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Drinking Water Infrastructure Plan – Crozet Area

Final Report
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List of Acronyms

Abbreviation	Definition
AC	asbestos cement
ACSA	Albemarle County Service Authority
ADD	average day demand
DI	ductile iron
DWIP	Drinking Water Infrastructure Plan
EBCT	empty bed contact time
GAC	granular activated carbon
GIS	Geographic information system
gpcd	gallons per capita per day
gped	gallons per employee per day
JPA	Joint Permit Application
MDD	maximum day demand
MIF	minimum instream flow
MG	million gallons
mgd	million gallons per day
NOM	natural organic matter
PAC	powdered activated carbon
PER	preliminary engineering report
RDU	residential dwelling unit
RWSA	Rivanna Water and Sewer Authority
SCADA	Supervisory Control and Data Acquisition
VADEQ	Virginia Department of Environmental Quality
VDH	Virginia Department of Health
VDOT	Virginia Department of Transportation
VFD	variable frequency drive
VWP	Virginia Water Protection
WTP	water treatment plant

Introduction and Purpose

This Drinking Water Infrastructure Plan was prepared to define the path for implementation of water supply, treatment and distribution improvements necessary to meet the finished water demands in the Crozet Service Area for the next 50 years.

The Community of Crozet, Virginia is a small residential community west of Charlottesville near the foothills of the Blue Ridge Mountains in Albemarle County. The community and portions of the surrounding area (hereinafter referred to as the Crozet Service Area) are provided with finished drinking water by the Rivanna Water and Sewer Authority (RWSA). Portions of the water distribution system are managed by RWSA with the remainder operated by the Albemarle County Service Authority (ACSA). The current permitted maximum day production of finished drinking water for the Crozet Service Area is 1 million gallons per day (mgd).

Over the past ten years, RWSA has seen an increase in annual finished water production at the Crozet Water Treatment Plant (WTP), from approximately 124 million gallons in 2007 to nearly 188 million gallons in 2016. As demands for finished drinking water have increased, the WTP operations staff have had several days where plant production approached the 1 mgd permit limit. While this is attributable to both finished water demands and distribution system operating procedures (i.e., tank filling procedures), this upward trend suggested a need for RWSA to undertake an assessment of future finished water demands and the ability of the existing infrastructure to meet those demands.

This Drinking Water Infrastructure Plan (DWIP) for the Crozet Service Area was developed to help RWSA answer the question of projected demands and infrastructure capacity. The foundation of the plan is a series of finished water demand projections developed based on historical planning documents, current zoning, finished water production records and billing data from ACSA. These projections identified the amount of finished drinking water needed in the Crozet Service Area through the year 2075, broken down into five-year increments. Using these production projections, the existing water supply, treatment and distribution infrastructure was studied to identify factors that limit the ability of the infrastructure to meet the projections and to develop potential solutions to address those limitations. The ultimate outcome of the DWIP is a “road map” that outlines the phasing of required infrastructure improvements needed to meet the incremental increases in demand with associated capital and operating cost estimates.

