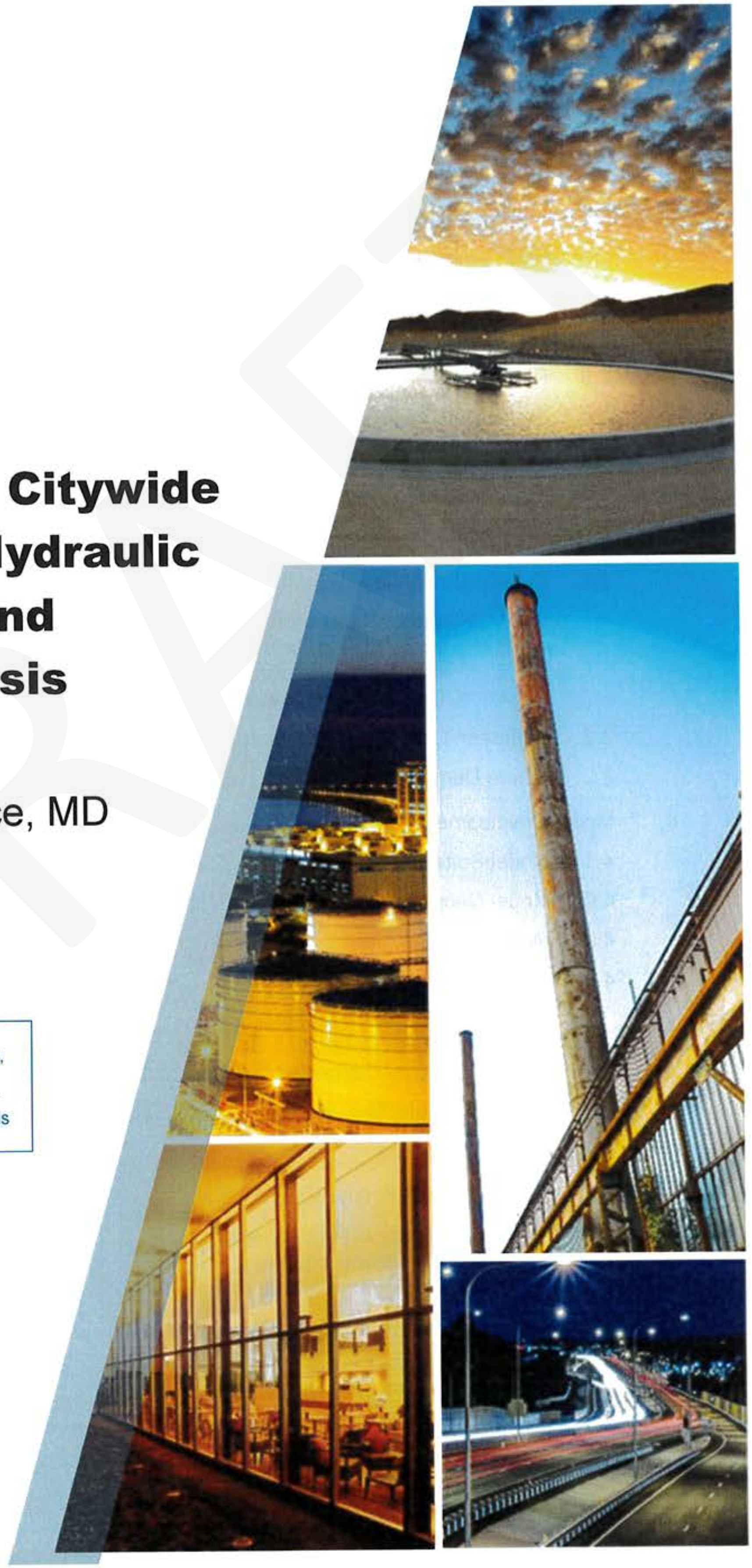
Havre de Grace Citywide Water System Hydraulic Model Update and Hydraulic Analysis

City of Havre de Grace, MD



Draft for Review

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1. Introduction

The City of Havre de Grace (City)'s current water system hydraulic model was developed by a previous consultant prior to 2007. The model was developed schematically and a uniform demand was applied across the entire network, not taking into account large users or other spatial demand variations. In 2009, GHD updated and calibrated the water model, as detailed in the 2009 *City of Havre de Grace Water Distribution System Model* report. GHD utilized geographic data obtained from Harford County to scale the provided model network and recompile the model. During GHD's update, demands were refined somewhat, but were still broken up into groups based on location and evenly applied across the water model within each group. Field testing and calibration were performed for the 2009 report; however, the results of the calibration indicated that there are system components that are unknown and additional pipe condition and hydrant flow testing was recommended, but not performed. The adjustments made to the water model during that calibration were significantly greater than would be normally accepted in engineering practice. As stated in that report the "difficulty of calibrating the Havre de Grace Water Model and the inability to significantly reduce disagreement between the simulated and observed static and residual pressures at several testing locations should be considered before using the model to aid design of future system infrastructure."

The hydraulic model has not been calibrated / validated since 2009, nor have future projections been comprehensively updated since that time. The City's distribution system was also recently surveyed, providing geographic data that can be used to develop a much more complete and detailed hydraulic model. The City requested GHD update the model based on currently available information, conduct hydrant flow field testing to calibrate the model, assist the City with future demand projections, and provide the City with a guide to system improvements to accommodate the projected growth.

The project goal is to develop a calibrated hydraulic model of the City's water distribution system, utilize the model to evaluate the City's ability to supply water, and assist the City in planning system improvements. This report details the process followed to develop the hydraulic model, the results from the model analysis, and recommended improvements to improve performance of the distribution system.

2. City Water Distribution System

2.1 Overview

The City operates and maintains a water distribution system that delivers water to residential, commercial, and industrial connections within the City. Water is supplied from the Havre de Grace Water Treatment Plant (WTP) which treats water withdrawn from the Susquehanna River.

The City consists of four pressure zones. Water is pumped into the lowest pressure zone, Zone 1, directly from the Havre de Grace WTP via the Havre de Grace WTP Pump Station (PS). Water is fed from Zone 1 into Zone 2 by the Graceview Booster PS. Water flows from Zone 2 into two drop zones, Zone 2D-I and Zone 2D-II, via pressure reducing valves (PRVs). Water can also flow from the drop zones into Zone 1 via PRVs, but these are kept closed. A simplified hydraulic profile of the distribution system is shown in Figure 1.

