6 l.f. of 12" RCP
1-DMH
2-CB

231 l.f. of 12" RCP
1-DMH
Connect 12" RCP to exist. DMH

16 l.f. of 12" RCP
2-CB
Connect 12" RCP to exist. DMH

Connect 15" RCP to existing CB

St. Augustine Church

Rectory

Backstreet Restaurant

Drainage Easement

Picket Fence

Brick Walk

Stone Wall

Bit. Conc. Parking Area

Gravel Drwy.

Bit. Conc. Drwy.

Conc. Walk

Granite Curb

Sidewalk

Tel. Duct

Post & Rail Fence

Stone Drwy.

Bit. Conc. Drwy.

Existing 15" RCP

EXIST. 12" RCP
INV.= 148.34
12" INV.= 148.00

12" RCP
S=0.044

15" RCP
S=0.037

15" INV.= 143.23

15" INV.= 146.69

15" RCP S=0.091

15" INV.= 125.74

15" INV.= 125.94

18" RCP S=0.010

18" INV.= 119.90

18" INV.= 120.10

18" RCP
S=0.021

EXIST. 5' STILE
18" RCP INV.= 114.47

18" INV.= 114.57

EXIST. 18" RCP INV.= 112.0

EXIST. DMH

DMH #4

DMH #3

DMH #2

DMH #1

CB #18

DMH #36

12" RCP S=0.054

12" INV.= 124.40

12" INV.= 124.60

12" INV.= 125.25

EXIST. 12" INV.= 125.35

15" RCP
S=0.018

15" RCP S=0.019

Telephone Duct (12)
Test Pit

15" INV.= 112.95

15" INV.= 114.11

6" INV.= 114.31

12" INV.= 114.37

INV. 12" RCP = 112.85

15" INV.= 112.65

SCALES H: 1"=40'
V: 1"=4'

GENERAL NOTES:
1) Manhole and Catchbasin frame & cover shall be Neenah type EA160 and EP210, respectively (or equal).

CONTRACT 1956/6-EC/52

Drainage Construction
HALL AVE. to BRADLEY RD. & ESSEX ST.
Town of Andover, Mass.

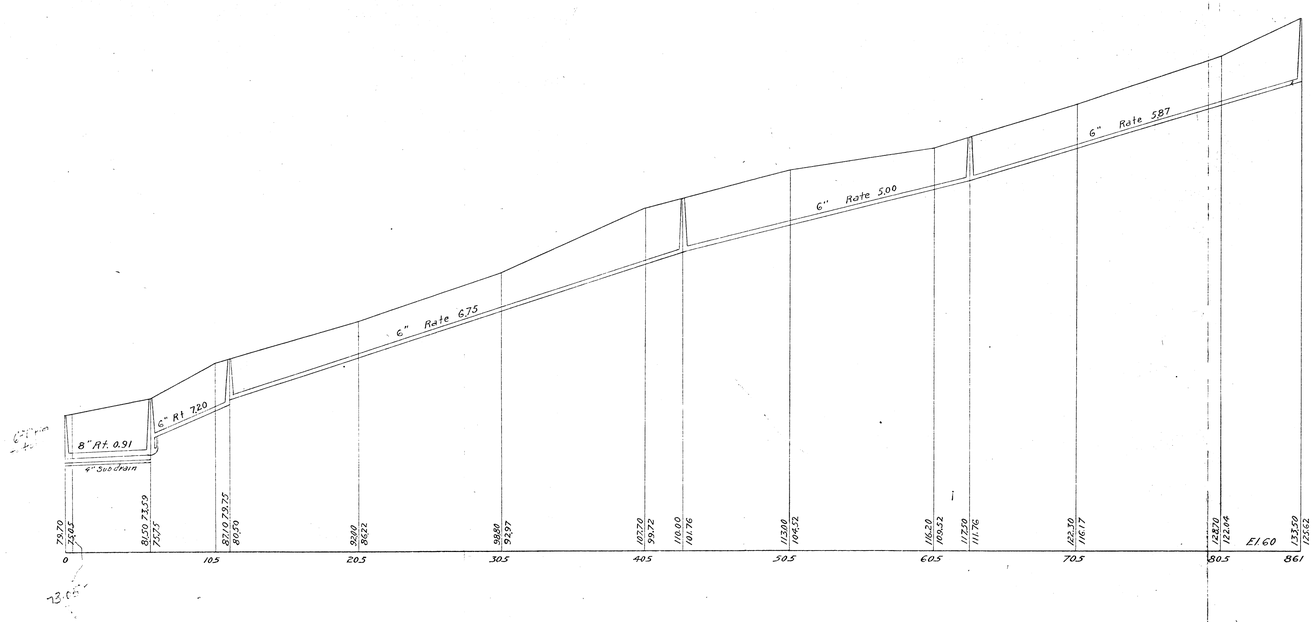
Scale as shown
John Avery, Jr.

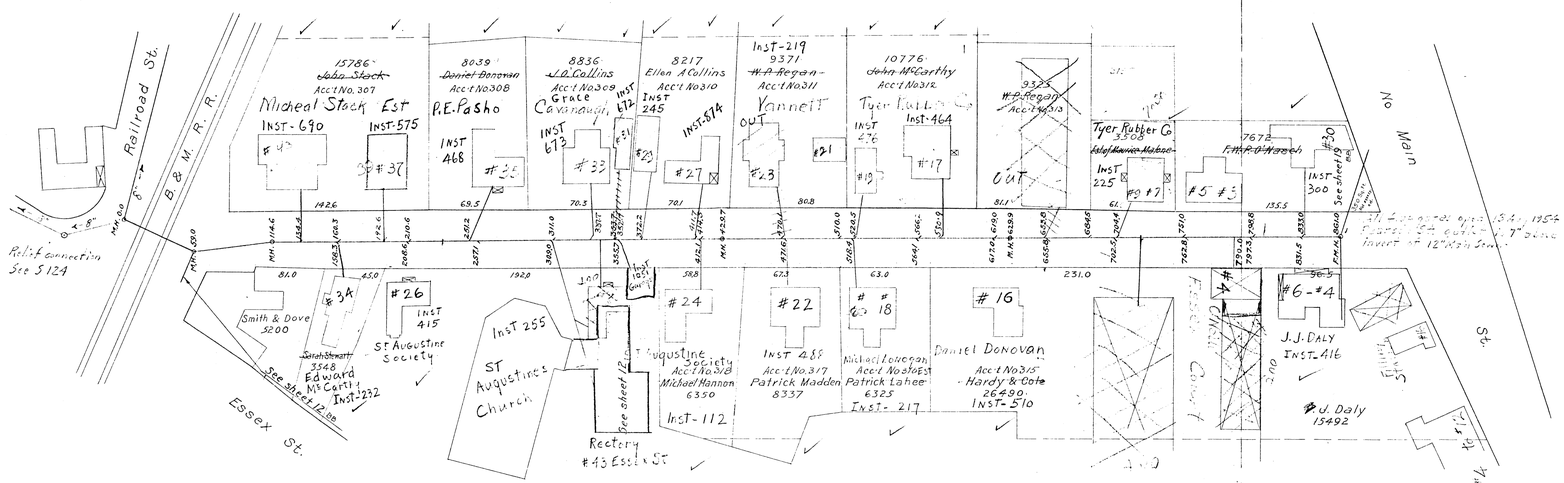
July, 1956
Town Engineer
Sheet 1 of 1

JOHN AVERY, JR. 13149 REGISTERED PROFESSIONAL ENGINEER

Drawn & Designed BKM

DR-121




15786 John Stack Acct No. 307
Micheal Stack Est
INST-690
INST-575
8039 Daniel Donovan Acct No 308
P.E. Pasho
INST 468
8836 J.D. Collins Acct No 309
Grace Cavanaugh
INST 673
INST 672
8217 Ellen A Collins Acct No 310
INST 245
INST-674
INST-219 9371 W.A. Regan - Acct No 311
Yannett
OUT
10776 John McCarthy Acct No 312
Tyler Rubber Co
INST 276
INST-464
9373 W.P. Regan Acct No 313
Tyler Rubber Co 3508
Inst of Marine Motors
INST 225
7672 F.M. Pearson
INST 300
See sheet 19
Main St.
Railroad St.
B & M. R. R.
Relief connection See 5124
Smith & Dove 5200
Sarah Straith 3548 Edward McCarty Inst-1232
See sheet 12
Essex St.
St Augustine Society
INST 415
InST 255
ST Augustines Church
See sheet 12
Rectory #43 Essex St
St Augustine Society Acct No 318 Michael Hannon 6350
INST-112
INST 488 Acct No 317 Patrick Madden 8337
Michael Lonergan Acct No 316 Patrick Lahee 6325
INST-217
Daniel DONOVAN Acct No 315 - Hardy & Cole 26490 INST-510
J.J. DALY INST-416
J. Daly 15492
Pearson Court
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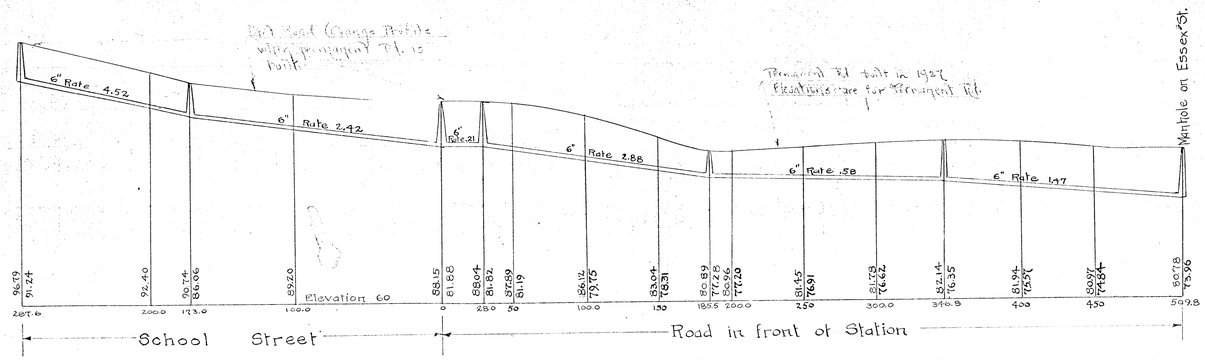
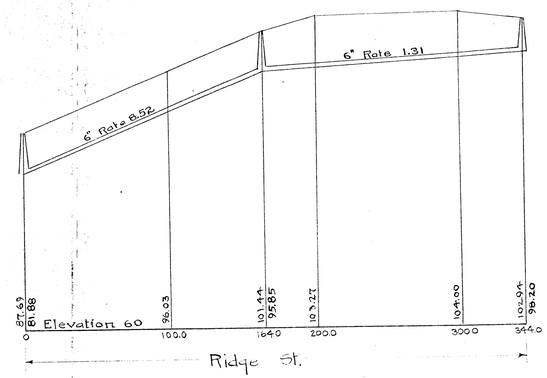
N.W. 51.46.6
N.W. 42.57
N.W. 47.10.3
N.W. 48.53.0
N.W. 65.58.9
N.W. 65.58.9
N.W. 65.58.9