1. Crozet Area Overview

1.1 General Description of the Service Area

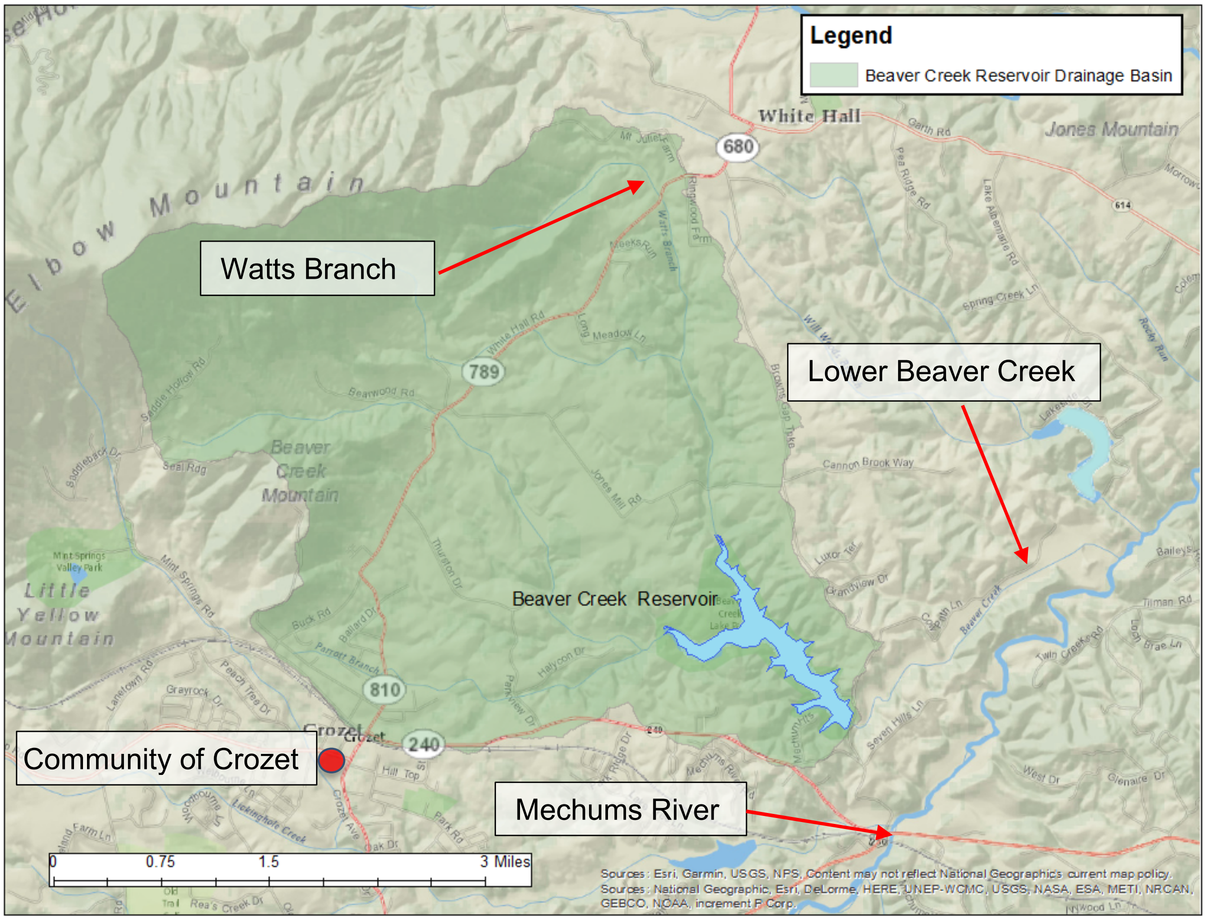
The Community of Crozet, Virginia lies approximately 15 miles to the west of Charlottesville in Albemarle County on the Atlantic slope of the Blue Ridge Mountains. Its scenic surroundings and proximity to Charlottesville have contributed to rapid growth since the turn of the millennium. The County's rural area policies designed to preserve the scenic nature of the County have also driven growth toward Crozet and away from surrounding rural areas. The community's estimated population in 2000 was 2,753, and presently it is approaching 7,000. The community's drinking water is treated at the Crozet WTP, which is located in the eastern part of the community and receives its raw water source from the Beaver Creek Reservoir. The current average day demand (ADD) for the Crozet Service Area is approximately 0.5 mgd.

1.1.1 Beaver Creek Dam and Reservoir

The Beaver Creek Reservoir was constructed in 1963 as a Soil Conservation Service (now the Natural Resource Conservation Service) project to provide water supply storage, flood storage, and sediment storage for the Crozet area. The watershed area draining to the reservoir is 9.39 mi^2 not including the reservoir's surface. It impounds about 568 million gallons (MG) of water below the principal spillway elevation of 538.7 feet above sea level and covers an area of about 102 acres based on the June 2017 bathymetric survey. The top of the dam is at 559.4 feet and is traversed by State Highway 680, also known as Brown's Gap Turnpike (Figure 1-1). Water released from the dam flows into the lower Beaver Creek, which converges with the Mechums River 1.8 river miles downstream of the dam. Figure 1-2 shows the location of the reservoir and drainage basin relative to the Comunity of Crozet and the Mechums River.