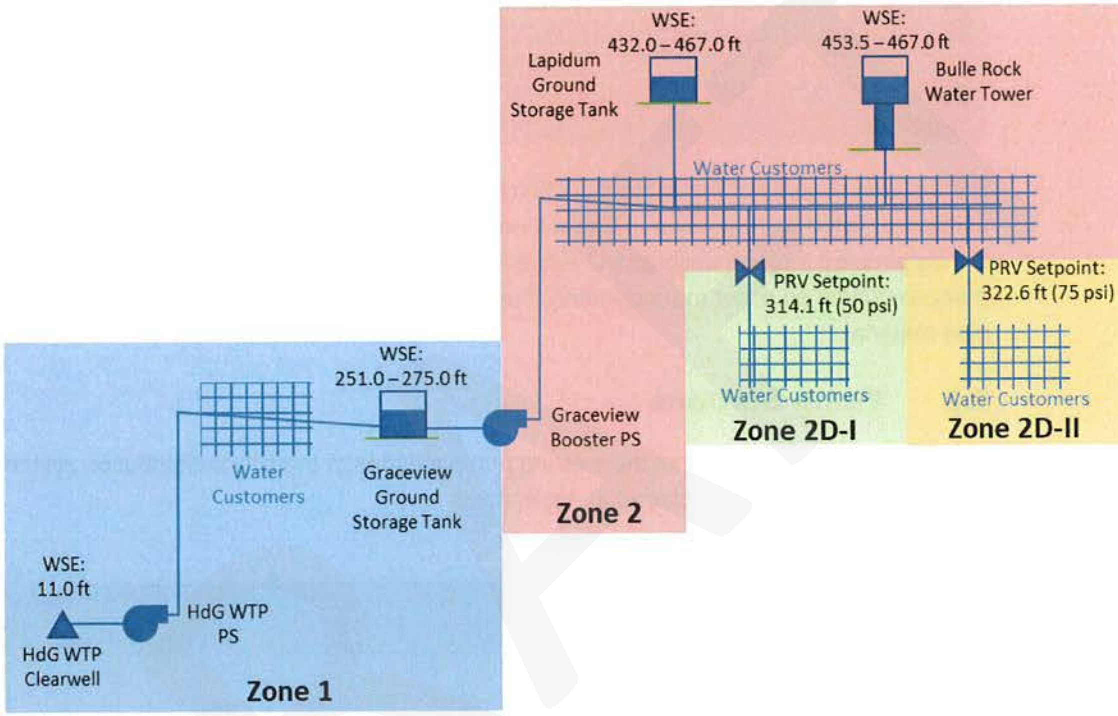

Figure 1 Havre de Grace Distribution System Simplified Hydraulic Profile

2.2 Distribution System Layout

In 2019, the City's water system was surveyed by Wallace Montgomery to develop an updated GIS database of all pipes, valves, and hydrants within the City, which were then reviewed by the City and imported into the City's Beehive Industries asset management system. GHD reviewed this data and worked with the City to identify and correct locations with data gaps, errors, and discrepancies. The data was also reviewed to make sure that each element had a unique identifier. A few pipes, listed as abandoned in the provided dataset, were removed.

See Figure A-1 in Appendix A for a map of the current water distribution system and City municipal boundary. See Table 1 for an inventory of pipe length by diameter of the cleaned up dataset.

Table 1 Distribution System Pipe Inventory

Pipe Diameter (Inches)	Length (ft)	Percent of Total Length
0.75	292	<0.1%
1	2,079	0.6%
1.5	399	0.1%
2	8,200	2.3%
3	320	<0.1%
4	35,769	9.9%
6	49,709	13.8%
8	183,144	50.7%

Pipe Diameter (Inches)	Length (ft)	Percent of Total Length
10	2,546	0.7%
12	66,343	18.4%
16	280	<0.1%
20	12,070	3.3%
30	29	<0.1%
Total	361,180	

For the vast majority of the system (77% by length), the GIS database has pipe material either unlisted or listed as "unknown". This will not impact the modeling as pipe hydraulic characteristics are determined through calibration. However, if the City wishes to use the GIS database for other purposes (such as asset management), they may want to perform field investigations to determine pipe material.

2.3 Pump Stations

See Table 2 for a summary of the existing pump stations in the City's distribution system. Pump curves for all pumps are included in Appendix B.

Table 2 Existing Pump Stations

Pump Station	Number of Pumps	Design Flow per pump (gpm)	Design Head (ft)	Location
Havre de Grace WTP PS [1]	3 [2]			Into Zone 1
3600 rpm		1400	300	
3250 rpm		1000	280	
2880 rpm		700	250	
Graceview Booster PS	3			Into Zone 2
Pumps 1 & 2		360	200	
Pump 3		500	200	

Notes:

1. Havre de Grace WTP PS pumps are on VFD.
2. Although there are four finished water pumps at the Havre de Grace WTP PS, Pump 1 is a smaller pump used for water supply to the County and is not hydraulically connected to the City's distribution system. As such, only Pumps 2, 3, and 4 are included in this analysis.

All three pumps at the Havre de Grace WTP PS are on variable frequency drives (VFDs), allowing their speed to be adjusted if required. During typical operation one pump is in operation and the speed is adjusted automatically based on the discharge pressure recorded by a pressure transducer installed on the common discharge header within the WTP.

Three pumps are installed at Graceview Booster PS. Pumps 1 and 2 are identical and run in automatic and alternate (depending on demand) from fall to the beginning of summer; however, it is rare that both pumps operate simultaneously. During summer, the larger Pump 3 is run in automatic and Pumps 1 and 2 are shut off and placed on backup.

2.4 System Storage

The existing system consists of two ground storage tanks and one elevated storage tank as detailed in Table 3 and Table 4.

Table 3 Existing Water Storage Tank Dimensions

Tank	Diameter (ft)	Elevation (Base) (ft) [1]	Elevation (Minimum) (ft) [2]	Elevation (Maximum) (ft)	Elevation (Overflow) (ft)
Graceview Ground Storage Tank	99.0	241.00	251.00	275.00	276.00
Lapidum Ground Storage Tank	40.0	432.00	432.00	467.00	468.00
Bulle Rock Water Tower [3]	50.0	430.50	430.50	467.00	468.00

Notes:

1. Elevation (Base) is the level datum, the elevation equivalent to a level reading of 0.0 ft.
2. Elevation (Minimum) is the lowest water surface elevation allowable in the tank to provide pressure of 35 psi to all customers in the pressure zone.
3. Bulle Rock Water Tower has a spherical tank and diameter varies by elevation. Diameter listed is an approximate average diameter over the tank's operating range.

Table 4 Existing Water Storage Tank Volume

Tank	Total Volume (gal)	Effective Volume (gal) [1]	Pressure Zone
Graceview Ground Storage Tank	2,000,000	1,382,000	1
Lapidum Ground Storage Tank	400,000	329,000	2
Bulle Rock Water Tower	500,000	500,000	2

Note:

1. Effective volume is based on the minimum and maximum elevations listed in Table 3.

2.5 Pressure Reducing Valves (PRVs)

Water flows from Zone 2 into two drop zones, Zone 2D-I and Zone 2D-II, via PRVs. See Table 5 for a summary of the PRV set points.

Table 5 PRV Set Points

Label	Location	Zone Feeding	Pressure Setting (psi)	Elevation	Hydraulic Grade Setting (ft)
WV-1422	Chapel Road	Zone 2D-I	50.0	198.50	314.11
WV-2001	Bulle Rock Parkway	Zone 2D-II	75.0	149.24	322.65

2.6 Improvement Projects Under Design

The City identified a number of improvement projects that are already under design and will be constructed in the immediate future. These projects include:

- **Old Bay Lane Waterline Extension**: Construction of approximately 1,300 feet of new 8" ductile iron pipe connecting the dead-end 8" pipe along Old Bay Lane to the dead-end 6" pipe in the City Maintenance Yard adjacent to the City of Havre de Grace Wastewater Treatment Plant (WWTP).
- **Waterline Replacement – Green St**: Replacement of approximately 1,500 feet of 4" pipe with new 8" ductile iron pipe along Green St from N Adams St to St. John St.

- **Waterline Replacement – Wilson St**: Replacement of approximately 1,400 feet of aged 8" pipe with new 8" ductile iron pipe along Wilson St from Seneca Ave to Bloomsbury Ave.

See Appendix C for maps showing the improvement projects under design.

3. Demand Analysis

3.1 Current Demands

3.1.1 Water Production Data

Water production data from July 2016 to June 2019 was provided by the City. See Appendix D for a graph and Table 6 for a summary of daily water production during this period.

Table 6 City Finished Water Production Data Summary

	Value
Average Day	1.361 mgd
Maximum Day	2.100 mgd
Maximum Day Multiplier	1.54

3.1.2 Water Billing Data

Water billing data for fiscal year 2019 (July 2018 to June 2019) were provided by the City. The total billed water and the total produced water over fiscal year 2019 are compared in Table 7.

Table 7 Water Production and Water Billing Data Comparison (FY 2019)

	Total Annual Volume	Average Daily Volume
Total Water Produced	517.212 mg	1.417 mgd
Total Water Billed	434.866 mg	1.191 mgd
Percent Billed (Total Billed / Total Produced)	84%	
Global Adjustment Factor (Total Produced / Total Billed)	1.19	

There is a minor amount of unbilled water, which can be caused by a number of factors including:

- Water lost in the distribution system due to leaks and water main breaks.
- Unmetered connections.
- Inaccurate or uncalibrated water meters underreporting water usage.

Although the amount of unbilled water is not unreasonably high, it is recommended that the City investigate potential causes of the unbilled water.

To account for the unbilled water, the total water produced was divided by the total water billed to determine a global adjustment factor.