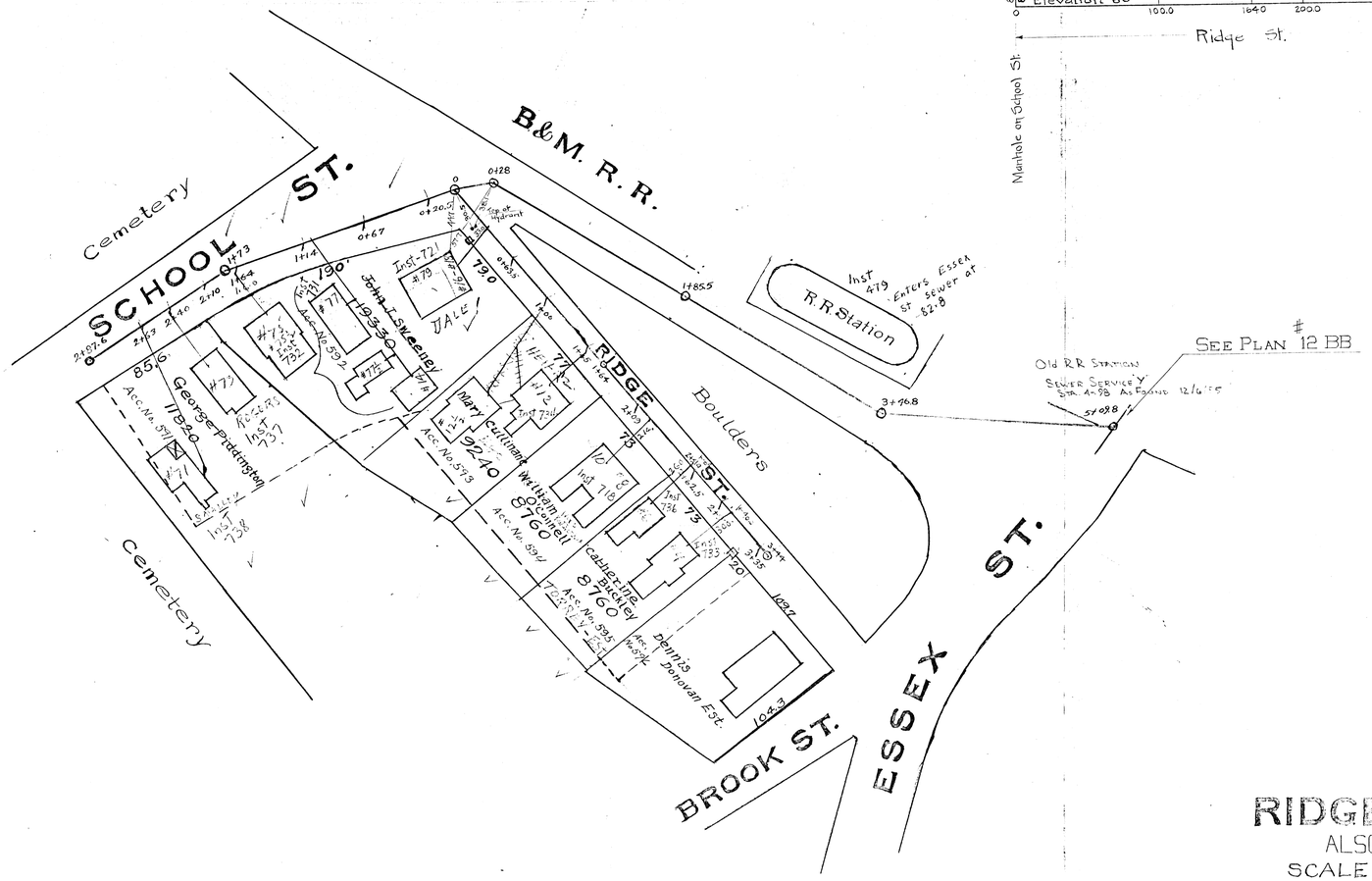
Dirt Road (Grade Profile when Permanent St. is built)

Permanent St. built in 1927
Existing grade for Permanent St.



287.6
173
114.6




B&M. R.R.
Cemetery
SCHOOL ST.
RIDGE ST.
ESSEX ST.
BROOK ST.
Boulders
Cemetery
R.R. Station
Old R.R. Station
SEE PLAN # 12 BB
Inst 479 Enters Essex St sewer at 82.8
Sewer Service Y Sta. 4+98 As Found 12/6/15
Old Road (Grade Profile when Permanent St. is built)
Permanent St. built in 1927 Existing grade for Permanent St.
Manhole on Essex St.
Manhole on School St.
Elevation 60
Ridge St.
Road in front of Station
School Street
Inst-72
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#1
John L. Sweeney 19330
George Puddingstown 17920
Acc. No. 578
Rogers Inst 757
Inst 758
Mary Cuttmann 9240
Acc. No. 579
Wm. Cuttmann 8760
Acc. No. 594
Catherine Buckley 8760
Acc. No. 595
Dennis Donovan Est.
Acc. No. 596
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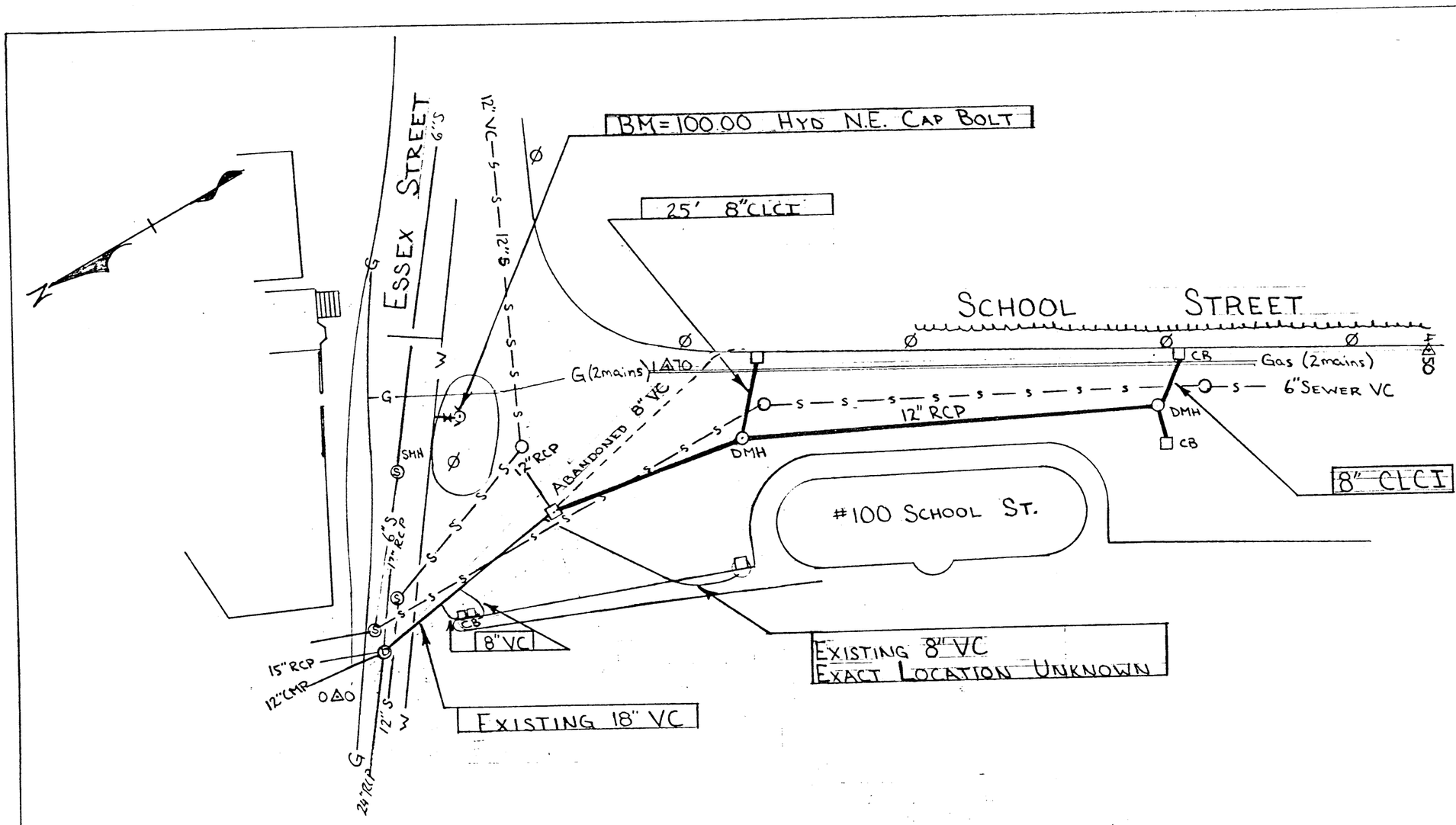
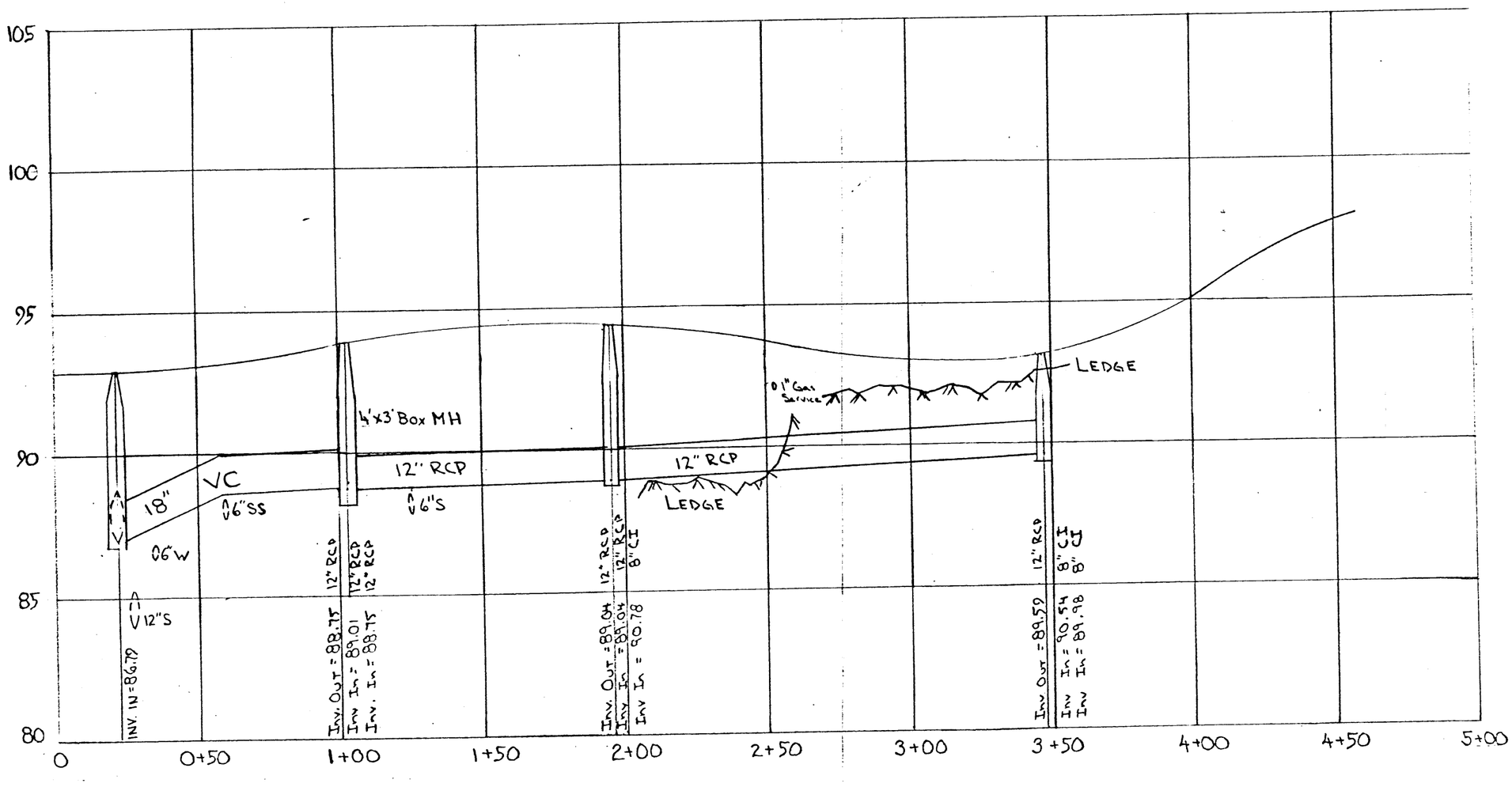