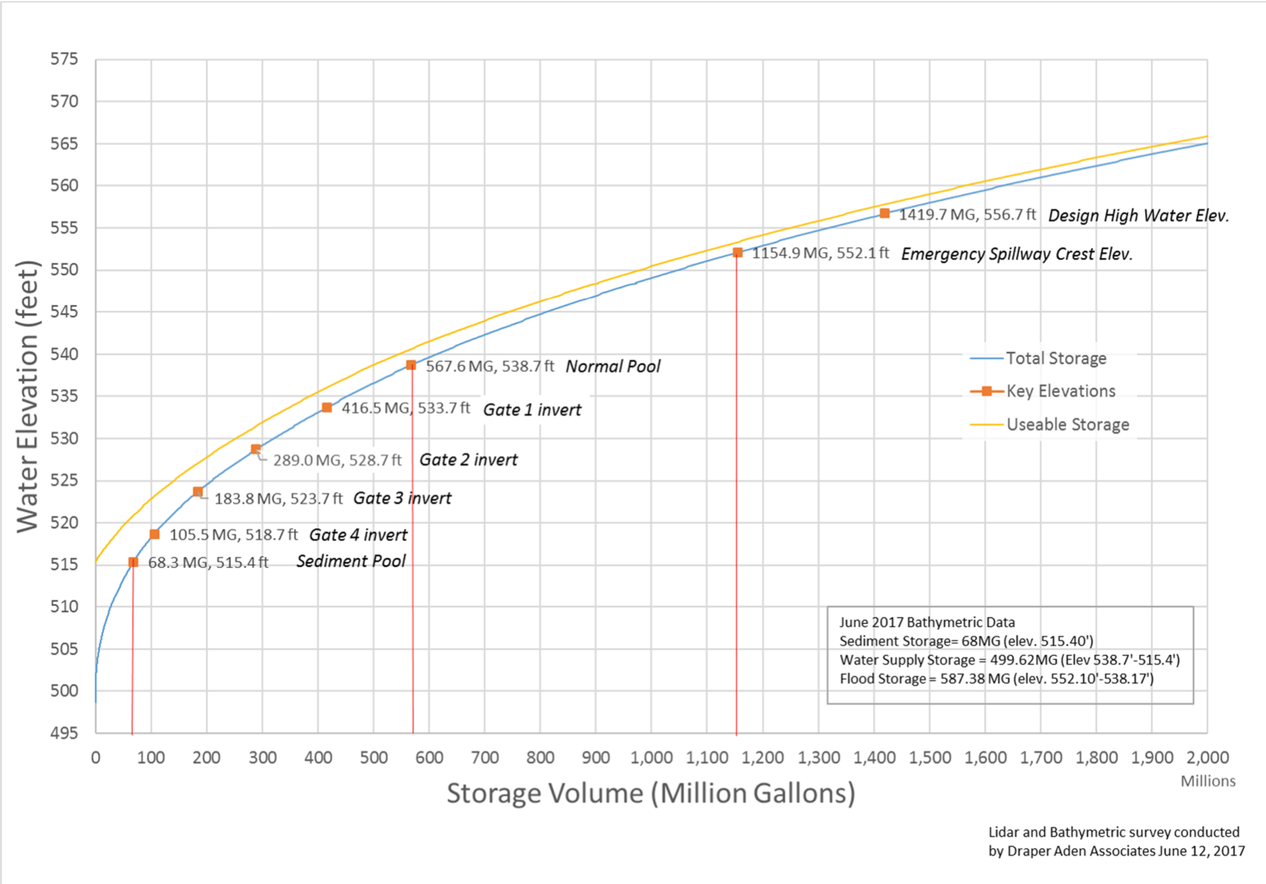
Figure 1-1: Photo of Beaver Creek Dam and Reservoir from Brown's Gap turnpike looking south