3.1.3 Demand Calculation

The average day demand (ADD) for each water account was calculated by multiplying the account's average daily volume of billed water by the global adjustment factor calculated in Table 7. The maximum day demand (MDD) for each water user was calculated by multiplying the ADD by the maximum day multiplier calculated in Table 6.

The Peak Hour Demand (PHD) was calculated based on the AWWA Standard Diurnal Curve for a 24-hour period (AWWA Manual M32, 1989). The peak hour multiplier per the Standard Diurnal Curve is 1.75 times the MDD, equivalent to 2.70 times the ADD.

See Table 8 for a summary of system demands.

Table 8 System Demand Summary

	Demand (mgd)
Total ADD (Total Water Billed * Global Adjustment Factor)	1.417
Total MDD (ADD * Maximum Day Multiplier)	2.182
Total PHD (MDD * Peak Hour Multiplier)	3.826

3.1.4 Demand Geolocation

Each billing account was georeferenced and linked to the associated parcel within the City, utilizing the billing account address. See Figure A-2 in Appendix A for a map of the current ADD. Total demand within each pressure zone is summarized in Table 9.

Table 9 System Demand Summary by Zone

Zone	ADD (mgd)	MDD (mgd)	PHD (mgd)
Zone 1	1.062	1.635	2.867
Zone 2	0.286	0.440	0.772
Zone 2D-I	0.045	0.070	0.123
Zone 2D-II	0.024	0.037	0.065
Total	1.417	2.182	3.826

3.2 Aberdeen Connection Demand

The City is in the process of negotiating an agreement with the City of Aberdeen (Aberdeen) to provide Aberdeen with a minimum demand of 300,000 gpd and a maximum demand of 900,000 gpd. Design of this connection is underway and the connection could be made in the very near future.

3.3 Future Demand Projections

The City Department of Planning and Zoning provided information on areas within the current City boundary that are anticipated to be developed and potential annexation areas not currently served by City water but anticipated to be in the future.

Future demand areas are shown in Figure A-3 in Appendix A and are listed below:

- **Blenheim Run**: A 9.8-acre undeveloped property, not currently within the City. The developer is proposing a mixed used project with 150 apartments with limited retail and office space.
- **Bulle Rock Redevelopment**: Multiple areas for development within the Bulle Rock area, currently undeveloped and within the City boundary. In total, approximately 764 units are estimated with a mix of single-family dwellings, townhouses, and condominiums.
- **Chapel Road Properties**: 12 developed single-family dwellings, not currently within the City.
- **Green-Ianniello-Patrone Properties**: Approximately 239 acres of undeveloped land within the current City boundary. Currently, 800 residential units are estimated with an equal mix of single-family dwellings, townhouses, and multifamily as well as limited commercial development.
- **Greenway Farms Phase 2 & 3**: Approximately 81 acres of undeveloped land within the current City boundary. Currently, 296 units of townhouses and condominiums are estimated.
- **Havre de Grace Heights**: 65 parcels, most of which are developed, outside of the City. The majority of parcels (43) are already served by City water, so estimated future demand is based on the remaining 22 parcels not currently served by City water.
- **Lampson Property**: Approximately 23 acres of undeveloped land outside of the City boundary. The property has the potential for up to 25 single-family lots.
- **Mixed Office Employment (MOE) Site**: Approximately 95 acres of undeveloped land within the City boundary. At this time, there are three separate development scenarios:
  - Scenario 1: 1,000,000 square feet of commercial space – 65% office, 20% retail, 15% institutional
  - Scenario 2: 1,000,000 square feet of commercial space – 40% office, 60% retail
  - Scenario 3: 400 residential units, 50% single-family dwellings, 50% townhomes

  To be conservative, the scenario with the highest demand (Scenario 1) will be assumed in all evaluations.
- **Old Bay Lane Industrial Redevelopment**: The City would like to provide water use allocation for a potential future high water use industrial tenant to be able to move into the Old Bay Lane industrial park. A future water demand of 100,000 gpd is allocated for this purpose.
- **Old Town Infill**: Undeveloped lots within the Old Town area of the City. There is the potential for up to 75 residential units when these lots are developed.
- **Shawnee Brooke**: 1 undeveloped parcel and 27 developed single-family dwellings, outside of the City boundary. Four of the parcels are already served by City water, so estimated future demand is based on the 24 parcels not currently served by City water.
- **Susquehanna Hills**: 2 undeveloped parcels and 93 developed single-family dwellings, outside of the City boundary.
- **Tranquility Homes Redevelopment**: Approximately 22 acres of undeveloped land within the City boundary. Anticipated number of residential units vary from 75 to 150 depending on lot size and type (single-family dwelling or townhomes). To be conservative, estimated demand was calculated based on the higher 150-unit estimate.

- **US Route 40 – Section 1**: 42 parcels, most of which are developed, outside of the City boundary. Nine of the parcels are already served by City water, so estimated future demand is based on the 33 parcels not currently served by City water. Developed properties are predominantly commercial, and future development type is unknown but assumed to be commercial for this study.
- **US Route 40 – Section 2**:19 parcels, most of which are developed, outside of the City boundary. Developed properties are predominantly commercial, and future development type is unknown but assumed to be commercial for this study.

See Appendix E for detailed calculation of the projected demand assumed for each future demand area. Calculations were based on unit demand assumptions obtained from Design Guidelines for Wastewater Facilities, from Maryland Department of the Environment Engineering and Capital Projects Program, published in 2016. A summary of the total projected demand is presented in Table 10.

Table 10 Future Demand Summary

Future Demand Area	Size / Quantity	Unit Demand (GPD/Unit)	Average Day Demand (gpd)
Residential	2,413 units	250	603,250
Commercial – Office	660,000 sf	0.09	59,400
Commercial – Retail	324,000 sf	0.18	58,320
Institutional / Medical Office Buildings	150,000 sf	0.62	93,000
Industrial	n/a	n/a	100,000
Total			913,970

Future demands were broken down into the following projection periods: 5-year, 10-year, and 20-year / buildout. The projection periods were selected for the following reasons:

- 5 year: In this time range growth and flow projections are almost certain to happen. This time period also is what is feasible for any new project from inception through construction.
- 10 year: This time range the actual growth is more speculative. While likely to happen, the probability of happening is not as great as early time frame projections. Likely an update will be necessary in 5-years before committing to the improvement.
- 20 year / Buildout: This time range is really about budgeting for the future. It allows the City to understand future expenditures.

A summary of the projected future demand is presented in Table 11.

Table 11 Future Demand Summary by Projection Period

Future Demand Area	Demand – 5 yr (gpd)	Demand – 10 yr (gpd)	Demand – 20 yr / Buildout (gpd)
Blenheim Run	40,200		
Bulle Rock Redevelopment	191,000		
Chapel Road Properties			3,000
Green-Ianniello-Patrone Properties	201,800		

Future Demand Area	Demand – 5 yr (gpd)	Demand – 10 yr (gpd)	Demand – 20 yr / Buildout (gpd)
Greenway Farms Phase 2 & 3	74,000		
Havre de Grace Heights		5,500	
Lampson Property		6,250	
MOE	187,500		
Old Bay Lane Industrial Redevelop.	100,000		
Old Town Infill	18,750		
Shawnee Brooke		6,000	
Susquehanna Hills		23,750	
Tranquility Homes Redevelopment	37,500		
US Route 40 – Section 1			11,880
US Route 40 – Section 2			6,840
Total	850,750	41,500	21,720
Grand Total (All Demand Periods)		913,970	

The total average day, maximum day, and peak hour demand, including existing and future demands, for each projection period are summarized in Table 12. Total existing and future demand within each pressure zone is summarized in

Table 13. Total future demand includes the Aberdeen connection demand (see Section 3.2).