SCALE:
VERT.: 1"=4'
HORIZ.: 1"=40'

TOWN OF ANDOVER, MA
SCHOOL / ESSEX STREET
DRAINAGE CONSTRUCTION

BRIAN W MOORE TOWN ENGINEER
SCALE: AS SHOWN NOV. 15, 1995

As Built DR 140

APPENDIX E: CDM RECOMMENDATIONS

18. Since both these sub-areas also exceeded the threshold for excessive infiltration, these portions of the Shawsheen River Interceptor are susceptible to both groundwater and to a lesser degree, river inflow.

5.2 Recommendations

The findings of the flow monitoring program show that the Andover sewer system is susceptible to both infiltration and inflow. The increase in extraneous flows can be attributed to a combination of sources including groundwater influence, rainfall-derived infiltration and inflow (RDII), and direct inflow sources such as roof leaders and foundation drains. In order to prioritize and phase the SSES work, the drainage sub-areas recommended for further investigation have been categorized as high priority and low priority. **Figure 5-1** shows the drainage sub-areas recommended for the next phase of the I/I remediation program categorized into high and low priority sub-areas. CDM Smith recommends phasing the investigation work over several seasons beginning with the highest prioritized sub-areas. The phased approach will allow for investigations with immediate follow-up rehabilitation work and enable the program to be tailored to the Town's needs. The investigations included in the SSES program will also provide data to the Town that can be integrated into their GIS system and allow the Town to prioritize and perform necessary maintenance.

Note sub-area 26 is comprised of privately owned sewers and therefore has not been included in the recommendations for the Town of Andover's SSES program investigations. A previous investigation of these sewers was conducted in 2012 by John G. Crowe Associates, Inc. System mapping and a May 2012 CCTV inspection report of sub-area 26 completed by Green Mountain Pipeline Services is included in **Appendix D**.

This section summarizes the recommendations for future sewer system inspection work to locate and identify sources of extraneous flow, and provides a schedule with estimated program costs.

5.2.1 Flow Isolation, CCTV Inspection and MSI of Sewers

Conventional I/I remediation practice includes performing flow isolation in drainage sub-areas where flow monitoring results indicate the presence of excessive infiltration and inflow. The objective is to limit the scope of costly investigation and system rehabilitation work to specific segments of the system having the highest levels of extraneous flow. Flow isolation is then typically followed by other source identification testing including closed-circuit television (CCTV) and house-to-house inspections to define the type and severity of individual I/I sources. Remediation costs can then be estimated so that a value-effectiveness analysis can be performed and a sewer system remediation program can be developed based on value-effective I/I removal.

Based on the findings of the flow monitoring program, CDM Smith recommends flow isolation in all high and low priority sub-areas. To maximize effectiveness, flow isolation should be performed during high groundwater periods. CDM Smith therefore recommends beginning the flow isolation with the two highest priority sub-areas (13 and 28) during the spring of 2018. Note that although sub-area 29 ranks slightly higher than sub-area 28, sub-area 28 was paired with sub-area 13 since they are located adjacent to each other geographically. Sub-area 29 is paired with sub-area 3, the next highest priority sub-area, since they are also geographically adjacent areas.

Immediately after flow isolation, follow-up CCTV inspection and multi-sensory inspections (MSI) shall be completed. MSI is recommended for all pipes greater than 18-inches in diameter. Based upon the flow isolation results, it is recommended that CCTV inspection of the sewer system be performed during the same high groundwater season as flow isolation, when infiltration at pipe joints and inflow from service connections can be detected. All CCTV work should be completed by crews certified by the National Association of Sewer Service Companies' (NASSCO) in the Pipeline Assessment Certification Program (PACP). The estimated costs in Section 5.3 assumes 100 percent of the total pipe length is flow isolated and 50 percent of the total pipe length requires follow-up CCTV inspections. Typically, flow isolation with follow-up CCTV inspections is less expensive than no flow isolation and 100 percent CCTV inspections. For example, for sub-areas 13 and 28, if 100 percent of the total pipe length is flow isolated and 50 percent of the total pipe requires follow-up CCTV inspection, the City would be saving approximately $18,000 in subcontractor cost alone when compared to no flow isolation and 100 percent CCTV inspections. Note that 50 percent is an estimate only and the exact amount of sewers to be CCTV inspected will be based upon the results of the flow isolation.

An MSI of the Shawsheen River Interceptor was conducted during October 2017 under a separate task. Therefore, investigation of the interceptor will not be included in the SSES program.

5.2.2 Manhole Inspection Program

CDM Smith recommends performing manhole inspections in both the high and low priority sub-areas, ideally during the high groundwater period. Manhole inspections can be completed in conjunction with CCTV inspections, as CCTV inspection crews are typically certified by NASSCO in the Manhole Assessment Certification Program (MACP). The inspections shall include visually identifying and quantifying sources of extraneous flow entering through defects such as pipe connections, defective shelves, or leaking walls. It is assumed that 90 percent of the manholes within the Town's sewer system will be inspected according to NAASCO's MACP Level 1 inspection, which does not require entry into the manhole. The inspection crew will visually inspect and take photos from the ground surface. The remaining 10 percent will require MACP Level 2 inspection, which requires confined space entry and more detailed evaluation of defects. The benefit of performing manhole inspections according to NASSCO's MACP guidelines is the ability to conduct a comprehensive risk assessment of all manholes within the Town's sewer system by utilizing the MACP defect coding system. Also, the inspections should include taking GPS points at the manholes to determine the horizontal coordinates, vertical rim elevation, and vertical invert elevation. Information from the manhole inspection program will be integrated into the Town's GIS mapping system to update the wastewater collection system mapping and further develop the Town's sewer system database. It is estimated that there are over 4,380 manholes in the Town's collection system. Sub-areas 13 and 28 include approximately 205 manholes, or five percent of the total number of manholes in the Town's sewer system.

5.2.3 Smoke Testing and House-to-House Investigation Program

The flow monitoring program identified a significant volume of inflow entering the sewer system during rainfall events. CDM Smith recommends smoke testing and house-to-house inspections in the high priority sub-areas and the six low priority sub-areas exceeding only the inflow threshold (sub-areas 6, 9, 10, 12, 14 and 18) as established by MassDEP. Since it is not necessary for these investigation programs to be completed during the high groundwater season, it is recommended

that smoke testing and house-to-house inspections be completed for sub-areas 13 and 28 during the summer/fall of 2018 to help locate potential inflow sources and to aid in the further stages of inflow removal. These investigation programs will help target where to implement inflow remediation programs and additional identification programs such as dye testing and CCTV inspection of sewer service connections. Following a review of the smoke testing and house-to-house data, additional inflow identification programs could follow in subsequent seasons; however, CDM Smith recommends not proceeding with these additional programs until the completion of the smoke testing and house-to-house investigation program. It should be noted that the Town's Sewer Use Regulations prohibit illegal connections to the sewer system. Investigation programs such as smoke testing and house-to-house inspections will help identify connections that are in violation of these regulations and allow the Town to begin the process of removing them from the collection system.