Figure 1-2: Area Map showing location of Beaver Creek Reservoir



Storage and elevation data for the reservoir are described in Figure 1-3. Key elevations are noted on the figure, including elevation of sluice gates and the principal and emergency spillways. Comparison of the original storage profile developed at the time of construction with the June 2017 bathymetric survey of the Beaver Creek Reservoir indicates that about 19.5 MG (3.3%) of storage has been lost due to sedimentation since the dam's impoundment in 1963. However, according to the same 2017 survey, the available storage below 515.4 feet is estimated at 68.3 MG, which is a net increase of 2.5 MG in the sediment storage zone. Thus, the water supply pool has a current estimated storage volume of 499 MG, or 22 MG less than the estimate at the time of impoundment and 18 MG less than the estimated storage at normal pool elevation from 1977. Nevertheless, this is a relatively slow accumulation of sediment compared to many reservoirs of similar age and is not a major cause for concern. Some of the storage loss/discrepancies may also be due to differences in reservoir survey methods.

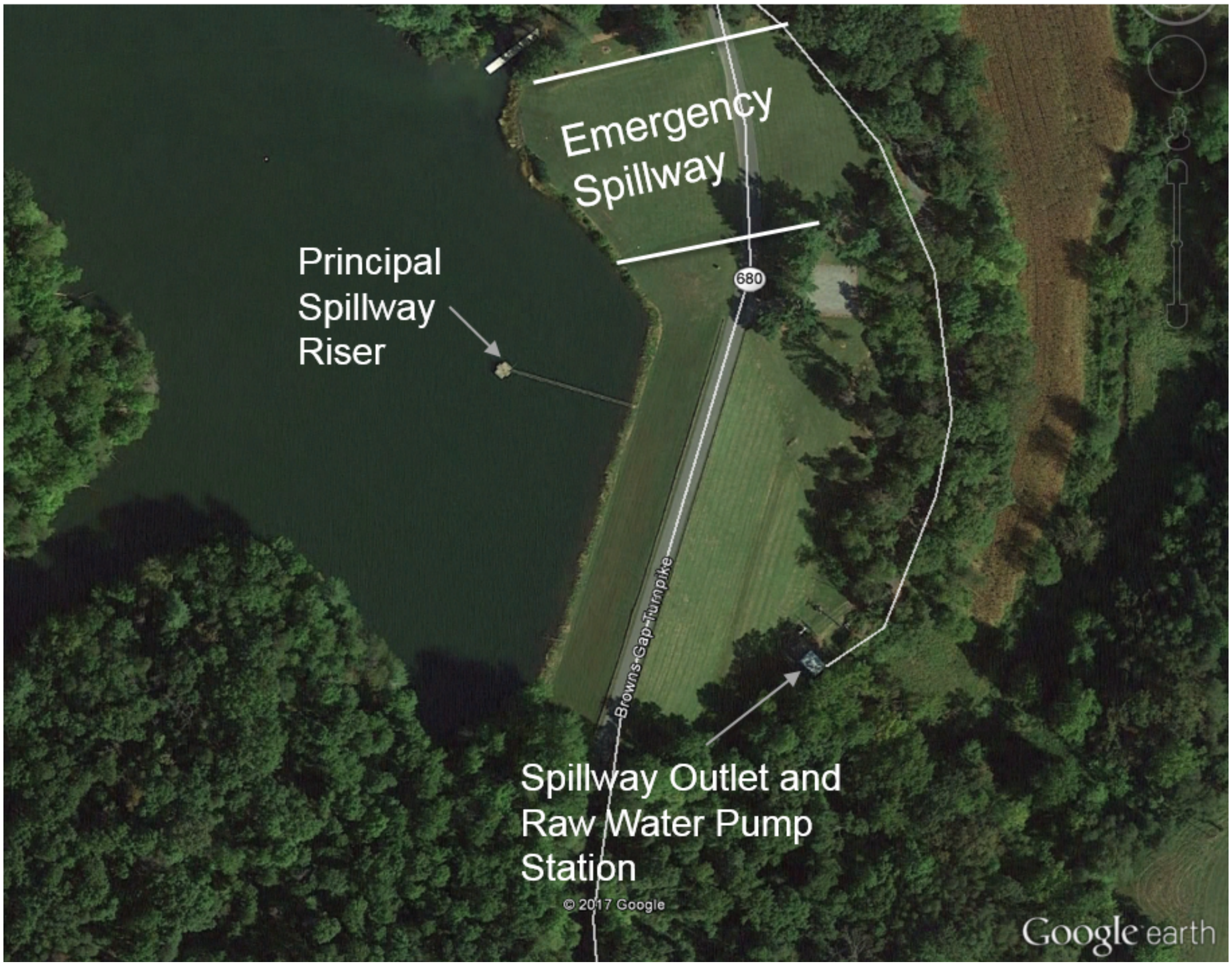
Figure 1-3: Stage – Storage Information for Beaver Creek Reservoir