Table 12 Total Demand Summary by Projection Period

	Demand – Current (gpd)	Demand - After Aberdeen Connection (mgd) [1]	Demand – 5 yr (mgd)	Demand – 10 yr (mgd)	Demand – 20 yr / Buildout (mgd)
Average Day	1.417	2.317	3.168	3.209	3.231
Maximum Day	2.182	3.082	4.392	4.456	4.490
Peak Hour	3.826	4.726	7.023	7.135	7.194

Note:

1. Maximum Aberdeen demand of 900,000 gpd assumed for all scenarios.

Table 13 System Demand Summary by Zone – Current & All Future Demands

Zone	ADD (mgd)	MDD (mgd)	PHD (mgd)
Zone 1	2.285	3.033	4.640
Zone 2	0.813	1.252	2.195
Zone 2D-I	0.048	0.074	0.130
Zone 2D-II	0.085	0.131	0.229
Total	3.231	4.490	7.194

4. Model Development

4.1 Model Software

A steady state hydraulic model of the City's water distribution system was developed in WaterCAD CONNECT Edition Update 2 by Bentley Systems, Inc. This program analyzes pressurized flow from storage structures and pumping systems through pipe networks and is one of the industry standards for hydraulic modeling.

4.2 Model Geometry

The cleaned up GIS database (see Section 2.2) was utilized as the basis of the hydraulic model. Pipes and valves were imported into the model utilizing the ModelBuilder tool and the model automatically created junction elements at the ends of each water pipe. Valve and junction elevations were interpolated from two-foot contour data, provided by the City, utilizing the Terrain Extractor (TRex) tool within WaterCAD.

The GIS database provided also included water hydrants, hydrant valves, and some (but not all) hydrant leader pipes. These items are not necessary for proper functioning of the hydraulic model and as such were excluded from the data brought into the model (as is typical when modeling distribution systems).

Some pipes in the GIS database were imported into the model but kept inactive in all scenarios. Pipes kept inactive include:

- 2" pipe along Alliance St, Union Ave, and Lafayette St, noted as "Disconnected" in the provided GIS data. Active 4" to 8" pipe runs parallel to the inactive pipe in this area.
- 8" pipe along Wilson St from Seneca Ave to Bloomsbury Ave, noted as "Currently shut off" in the provided GIS data. This pipe runs through the proposed Tranquility Homes Redevelopment and is assumed to stay inactive until that development is complete.
- 8" pipe extending south from Timonium Ct, noted as "Disconnected" in the provided GIS data. This pipe is a dead-end and would have no impact on the hydraulic model.
- 8" to 20" pipe within the Havre de Grace WTP and along St. John St, Warren St, the Amtrak corridor, and Route 40, terminating near Martha Lewis Blvd. This is a dedicated pipeline owned and operated by Harford County and not part of the City's distribution system.

4.3 Model Demand

Parcels identified as water users (see Section 3.1.3) were imported into the model as Customer Meter elements, located at the centroid of each parcel polygon. Each Customer Meter element was associated to the nearest junction or valve. For each demand scenario, the demand for each parcel was calculated and imported into the relevant model scenario.

4.4 Model Calibration

4.4.1 Hydrant Flow Testing

GHD conducted hydrant flow testing of the City's water distribution system on November 19, 2019. A total of fifteen (15) hydrant flow test locations were chosen to accurately illustrate the condition of different sections of the distribution system. See Figure A-4 for a map of the hydrant flow test locations.

At each location, a flow hydrant and test hydrant were selected. At the flow hydrant, pitot pressure was recorded and converted to an observed flow rate. At the test hydrant, static and residual pressures were recorded. Start and stop time for each test were recorded, and tank levels and pump status updates were provided throughout the day of testing. See Table 14 for a summary of the hydrant flow testing.

Table 14 November 19, 2019 Hydrant Flow Testing Summary

Site	Start Time	End Time	Flow Hydrant			Test Hydrant		
			Hydrant ID	Pitot Pressure (psi)	Calculated Flow (gpm)	Hydrant ID	Static Pressure (psi)	Residual Pressure (psi)
17	9:40 AM	9:42 AM	WH330	7	< 533	WH329	106	28
16	10:03 AM	10:05 AM	WH312	26	860	WH313	110	56
9	10:25 AM	10:27 AM	WH494	23	809	WH493	73	52
14	10:47 AM	10:49 AM	WH317	27	876	WH316	88	60
12	11:08 AM	11:10 AM	WH113	20	754	WH114	106	60
13	11:28 AM	11:30 AM	WH97	24	826	WH98	64	48
11	11:43 AM	11:45 AM	WH36	17	695	WH35	64	40
10	12:00 PM	12:02 PM	WH47	15	653	WH46	56	26
15	12:11 PM	12:13 PM	WH64	35	998	WH65	84	62
18	1:13 PM	1:15 PM	WH230	45	1131	WH231	126	86
19	1:32 PM	1:34 PM	WH166	3	< 533	WH167	126	122
3	1:56 PM	1:58 PM	WH464	42	1093	WH463	100	84
8	2:15 PM	2:17 PM	WH399	28	893	WH398	68	56
6	2:33 PM	2:35 PM	WH372	68	1390	WH371	130	120
4	2:50 PM	2:52 PM	WH2	27	876	WH1	60	56

4.4.2 Model Calibration Scenario

The results of the hydrant flow testing were utilized to calibrate the hydraulic model. Separate model scenarios were set up for each flow test. For each scenario, the water surface elevation in each tank and the operational setting of Graceview PS were set based on the information provided by the City. The Havre de Grace WTP PS contributes a significant amount of head to Zone 1, and the pump head can vary significantly as the pump speed is adjusted by the VFD (as discussed in Section 2.3). Therefore, for calibration purposes, a hydraulic grade line (HGL) was set at the Havre de Grace WTP PS such that the model matched, as best as possible, the observed static pressure for each hydrant flow test.

Pipe roughness, and therefore headloss, is modeled though the use of dimensionless Hazen-Williams C coefficients, with a higher C coefficient indicating a smoother pipe with lower headloss, and a lower C coefficient indicating a rougher pipe with higher headloss. Typically, Hazen-Williams C coefficients range from around 60 to 150.

Prior to calibration, the Hazen-Williams C coefficient for all pipes in the model was set to 130. The model was then calibrated by adjusting the pipe C values throughout the distribution system so that the model pressures matched, as close as possible, the observed residual pressure for each hydrant flow test. To do this, the distribution system was broken down into different geographic areas in which the C value is assumed to be consistent, and the C value of the pipes in each area were adjusted together. See Figure A-5 in Appendix A for a map of each C value area.

The stipulated model calibration goals were to obtain model pressures at the modeled test hydrants that were within +/- 20% or 10 PSI, whichever is greater, of the values measured at the field test hydrants.

4.4.3 Model Calibration Results

For the Zone 1 tests, an HGL was set at the Havre de Grace WTP PS discharge point based on the observed static pressures at the hydrant flow test. The pump head necessary to achieve the HGL was calculated and compared to the flow recorded by the City's flow meter installed within the pump station. See Table 15 for a summary and Figure 2 for a comparison of the model pump operating point to the installed pump curves. As shown, the assumed operating points are within the expected range of the pump.

Table 15 Assumed WTP HGL for Zone 1 Hydrant Flow Tests

Test Location	HdG WTP HGL (ft)	Pump TDH (ft)	Pump Flow (gpm)
17	291	281	1,096
16	328	318	1,113
9	330	320	1,151
14	335	325	1,189
13	329	319	1,184
15	335	325	1,161
18	320	310	1,116
19	328	318	1,053