5.3 Program Schedule and Estimated Costs

This report section provides a summary of the program schedule and estimated costs for implementing Andover's comprehensive SSES. The program has been developed in coordination with the Town's *Sanity Sewer Master Plan* (CDM Smith, June 2016) and season sensitivity (i.e., high groundwater, low groundwater, etc.) investigations.

5.3.1 Program Schedule

The diagnostic investigations and engineering analyses outlined above constitute a comprehensive SSES.

Once investigations are complete for the designated sub-areas, analysis of the investigation data will result in recommendations for rehabilitation work to address the I/I sources identified. This process will continue as the Town moves down the list of prioritized sub-areas. The next phase of the I/I remediation program has been divided into eight investigation areas; the 16 priority (high and low) sub-areas divided into groups of two, to be implemented over the next fifty years in conjunction with rehabilitation work resulting from the investigations to meet the goals of the Town. Each has been tailored to cost-effectively locate and remove sources of extraneous flow.

The Town's *Sanitary Sewer Master Plan* outlined a schedule for implementation of a flow monitoring and comprehensive SSES program over a period of ten years. The proposed schedule for the SSES program outlined in **Figure 5-6** modifies this schedule based on the findings of this I/I study, updated costs for inspection and rehabilitation work as well as expected funding levels. This schedule will enable the Town to effectively manage the program, provide flexibility in the program based upon the results of the investigations and cost-effectively manage limited resources. Completion of the SSES program and any subsequent investigations will allow the Town to develop alternatives to reduce extraneous flows and compile sufficient data to prepare a cost-effective remediation program.

As shown in **Figure 5-6**, the Town plans to investigate and perform follow-up remediation work on the portion of the sewer system within the top four priority sub-areas (sub-areas 3, 13, 28 and 29) over the next 10 years. These four sub-areas contribute over 19 percent of the total net infiltration to the Town's sewer system. In regard to inflow, these four sub-areas contribute approximately 23 percent of the total net inflow to the sewer system. Overall, the Town will

Task	Year 1	Year 2	Year 3	Year 4	Year 5	Year 6	Year 7	Year 8	Year 9	Year 10	Beyond 10 years
Investigation											
Sub-areas	13, 28						3, 29				4, 5, 1-2, 25
Flow Isolation											
Cleaning and CCTV											
Manhole Inspections											
Smoke Testing											
House to House Investigations											
Engineering and Contingency											
Investigation Sub-total	$152,000						$291,000				$732,000
Rehabilitation											
Sub-areas		13, 28	13, 28	13, 28	13, 28	13, 28		3, 29	3, 29	3, 29	3, 29
Rehabilitation											
Rehabilitation Sub-total		$250,000	$250,000	$250,000	$250,000	$250,000		$250,000	$250,000	$250,000	$1,750,000

Estimated Investigation Cost	**$1,175,000**
Estimated Design and Rehabilitation Cost	**$3,750,000**
Total Cost	**$4,925,000**

CDM Smith

FIGURE 5-6
TOWN OF ANDOVER, MASSACHUSETTS
FLOW MONITORING SUMMARY REPORT
SSES PROGRAM SCHEDULE

target approximately 27 percent of the total I/I entering the sewer system by investigating and rehabbing these four sub-areas, which consist of only 12 percent of the Town's system by length. As previously discussed in this report, the highest priority sub-areas, 13 and 29, contribute the most I/I per gallon/day/inch-mile. Therefore, I/I remediation within these sub-areas will remove the largest amount of I/I per inch-mile of pipe. Once these four sub-areas are completed, the Town will either begin the follow-up investigations of the next two sub-areas on the priority list or step back and re-evaluate the remaining sub-areas that have not been rehabilitated.

5.3.2 Program Costs

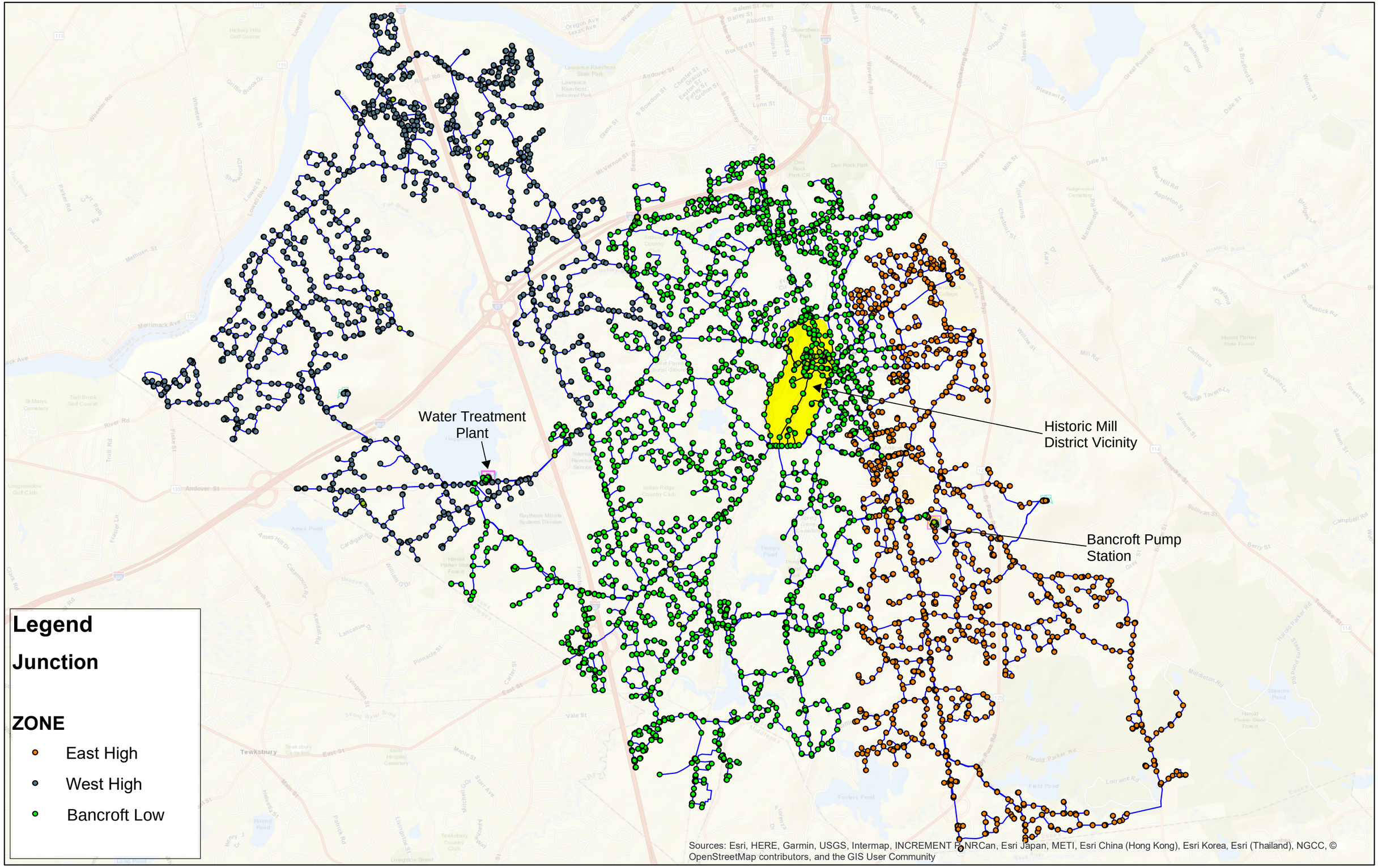
Based on the schedule shown in **Figure 5-6**, CDM Smith estimates that all investigation work recommended for completion in 2018 will cost approximately $152,00, which includes all engineering and subcontractor costs associated with the following work in sub-areas 13 and 28:

- Flow isolation (35,600 LF)
- Cleaning and/or CCTV inspection (17,800 LF)
- Manhole inspections (205 manholes)
- Smoke testing (35,600 LF)
- House to House Investigations (645 buildings)

Note that the costs provided in this Section are a planning level Opinion of Probable Project Cost and will need to be refined prior to work commencing. CDM Smith will work closely with the Town throughout the program to develop in-house cost saving approaches. Data generated from these investigation programs will be the framework for a cost-effective remediation program. Moreover, the remediation program can be prioritized and scheduled to be constructed in conjunction with pending utility and roadway work and adjacent planned developments.

The Town is committed to allocating a minimum of $500,000 biannually to I/I remediation work in the form of a Town Meeting appropriation (local funding). As shown in **Figure 5-6**, this allotment will allow the Town to completely investigate and provide follow-up rehabilitation of sewers located in sub-areas 3, 13, 28 and 29 within the next 17 years. If necessary, the Town will explore other avenues of available funding such as the State Revolving Fund (SRF) low interest loan program to ensure the completion of investigations and recommended rehabilitation work outlined in the SSES program. Note that there is no guarantee that the application for an SRF loan would be approved by the state given the overall need of the project in comparison to other applications submitted. The SRF loan would also require an appropriation to be approved at Town Meeting.