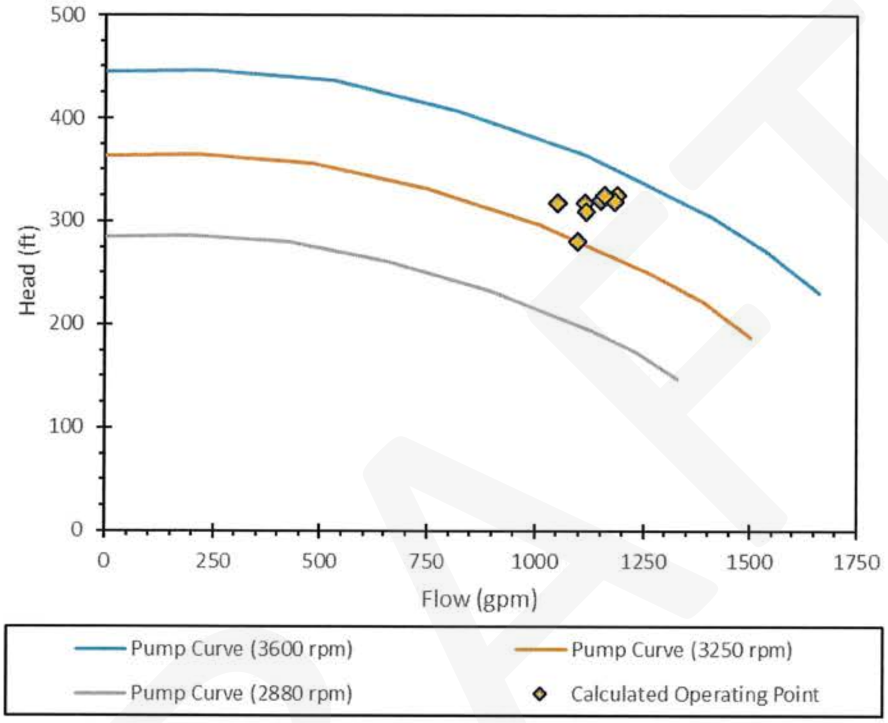

Figure 2 Havre de Grace WTP PS Assumed Operating Points Compared to Pump Curves

The Hazen-Williams C values used in the calibrated model are summarized in Table 16.

Table 16 Hazen-Williams C Coefficients from Calibration

Adjustment Group	Hazen-Williams C
Bayview Estates	63
Blenheim Ln Development	110
Bulle Rock Neighborhood (Lower)	100
Bulle Rock Neighborhood (Upper)	130
Canvasback Dr Neighborhood	130
Chapel Rd Main	110
Downtown	39
Graceview PS + Tank	130
Greenway Farm	105
Havre de Hills Neighborhood	115
Level Rd Main	120
Maryland Ave to Bayview Dr Neighborhood	78
Old Bay Ln Industrial	41
Route 40 Mains	80
Seneca Ave Neighborhood + WWTP	52
Upper Downtown	90
Village Dr to Lewis Ln Neighborhood	114

Adjustment Group	Hazen-Williams C
Wakefield Dr to Graceview Dr Neighborhood	60
Tydings Rd to Ontario St Neighborhood	110

The Hazen-Williams C values obtained from the calibration are reasonable given what is known about the distribution system. The Downtown area is the oldest part of the system and previous analysis and field observations have shown evidence of severe tuberculation in this area, so a very low C value for this area makes sense. It's likewise not surprising that the Seneca Ave Neighborhood + WWTP and Old Bay Ln Industrial areas have low C values, given the age of the system there. The newest developments, including Blenheim Ln Development, Bulle Rock Neighborhood (Upper), Canvasback Dr Neighborhood, and other areas in Zone 2, have the highest C values, as would be expected.

The results of the calibration are summarized in Table 17.

Table 17 Calibration Results

Test Location	Pressure Zone	Static Pressure (psi)			Residual Pressure (psi)		
		Observed	Simulated	% Diff.	Observed	Simulated	% Diff.
17	Zone 1	106	106	0%	28	28	0%
16	Zone 1	110	109	-1%	56	57	2%
9	Zone 1	73	73	0%	52	53	2%
14	Zone 1	88	87	-1%	59	61	3%
12	Zone 2	106	101	-5%	60	60	0%
13	Zone 1	64	64	0%	48	48	0%
11	Zone 2D-I	64	62	-3%	40	41	3%
10	Zone 2D-I	56	55	-2%	26	25	-4%
15	Zone 1	84	83	-1%	62	62	0%
18	Zone 1	127	126	-1%	87	88	1%
19	Zone 1	126	125	-1%	122	120	-2%
3	Zone 2	100	97	-3%	84	84	0%
8	Zone 2D-II	68	70	3%	56	58	4%
6	Zone 2	130	127	-2%	120	116	-3%
4	Zone 2	60	58	-3%	56	55	-2%

All pressures are within +/- 5% between observed and simulated conditions.

4.5 Model Hydraulic Settings

Historic tank levels and pressure readings at the WTP were provided in hour increments from January to October 2020. After excluding outliers, the range of tank levels and WTP hydraulic grade was determined. The model was set up utilizing the minimum observed tank levels and WTP hydraulic grade, as this provides the minimum system pressures and is thus the most conservative assumption. See Table 18 for a summary of the initial settings used in the model.

Table 18 Model Settings

Location	Tank Level Used for Model (ft)	Discharge Pressure Used for Model (psi)	Equivalent Hydraulic Grade ft)
Havre de Grace WTP PS	n/a	107	257.6
Graceview Ground Storage Tank	16.0	n/a	257.0
Lapidum Ground Storage Tank	16.0	n/a	448.0
Bulle Rock Water Tower	23.0	n/a	453.5

For the purposes of the model, it is assumed that one of the smaller two pumps at Graceview Booster PS (Pump 1 or Pump 2) is in operation.

5. Model Analysis / Results

The calibrated distribution system model was updated to include the improvement projects already under design that will be constructed in the immediate future (as detailed in Section 2.6). The Hazen-Williams C value for the new ductile iron pipe was set to 110 in the model. All model analysis was performed on the system after completion of these projects.

5.1 Current System Analysis

5.1.1 Steady State Evaluation

The current distribution system was evaluated under current average day, maximum day, and peak hour demand conditions (as detailed in Section 3.1) at minimum tank levels and WTP hydraulic grade (as detailed in Section 4.5). This analysis gives the anticipated minimum pressure for each water customer in the system. The results of the steady state evaluation are shown in Figure F-1, Figure F-2, and Figure F-3 in Appendix F and a summary of the model results is presented in Table 19.

Table 19 Water Customer Pressures – Current System

Pressure Category	Water Customer Count		
	ADD	MDD	PHD [1]
< 30.0 psi	0	0	0
30.0 to 34.9 psi	0	0	1
35.0 to 39.9 psi	6	11	67
40.0 to 59.9 psi	912	942	968
60.0 to 79.9 psi	1,213	1,201	1,192
80.0 to 99.9 psi	2,378	2,454	2,567
≥ 100.0 psi	779	680	493
Total	5,288	5,288	5,288

Note:

1. PHD pressure is provided for informational purposes. Though there are reductions in system pressure below 35 psi at PHD, this is to be expected and would be localized events during extreme conditions for short periods of time.

Per *Recommended Standards for Water Works, 2012 Edition* (otherwise known as "10 State Standards"), the normal working pressure in a distribution system should be at least 35 psi. This criteria is maintained during average day and max day conditions.

The Havre de Grace WTP PS needs to provide adequate flow to meet maximum day demands in all pressure zones, and the Graceview Booster PS needs to provide adequate flow to meet maximum day demands in pressure zones 2, 2D-I, and 2D-II. As summarized in Table 20, the current pump station capacities are sufficient to meet system demands.

Table 20 Pump Station Capacity Analysis – Current System

Location	Current Capacity (gpm)	Feeds Zones	Required Capacity at MDD (mgd)	Required Capacity at MDD (gpm)
Havre de Grace WTP PS [1]	2,800	1, 2, 2D-I, 2D-II	2.182	1,515
Graceview Booster PS [2]	500	2, 2D-I, 2D-II	0.547	380

Notes:

1. Havre de Grace WTP PS capacity is based on two of the three pumps in operation at full speed.
2. Graceview Booster PS capacity is based on the capacity of the larger Pump 3 run during high demand periods.

5.1.2 Fire Flow Evaluation

Per discussions with the City, each pressure zone must include enough storage for 2 hours of 1,000 gpm fire flow plus the maximum daily demand within the zone. In addition, the water distribution system must be able to supply 1,000 gpm of fire flow at 20 psi residual pressure during maximum day demand (and with tanks at the minimum level of their operating range).

See Table 21 for a summary of current system storage. As shown, sufficient storage is currently provided in Zone 2, but additional storage is required in Zone 1.

Table 21 Storage Analysis – Current System

	Zone 1	Zone 2 [1]
Fire Flow Volume (1,000 gpm for 2 hours) (gal)	120,000	120,000
Maximum Day Demand Volume (gal)	1,635,000	547,000
Required Storage (gal)	1,755,000	667,000
Provided Storage (gal)	1,382,000	829,000
Additional Storage Required (gal)	373,000	-162,000

Note:

1. Zone 2 demands include demands within the drop zones, Zone 2D-I and 2D-II.

A fire flow analysis was conducted using the distribution system model. The analysis determines the fire flow available given certain pressure, velocity, and flow constraints set by the engineer. The automated analysis does not determine the length of time for which the specified flow is available. It only determines the flow available and resulting pressures throughout the model. By examining the results from the analysis, it is possible to determine areas of the distribution system where adequate fire flow is unavailable.

The required minimum pressure was set to 20 psi and a velocity constraint was not used. Nodes on the suction side of pump stations, near the water storage tanks, and immediately downstream of PRVs were excluded from the analysis, since adequate pressure cannot be obtained in these vicinities, and customers and fire hydrants do not connect to the system in these areas. Only junctions connected to fire hydrants were evaluated for available fire flow; however, all junctions (except those mentioned above) were evaluated against the pressure criteria during the fire flow evaluation.

The available fire flow throughout the distribution system is shown in Figure F-4 in Appendix F and a summary is presented in Table 22.

Table 22 Available Fire Flow – Current System

Fire Flow Category	Junction Count				
	Zone 1	Zone 2D-I	Zone 2D-II	Zone 2	Total
< 100.0 gpm	1	0	0	0	1
100.0 to 499.9 gpm	8	0	0	0	8
500.0 to 749.9 gpm	11	4	0	0	15
750.0 to 999.9 gpm	22	9	0	2	33
≥ 1,000.0 gpm	269	12	39	185	505
Total	311	25	39	187	562

As shown, there are a number of areas with insufficient available fire flow.

5.1.3 Recommended Improvements

To meet system storage requirements, the following improvement is recommended:

- Increase storage within Zone 1 by at least 400,000 gallons. For the purposes of this evaluation, the additional storage was added to the model via an elevated storage tank in the vicinity of Old Bay Lane, as this area is hydraulically distant from Graceview Ground Storage Tank and Havre de Grace WTP PS, and hydraulically close to the proposed Aberdeen connection (see Section 5.2). However, additional evaluation, as detailed in Section 6, is required prior to moving forward with any improvements related to system storage.

To increase fire flow to 1,000 gpm at all fire hydrants in Zone 1, the following improvements are recommended:

- Maintain Graceview Tank level above 22 ft and WTP PS discharge pressure above 115 psi.
- St. James Ter – Replace 50 ft of 4" pipe and 370 ft of 6" pipe with 8" pipe.
- Conesteo St – Replace 300 ft of 4" pipe with 8" pipe.
- Franklin St, between N Stokes St and N Union Ave – Replace 810 ft of 4" pipe with 8" pipe.
- Congress Ave, between S Juniata St and S Stokes St – Replace 1,000 ft of 4" pipe with 8" pipe.
- Union Ave, between Pennington Ave and Congress Ave – Replace 430 ft of 4" pipe with 8" pipe.
- Bourbon St, between Strawberry Ln and Market St – Replace 240 ft of 4" pipe with 8" pipe.

- Washington St, between fire hydrant near Fountain St and Girard St – Replace 340 ft of 4" pipe with 8" pipe.
- Revolution St, between S Juniata St and S Freedom Ln – Replace 1,240 ft of 4" pipe with 8" pipe.
- Concord Pl – Replace 550 ft of 6" pipe with 8" pipe.
- Lewis St, near intersection with S Washington St – Replace 100 ft of 4" pipe with 8" pipe.
- S Adams St, between Commerce St and Chesapeake Dr – Replace 260 ft of 2" pipe with 8" pipe.
- Commerce St, between S Adams St and Strawberry Ln – Replace 1,790 ft of 4" pipe with 8" pipe.
- Tydings Memorial Park - Replace 1,060 ft of 6" pipe with 8" pipe.
- Webb Ln – Replace 480 ft of 6" pipe with 8" pipe.
- Vandiver Ct – Replace 350 ft of 6" pipe with 8" pipe.
- Seneca Ave, north of Bloomsbury Ave – Replace 340 ft of 6" pipe with 8" pipe.
- Chesapeake Dr, between Giles St and Concove Way – Replace 2,940 ft of 6" pipe with 8" pipe.
- Havre de Grace WWTP – Replace 1,240 ft of 4" pipe and 150 ft of 6" pipe along Jerry Foster Way with 8" pipe and replace 290 ft of 2" pipe and 1,270 ft of 4" pipe at the WWTP site with 6" pipe.

To increase fire flow to 1,000 gpm at all fire hydrants in Zone 2D-I, the following improvements are recommended:

- Increase set point of the PRV feeding Zone 2D-I to at least 55 psi.
- Add a second PRV between Zone 2 and Zone 2D-I at the intersection of Goforth Dr and Tidewater Dr, replacing the isolation valve there currently.

Pressures and available fire flow in Zone 2D-II are sufficient, and no improvements are recommended.

To increase fire flow to 1,000 gpm at all fire hydrants in Zone 2, the following improvements are recommended:

- Barrett St, Mardin Rd, and Bayview Dr – Replace 2,330 ft of 6" pipe with 8" pipe.

The available fire flow throughout the distribution system after making the above recommended improvements is shown in Figure F-5 in Appendix F and a summary is presented in Table 23.

Table 23 Available Fire Flow – Current System – After Recommended Improvements

Fire Flow Category	Junction Count				
	Zone 1	Zone 2D-I	Zone 2D-II	Zone 2	Total
< 100.0 gpm	0	0	0	0	0
100.0 to 499.9 gpm	0	0	0	0	0

Fire Flow Category	Junction Count				
	Zone 1	Zone 2D-I	Zone 2D-II	Zone 2	Total
500.0 to 749.9 gpm	0	0	0	0	0
750.0 to 999.9 gpm	0	0	0	0	0
≥ 1,000.0 gpm	311	25	39	187	562
Total	311	25	39	187	562

As shown, with the recommended improvements there is at least 1,000 gpm of available fire flow at all fire hydrants.

5.2 Aberdeen Connection

As mentioned in Section 3.2, the City entered into an agreement with the City of Aberdeen to provide Aberdeen with 300,000 to 900,000 gpd of potable water. Water to Aberdeen will be supplied by a new 12" water main, which will connect to the existing City of Havre de Grace distribution system near the intersection of Revolution St and Pulaksi Hwy.

The impact of the Aberdeen connection on the City's water distribution system was evaluated primarily on the current system as outlined in Section 5.1, with only the improvements related to system operation. This includes maintaining Graceview Tank level above 22 ft, maintaining WTP PS discharge pressure above 115 psi, and setting the Zone 2D-I PRV to 55 psi (see Section 5.1.3). The other recommended current system improvements would require construction and will not be able to be completed prior to the Aberdeen connection coming online, anticipated within the next 6 to 12 months.

The impact of the current system after implementing all of the recommended improvements listed in Section 5.1.3 was also evaluated. This represents not only the available fire flow anticipated after the Aberdeen demand is added and the current system improvements are implemented, but also indicates the relative impact of the Aberdeen demand on the system by excluding the portions already with insufficient fire flow during current conditions.

5.2.1 Steady State Evaluation

The current distribution system with current demands and 900,000 gpd of demand at the Aberdeen connection point was evaluated under average day, maximum day, and peak hour demand conditions. The results of the steady state evaluation are shown in Figure G-1, Figure G-2, and Figure G-3 in Appendix G and a summary of the model results is presented in Table 24.

Table 24 Water Customer Pressures – Current System with Aberdeen Connection

Pressure Category	Water Customer Count		
	ADD	MDD	PHD [1]
< 30.0 psi	0	0	0
30.0 to 34.9 psi	0	0	18
35.0 to 39.9 psi	0	9	80
40.0 to 59.9 psi	849	897	1,039
60.0 to 79.9 psi	1,198	1,172	1,125

Pressure Category	Water Customer Count		
	ADD	MDD	PHD [1]
80.0 to 99.9 psi	2,030	2,282	2,436
≥ 100.0 psi	1,211	928	590
Total	5,288	5,288	5,288

Note:

1. PHD pressure is provided for informational purposes. Though there are reductions in system pressure below 35 psi at PHD, this is to be expected and would be localized events during extreme conditions for short periods of time.