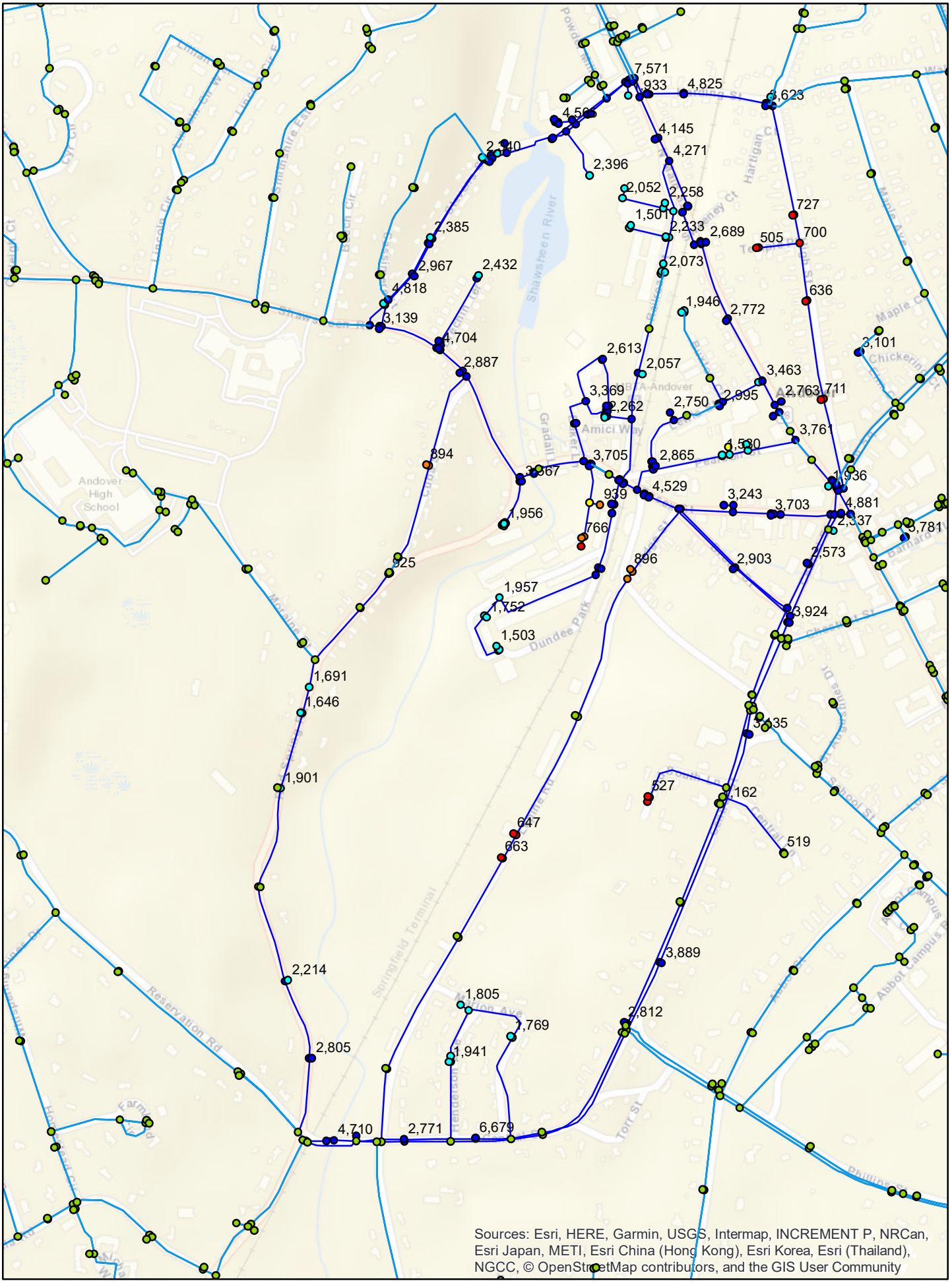
APPENDIX F: DRINKING WATER SYSTEM FIGURE


Water Treatment Plant
Historic Mill District Vicinity
Bancroft Pump Station
Legend
Junction
ZONE
East High
West High
Bancroft Low
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, © OpenStreetMap contributors, and the GIS User Community

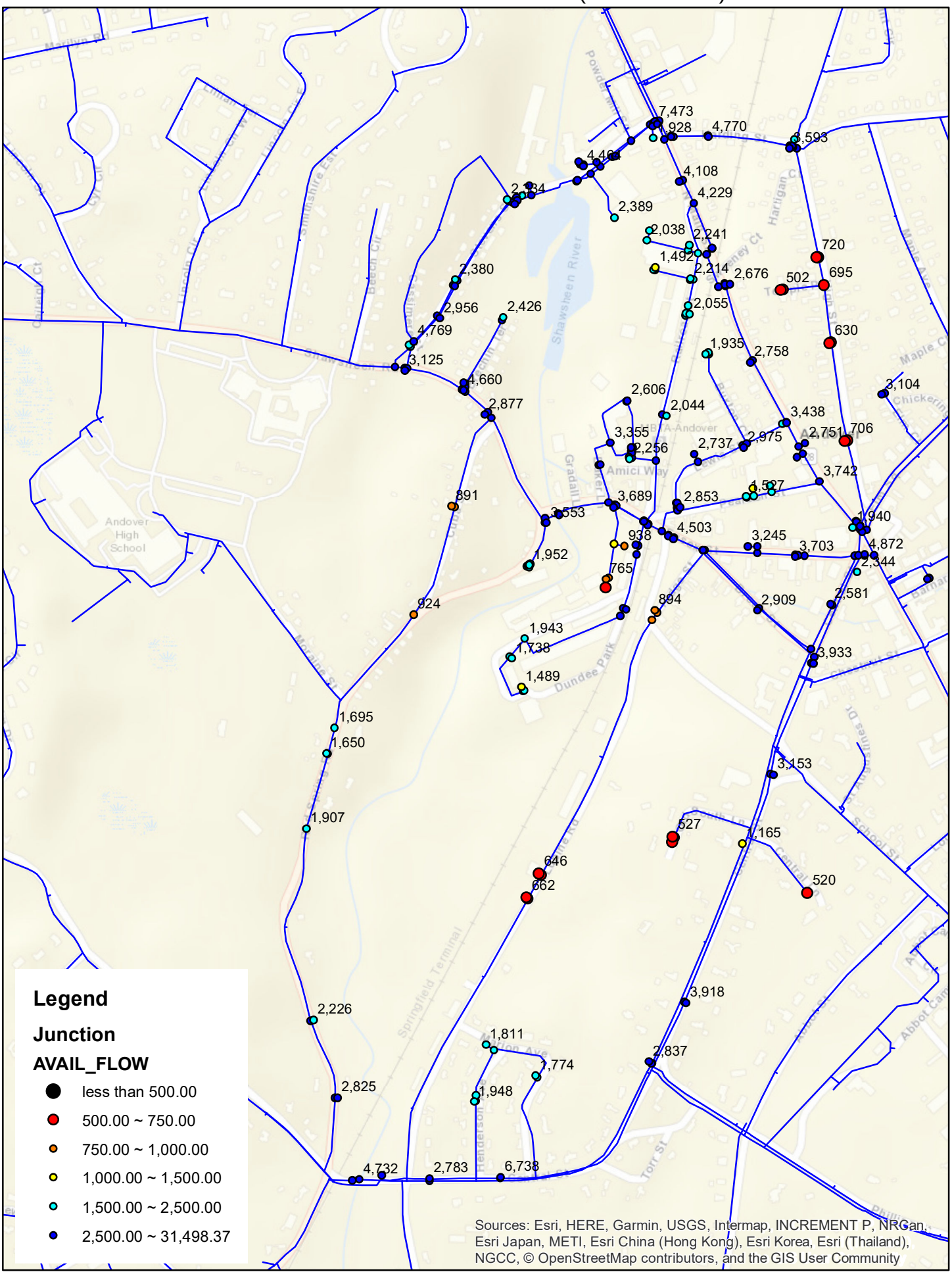
APPENDIX G: HMD HYDRAULIC MODEL FIGURES

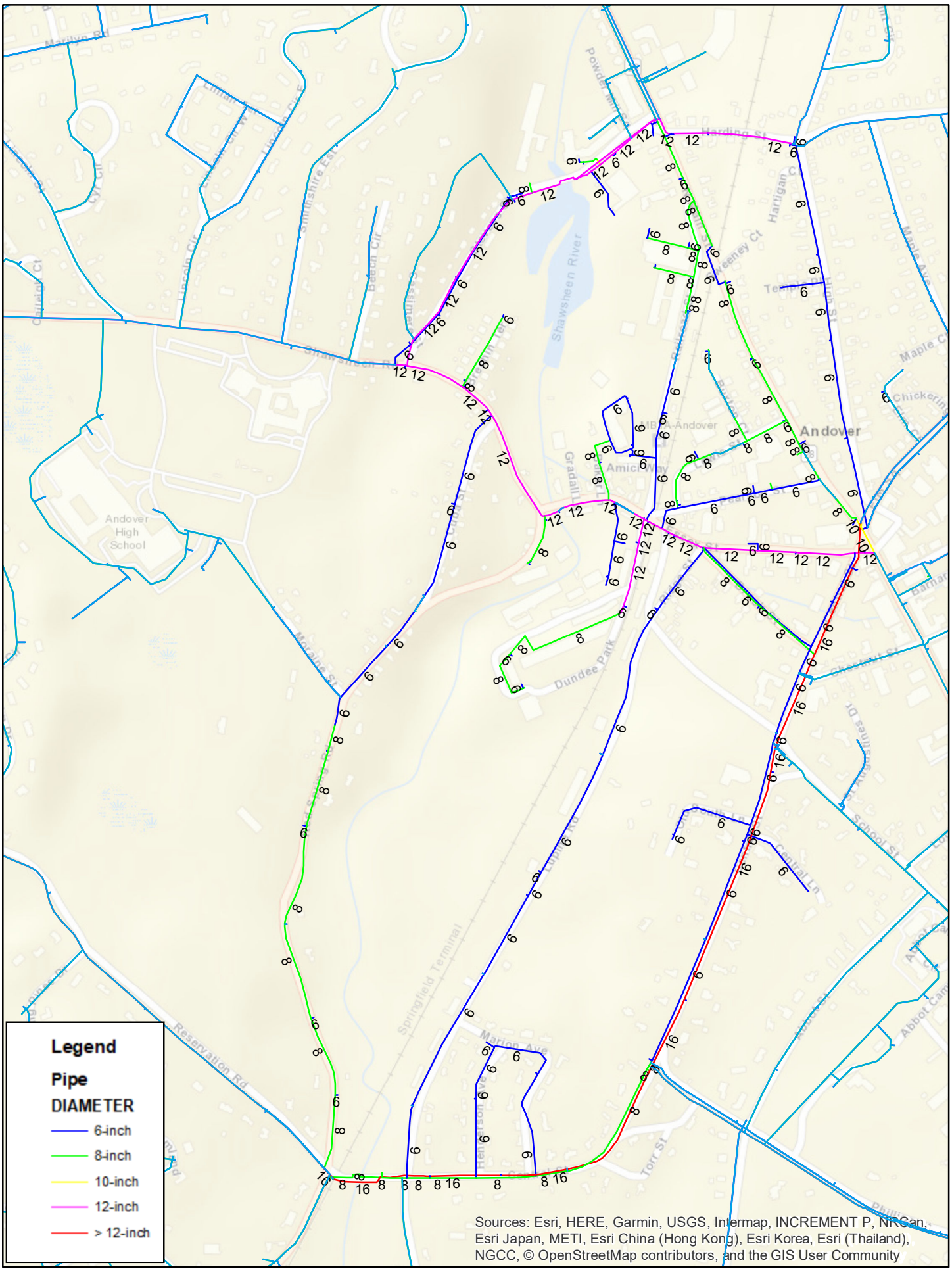
- Water Main Pipe Diameters
- Water Main Roughness Factors
- Water Main Materials
- Scenario 1B – AFF (MDD Conditions at Base Demand)
- Scenario 1B - AFF (MDD Conditions at Peak Demand)
- Scenario 2B – AFF (MDD Conditions at Base Demand)
- Scenario 2B - AFF (MDD Conditions at Peak Demand)