The addition of the Aberdeen demand results in a slight pressure drop for some customers, but pressures for all water customers remains above 35 psi during ADD and MDD conditions.

The results of the steady state evaluation including the Aberdeen demand, on the current system with the improvements listed in Section 5.1.3, is presented in Table 25.

Table 25 Water Customer Pressures – Current System (Improved) with Aberdeen Connection

Pressure Category	Water Customer Count		
	ADD	MDD	PHD [1]
< 30.0 psi	0	0	0
30.0 to 34.9 psi	0	0	14
35.0 to 39.9 psi	0	5	64
40.0 to 59.9 psi	839	888	1,025
60.0 to 79.9 psi	1,206	1,181	1,099
80.0 to 99.9 psi	1,999	2,251	2,475
≥ 100.0 psi	1,244	963	611
Total	5,288	5,288	5,288

Note:

1. PHD pressure is provided for informational purposes. Though there are reductions in system pressure below 35 psi at PHD, this is to be expected and would be localized events during extreme conditions for short periods of time.

As shown, pressures at existing water customers remain above 35 psi during ADD and MDD conditions.

As summarized in Table 26, the current Havre de Grace WTP PS is sufficient to meet current maximum day demand with the Aberdeen connection demand.

Table 26 Pump Station Capacity Analysis – with Aberdeen Connection

Location	Current Capacity (gpm)	Feeds Zones	Required Capacity at MDD (mgd)	Required Capacity at MDD (gpm)
Havre de Grace WTP PS [1]	2,800	1, 2, 2D-I, 2D-II	3.082	2,140

Location	Current Capacity (gpm)	Feeds Zones	Required Capacity at MDD (mgd)	Required Capacity at MDD (gpm)
Graceview Booster PS [2]	500	2, 2D-I, 2D-II	0.547	380

Notes:

1. Havre de Grace WTP PS capacity is based on two of the three pumps in operation at full speed.
2. Graceview Booster PS capacity is based on the capacity of the larger Pump 3 run during high demand periods.

5.2.2 Fire Flow Evaluation

Storage for the Aberdeen connection demand does not need to be provided by the City; therefore, the storage evaluation does not change from the current system analysis (see Section 5.1.2).

The available fire flow throughout the current distribution system during maximum day demand with the Aberdeen connection demand is shown in Figure G-4 in Appendix G and a summary is presented in Table 27.

Table 27 Available Fire Flow – Current System with Aberdeen Connection

Fire Flow Category	Junction Count				
	Zone 1	Zone 2D-I	Zone 2D-II	Zone 2	Total
< 100.0 gpm	1	0	0	0	1
100.0 to 499.9 gpm	8	0	0	0	8
500.0 to 749.9 gpm	12	3	0	0	15
750.0 to 999.9 gpm	74	8	0	2	84
≥ 1,000.0 gpm	216	14	39	185	454
Total	311	25	39	187	562

The hydrants with significant fire flow deficiencies (available fire flow of less than 500 gpm) were not impacted significantly by the addition of the Aberdeen demand. The Aberdeen demand does result in one additional hydrant having fire flow slightly less than 750 gpm and 53 additional hydrants having fire flow less than 1,000 gpm.

The available fire flow throughout the current distribution system during maximum day demand with the Aberdeen connection demand, after implementing the improvements listed in Section 5.1.3, is shown in Figure G-5 in Appendix G and a summary is presented in Table 28.

Table 28 Available Fire Flow – Current System (Improved) with Aberdeen Connection

Fire Flow Category	Junction Count				
	Zone 1	Zone 2D-I	Zone 2D-II	Zone 2	Total
< 100.0 gpm	0	0	0	0	0
100.0 to 499.9 gpm	0	0	0	0	0
500.0 to 749.9 gpm	0	0	0	0	0
750.0 to 999.9 gpm	0	0	0	0	0
≥ 1,000.0 gpm	311	25	39	187	562
Total	311	25	39	187	562

As shown, with the current recommended improvements (including providing storage in the vicinity of Old Bay Lane) and the Aberdeen demand, there is at least 1,000 gpm of available fire flow at all fire hydrants.

5.2.3 Recommended Improvements

Although addition of the Aberdeen Connection demand decreases available fire flow within Zone 1, available fire flow remains above 745 gpm for all hydrants not already below 1,000 gpm, and customer pressures remain above 35 psi during MDD. Therefore, while not ideal, there is not anticipated to be significant impacts to customers if no improvements are made prior to supplying water to Aberdeen. Implementing the improvements recommended in Section 5.1.3 (including providing storage in the vicinity of Old Bay Lane) is shown by the model to increase available fire flow for all hydrants to at least 1,000 gpm.

5.3 Future System Analysis

As mentioned in Section 3.3, there are a number of areas where additional demand is projected over the next 20 years. The distribution system was evaluated with all future demands to determine what improvements to the existing system would be required. The evaluation only looked at the existing system and does not include new piping or pump stations to supply water to future demand areas.

For the purposes of the future evaluation, it is assumed that all recommended improvements from the current system analysis (see Section 5.1.3) have been implemented.

5.3.1 Steady State Evaluation

The current distribution system with all projected future demands was evaluated under average day, maximum day, and peak hour demand conditions. The results of the steady state evaluation are shown in Figure H-1, Figure H-2, and Figure H-3 in Appendix H and a summary of the model results is presented in Table 29.

Table 29 Water Customer Pressures – Future Buildout Demands

Pressure Category	Water Customer Count		
	ADD	MDD	PHD [1]
< 30.0 psi	0	0	0
30.0 to 34.9 psi	0	0	0
35.0 to 39.9 psi	0	0	2
40.0 to 59.9 psi	798	824	898
60.0 to 79.9 psi	1,236	1,216	1,179
80.0 to 99.9 psi	1,689	1,975	2,334
≥ 100.0 psi	1,565	1,273	875
Total	5,288	5,288	5,288

Note:

1. PHD pressure is provided for informational purposes. Though there are reductions in system pressure below 35 psi at PHD, this is to be expected and would be localized events during extreme conditions for short periods of time.

As shown, pressures at existing water customers remain above 35 psi during ADD and MDD conditions with the addition of all projected future demands.

As summarized in Table 30, with the additional projected future demands, the capacity of the Havre de Grace WTP PS and the Graceview Booster PS must be increased to meet demand.

Table 30 Pump Station Capacity Analysis – Future Buildout Demands

Location	Current Capacity (gpm)	Feeds Zones	Required Capacity at MDD (mgd)	Required Capacity at MDD (gpm)
Havre de Grace WTP PS [1]	2,800	1, 2, 2D-I, 2D-II	4.490	3,118
Graceview Booster PS [2]	500	2, 2D-I, 2D-II	1.457	1,012

Notes:

1. Havre de Grace WTP PS capacity is based on two of the three pumps in operation at full speed.
2. Graceview Booster PS capacity is based on the capacity of the larger Pump 3 run during high demand periods.

5.3.2 Fire Flow Evaluation

See Table 31 for a summary of system storage with future demands. As shown, with the additional projected future demands, additional storage must be provided in Zone 1 and Zone 2.

Table 31 Storage Analysis – Future Buildout Demands

	Zone 1	Zone 2 [1]
Fire Flow Volume (1,000 gpm for 2 hours) (gal)	120,000	120,000
Maximum Day Demand Volume (gal)	2,133,000	1,457,000
Required Storage (gal)	2,253,000	1,577,000
Provided Storage (gal)	1,382,000	829,000
Recommended Additional Storage (Current Improvements) (gal) [2]	400,000	-
Additional Storage Required (Future) (gal)	471,000	748,000

Notes:

1. Zone 2 demands include demands within the drop zones, Zone 2D-I and 2D-II.
2. See Section 5.1.3 for improvements recommended on the current system.