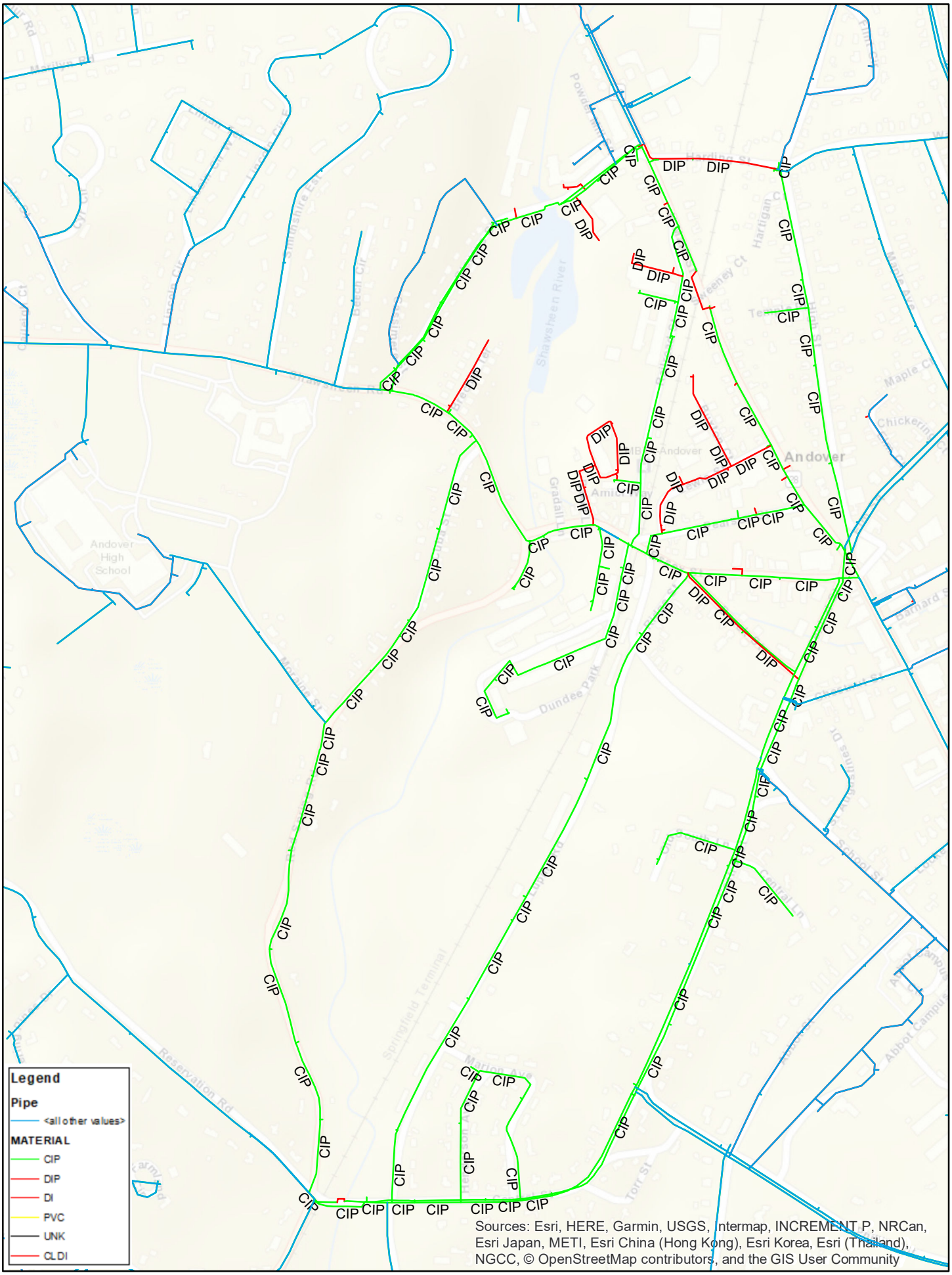
Scenario 2B - Available Fire Flow (Base Demand)

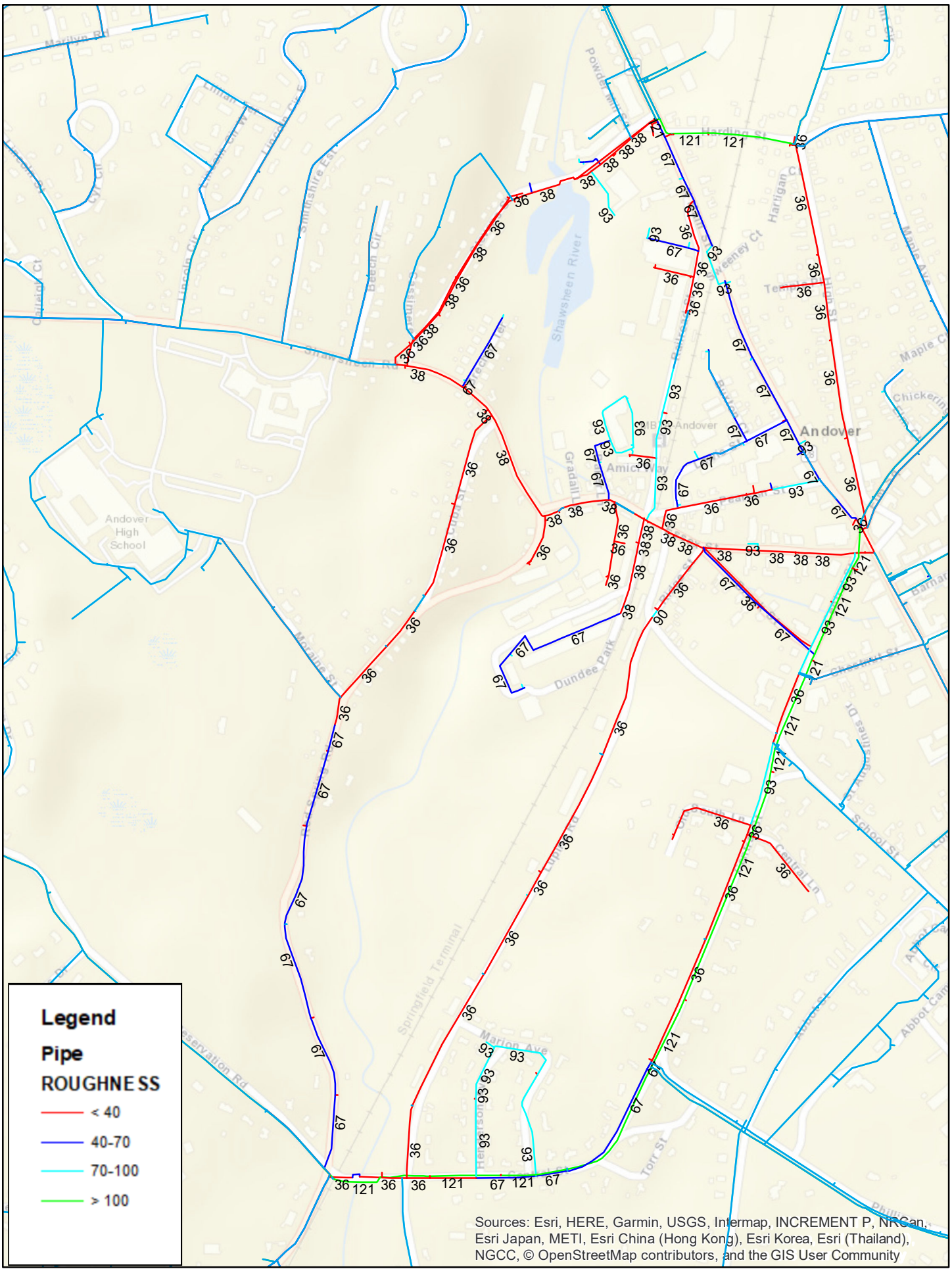


Scenario 2B - Available Fire Flow (Peak Demand)