The available fire flow throughout the distribution system during future maximum day demand is shown in Figure H-4 in Appendix H and a summary is presented in Table 32.

Table 32 Available Fire Flow – Future Buildout Demands

Fire Flow Category	Junction Count				
	Zone 1	Zone 2D-I	Zone 2D-II	Zone 2	Total
< 100.0 gpm	0	0	0	0	0
100.0 to 499.9 gpm	0	0	0	0	0
500.0 to 749.9 gpm	0	0	0	0	0
750.0 to 999.9 gpm	0	0	0	0	0
≥ 1,000.0 gpm	311	25	39	187	562
Total	311	25	39	187	562

As shown, with the addition of the future demands, there is still at least 1,000 gpm of available fire flow at all fire hydrants

5.3.3 Recommended Improvements

The following system improvements are recommended to meet projected future demands:

- Increase capacity of the Havre de Grace WTP PS to at least 3,120 gpm. This can be achieved if the capacity of all finished water pumps is increased to 1,560 gpm at full speed.
- Increase capacity of the Graceview Booster PS to at least 1,020 gpm. This can be achieved either by having two pumps, each rated for 1,020 gpm, or by having one pump with a capacity of 1,020 gpm and two pumps each with a capacity of 510 gpm.
- Increase storage within Zone 1 by an additional 500,000 gallons (in addition to the 400,000 recommended in Section 5.1.3). For the purposes of this evaluation, the additional storage was added to the model via an elevated storage tank in the vicinity of Old Bay Lane, as the majority of future demands within Zone 1 are in this area. However, additional evaluation, as detailed in Section 6, is required prior to moving forward with any improvements related to system storage.
- Increase storage within Zone 2 by 750,000 gallons. Given the high elevation of Green-Ianniello-Patrone Properties, MOE, and Susquehanna Hills, a new pump station and pressure zone will need to be included with the development of these properties. It is recommended that the additional storage be provided by a new 750,000 gallon elevated storage tank in the new, higher pressure zone. However, additional evaluation, as detailed in Section 6, is required prior to moving forward with any improvements related to system storage.

The available fire flow throughout the distribution system after making the above recommended improvements is shown in Figure H-5 in Appendix H and a summary is presented in Table 33.

Table 33 Available Fire Flow – Future Buildout Demands – After Recommended Improvements

Fire Flow Category	Junction Count				
	Zone 1	Zone 2D-I	Zone 2D-II	Zone 2	Total
< 100.0 gpm	0	0	0	0	0
100.0 to 499.9 gpm	0	0	0	0	0
500.0 to 749.9 gpm	0	0	0	0	0
750.0 to 999.9 gpm	0	0	0	0	0
≥ 1,000.0 gpm	311	25	39	187	562
Total	311	25	39	187	562

As shown, with the recommended improvements is the model indicates at least 1,000 gpm of available fire flow at all fire hydrants.

5.3.3.1 Recommended Improvement Timing

The majority of future demand within Zone 1 is due to Greenway Farms Phase 2 & 3 and Old Bay Lane Industrial Redevelopment. Zone 1 improvements should be completed prior to the addition of these future demands, projected within the next five years. Systemwide, 0.950 MGD of additional demand can be added before improvements are required at the Havre de Grace WTP PS. Assuming 250 GPD per EDU, this equates to 3,800 EDUs

The majority of future demand within Zone 2 is due to Green-Ianniello-Patrone Properties and MOE. Zone 2 improvements should be completed prior to the addition of these future demands, projected within the next five years. Throughout Zone 2, Zone 2D-I, and Zone 2D-II, a total of 0.173 MGD of additional demand can be added before improvements are required at the Graceview Booster PS, and a total of 0.162 MGD of additional demand can be added before additional Zone 2 storage is required. Assuming 250 GPD per EDU, this equates to 692 and 648 EDUs respectively.

6. Additional Evaluation

Additional evaluation of the system is recommended prior to making any of the significant modifications to pump stations and system storage outlined above. It is recommended that the additional evaluation include extended period simulation (EPS) of the system to model draw and fill cycles of the tanks and see how the tanks hydraulically interact with each other during typical system operation. An EPS evaluation will also allow for investigation of the impact of the improvements on water age and water quality. The developed and calibrated steady state model detailed in the report would serve as the basis for an EPS evaluation, and the system geometry, pipe characteristics, and demand distribution would not need to be updated. To perform an EPS evaluation, the model would need to be updated with diurnal curves for all demands and control logic to determine when pumps turn on and off (or when VFDs adjust speed). Additional calibration would be required to match historic tank behavior.

Condition assessments on all three of the City's existing storage tanks have indicated that these tanks need to be rehabilitated or replaced in the near future due to age and condition. A detailed storage evaluation utilizing the EPS model is recommended to determine an approach to provide the required storage in each pressure zone that is hydraulically optimal, cost effective, and reliable. Possible alternatives to be investigated include rehabilitating or replacing the existing tanks in-kind and adding additional storage elsewhere, replacing the existing tanks with larger tanks to cover all required storage, or some combination thereof. Such an evaluation would also include a more detailed siting study, taking into account current land ownership, potential for City to purchase new land, etc.

7. Opinion of Probable Construction Costs

The opinion of probable construction cost for the recommended hydraulic improvements is presented in Table 34. Current system improvements, as detailed in Sections 5.1.3, consist of pipe improvements to improve fire flow within the existing system with current demands, and cost to add 400,000 gal of additional storage to Zone 1. Future system improvements, as detailed in Section 5.3.3, consist of pump station improvements and increased system storage to meet all future demands. Costs presented are just for the hydraulic improvements identified in this report and do not include rehabilitation / replacement of existing components due to age or condition. A detailed cost estimate is provided in Appendix I.

Table 34 Opinion of Probable Cost

	Current System Improvements [1]	Future System Improvements
Construction Cost	$6,436,000	$7,628,000
Contingency (30%)	$1,931,000	$2,289,000
Probable Construction Cost	$8,367,000	$9,917,000
Admin., Eng., and Legal (30%)	$2,511,000	$2,976,000
Probable Project Cost	$10,878,000	$12,893,000

Note:

1. Costs are just for hydraulic improvements recommended in this report and do not include rehabilitation / replacement of existing components due to age or condition.

The above cost estimate is approximate and meant for planning-level purposes only. Although calculated as combined projects for each time period, it is likely that improvements would be broken up into multiple, smaller projects. The cost to provide additional storage is based on assumptions regarding tank sizes, but as discussed in Section 6, the storage requirements are subject to further evaluation (for example, two smaller tanks could be provided in lieu of one larger tank, or one larger tank could be provided to meet current and future storage requirements for a zone).

As discussed in Section 6, it is recommended that the hydraulic improvements recommended in this report be coordinated with other anticipated system improvements to ensure the most cost-effective solution is achieved.

8. Summary and Conclusion

Based on historic demands and a hydraulic model calibrated from the hydrant flow testing performed in the field, there are a number of areas of the existing system where sufficient fire flow is not currently available. These areas should be investigated further and system improvements implemented as soon as possible. Additional storage is required within Zone 1 to provide adequate storage based on current system demands and required fire flow volume.

With the addition of the Aberdeen Connection demand of 900,000 gpd, available fire flow is reduced within portions of Zone 1. However, available fire flow remains above 745 gpm for all hydrants not already below 1,000 gpm, and customer pressures remain above 35 psi during MDD. The model indicates that providing additional storage in the vicinity of Old Bay Ln (in addition to recommended pipe improvements to the current system) would improve available fire flow to 1,000 gpm throughout the system after connection of the Aberdeen demand. Additional evaluation of the system, including EPS modeling, is recommended prior to design of any additional system storage to determine the impacts on the distribution system.

To supply the projected future demands, the capacity of both the Havre de Grace WTP PS and Graceview Booster PS will need to be increased. Additional storage will also be required in both Zone 1 and Zone 2. These improvements are anticipated to be required within the next 5 years, based on the projected time frame of the future demands. Additional evaluation of the system, including EPS modeling, is recommended prior to design of these improvements to determine the impacts on the distribution system.

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