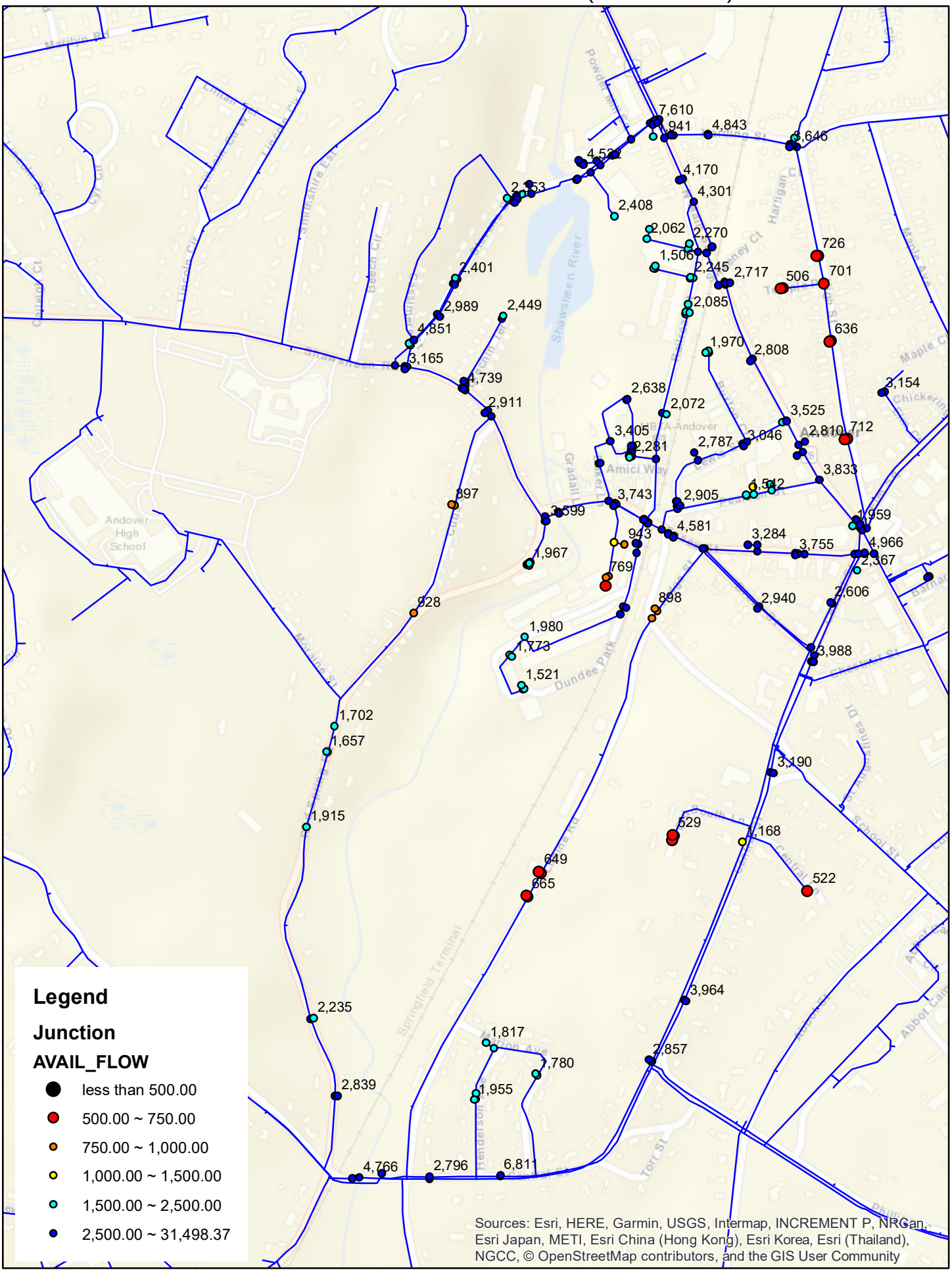



Legend
Pipe
DIAMETER
6-inch
8-inch
10-inch
12-inch
> 12-inch
Andover
Andover High School
Shawsheen River
Springfield Terminal
Dundee Park
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, © OpenStreetMap contributors, and the GIS User Community

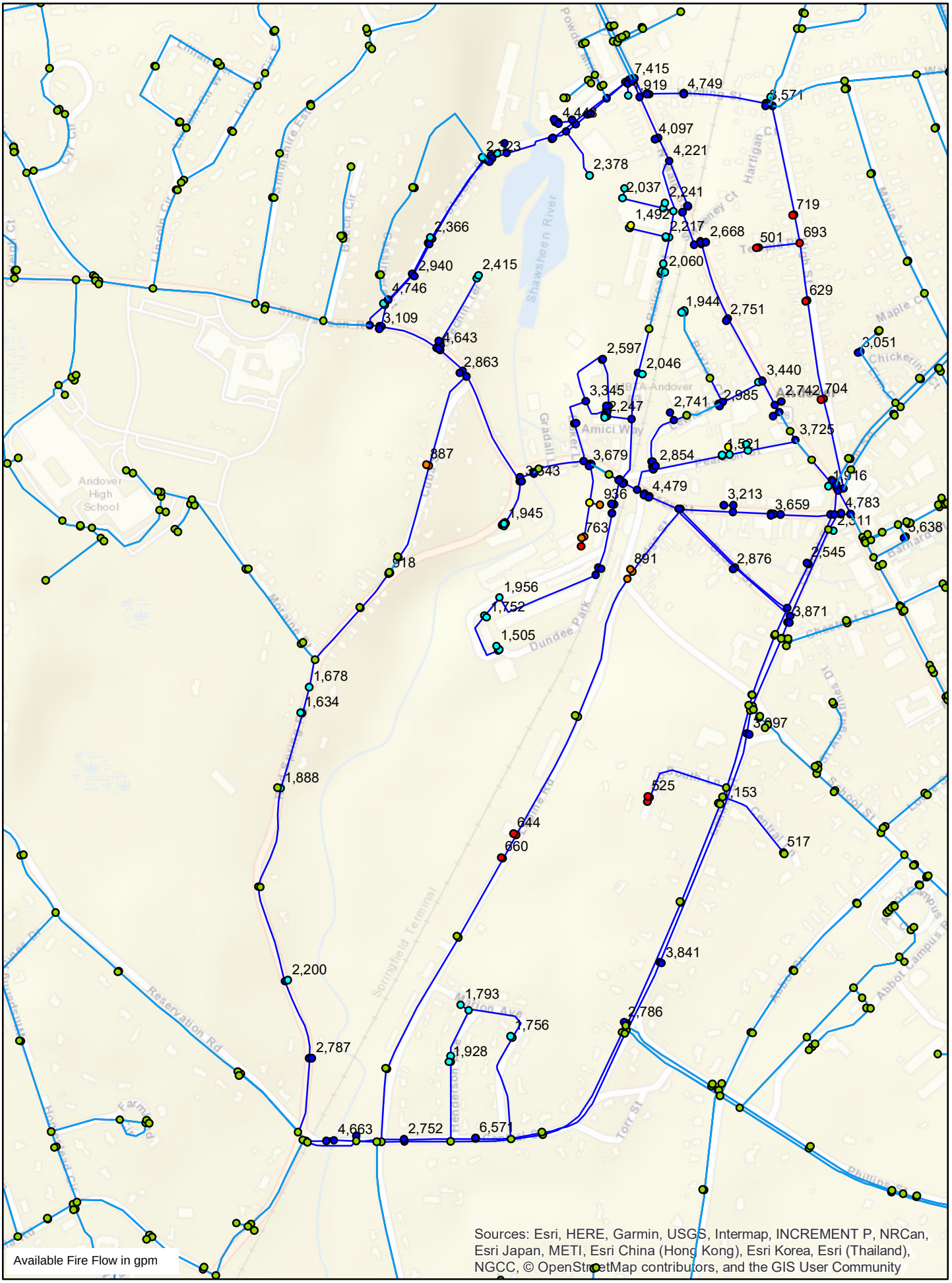

Legend
Pipe
<all other values>
MATERIAL
CIP
DIP
DI
PVC
UNK
CLDI
Andover
Andover High School
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, © OpenStreetMap contributors, and the GIS User Community


Legend
Pipe
ROUGHNESS
< 40
40-70
70-100
> 100
Andover
Andover High School
Shawsheen River
Dundee Park
Springfield Terminal
Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, © OpenStreetMap contributors, and the GIS User Community

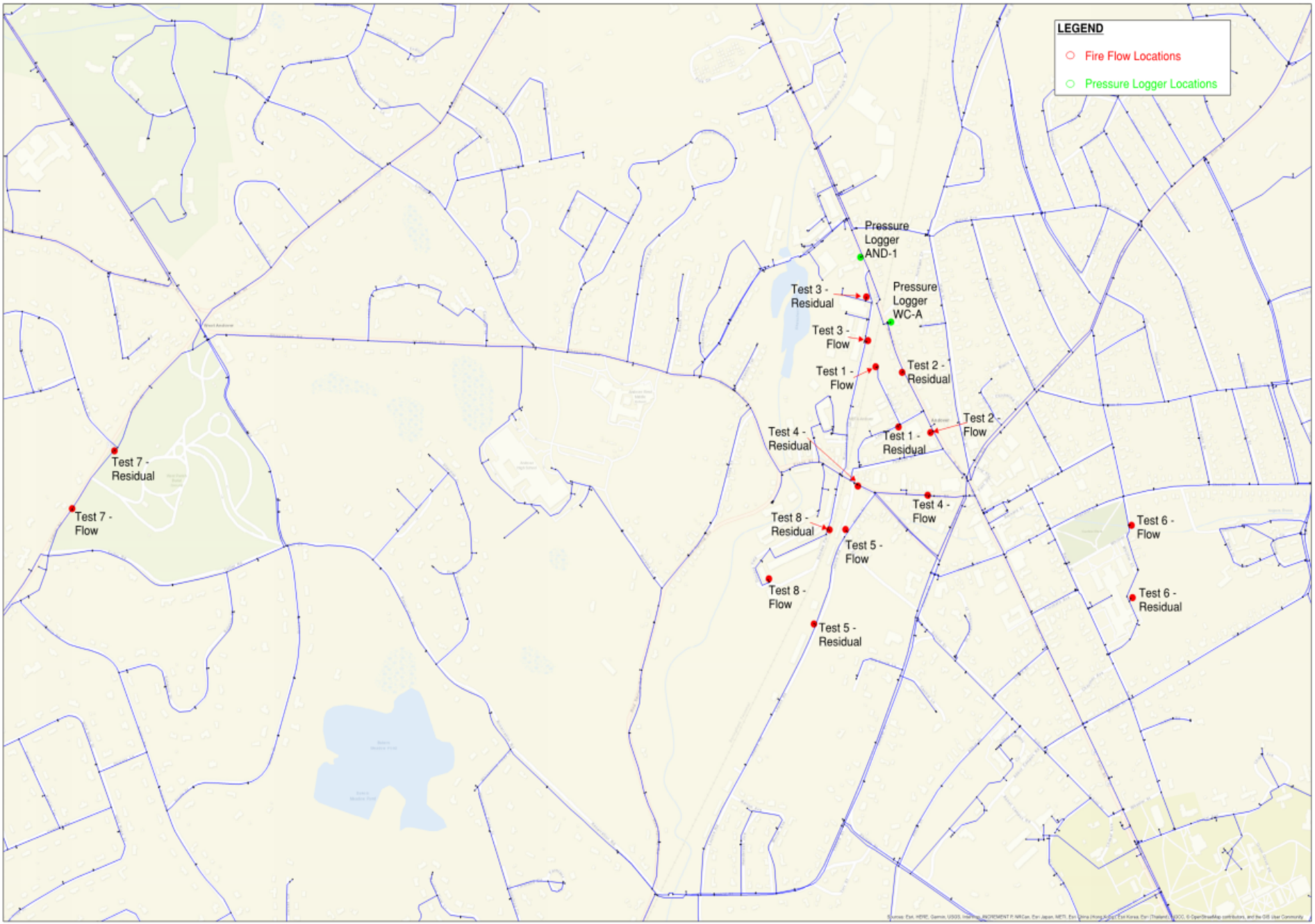
Scenario 1B - Available Fire Flow (Base Demand)



Scenario 1B - Available Fire Flow (Peak Demand)



APPENDIX H: HYDRANT FLOW TEST LOCATIONS


LEGEND
Fire Flow Locations
Pressure Logger Locations
Pressure Logger AND-1
Test 3 - Residual
Pressure Logger WC-A
Test 3 - Flow
Test 2 - Residual
Test 1 - Flow
Test 2 - Flow
Test 4 - Residual
Test 1 - Residual
Test 7 - Residual
Test 4 - Flow
Test 7 - Flow
Test 8 - Residual
Test 6 - Flow
Test 5 - Flow
Test 8 - Flow
Test 6 - Residual
Test 5 - Residual

APPENDIX I: CDM REPORT FIGURES

Figure 4-5
Dascomb Road Wet Weather Scenario with Peak Flow from Expansion Area

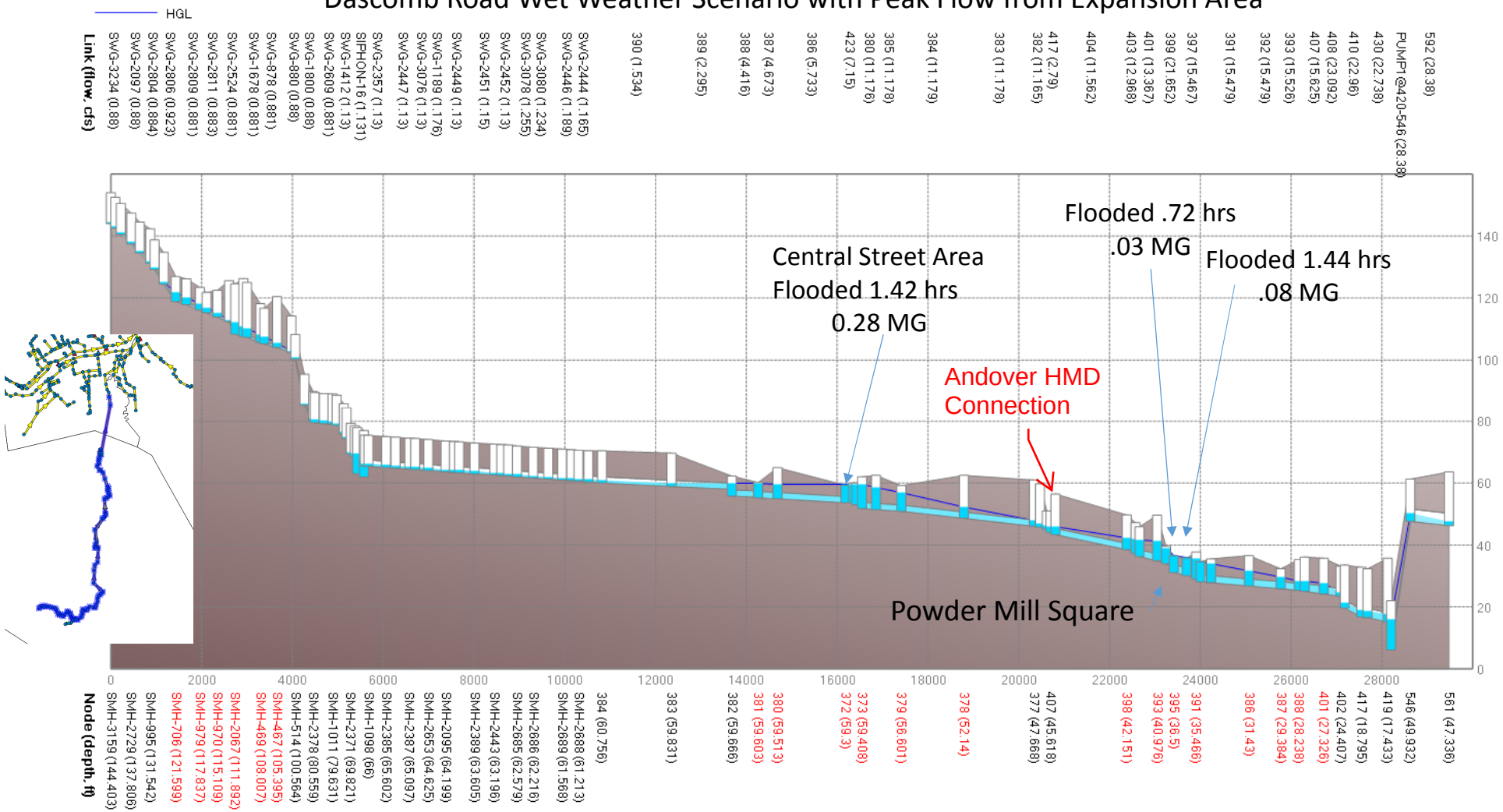
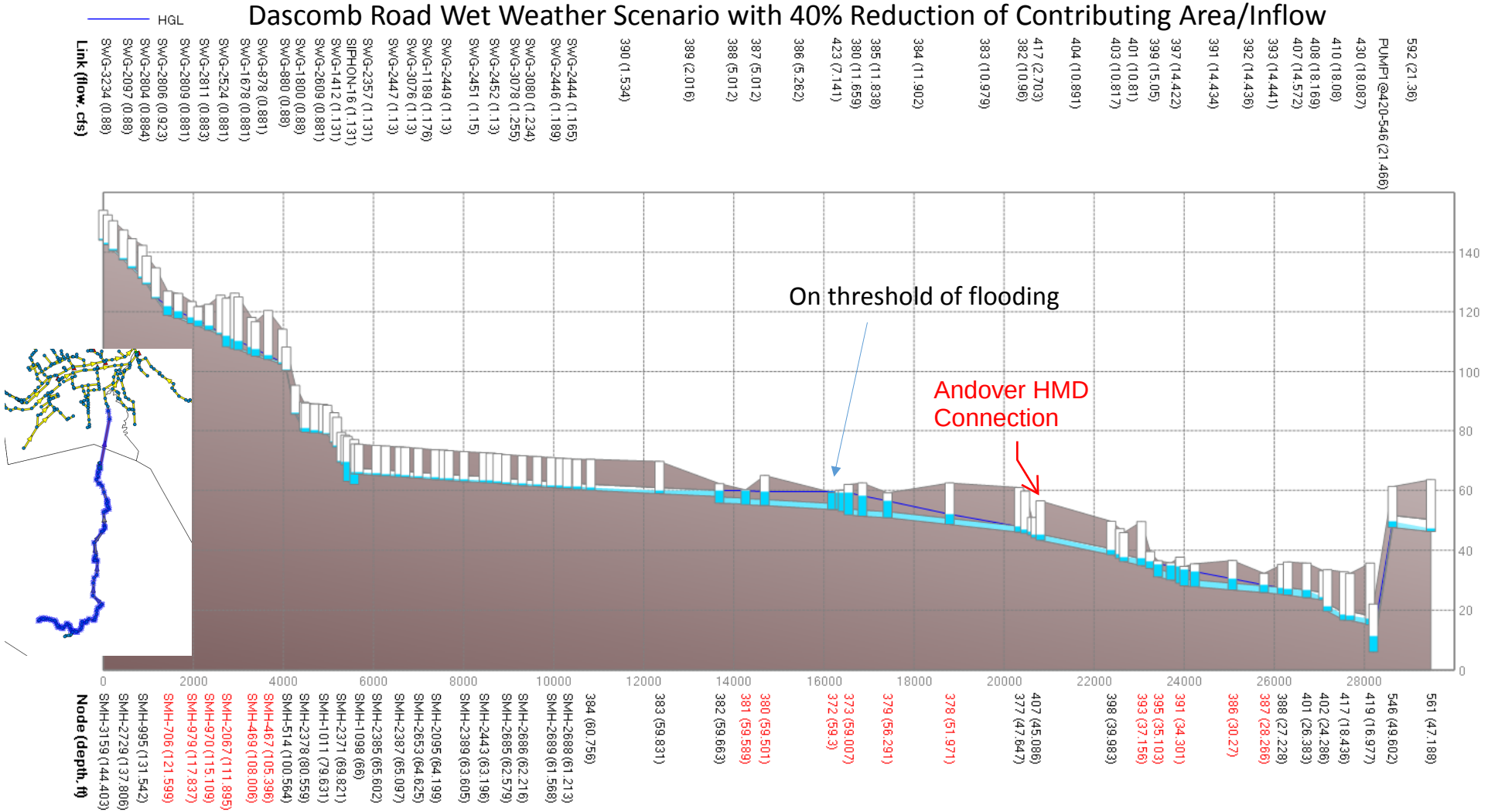


Figure 4-6

Dascomb Road Wet Weather Scenario with 40% Reduction of Contributing Area/Inflow



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