RWSA currently has a Virginia Department of Health (VDH) permit for 1 mgd of treatment capacity at the Crozet WTP. The reservoir has had “excluded” status in the VADEQ VWP permit classification system. The “excluded” status allows the treatment and production of 1.0 mgd, and an associated reservoir withdrawal marginally greater than 1.0 mgd to account for plant use water, without the need for a VWP permit.

Water is released from the dam through the spillway riser (at left in Figure 1-1 and Figure 1-4) and is connected to the shore of the reservoir via a narrow “catwalk” bridge.

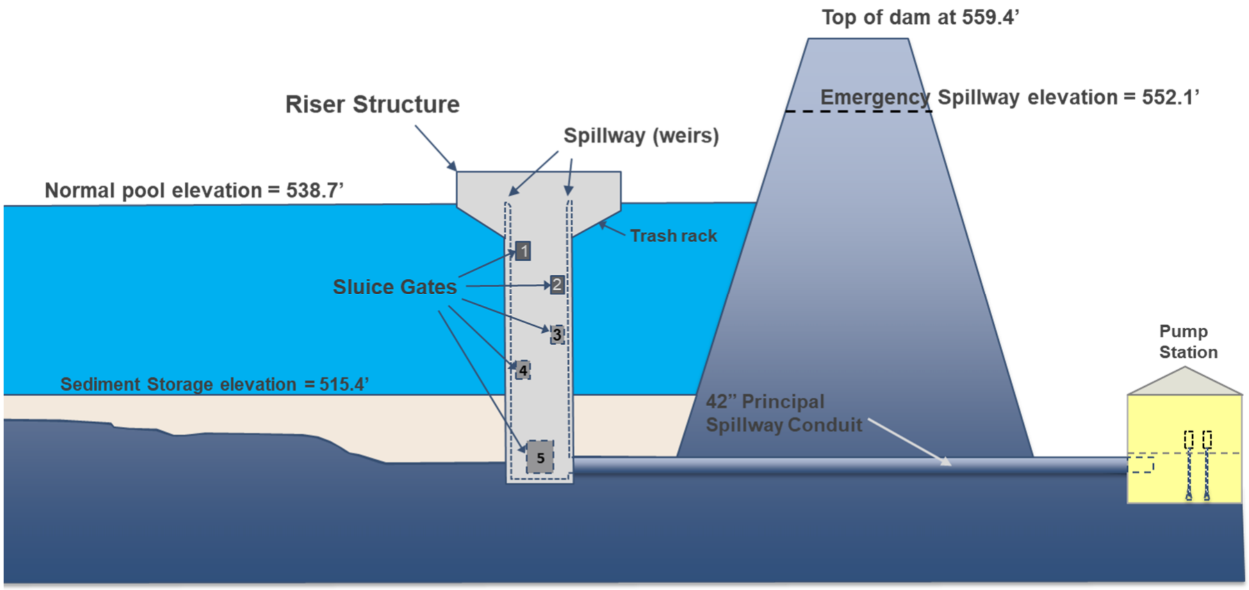
Figure 1-4: Aerial view of Beaver Creek Reservoir and Dam



The riser structure (Figures 1-4 and 1-5) serves as the principal spillway for the dam. It also includes four 10-inch by 10-inch sluice gates which are manually operated to provide multi-level withdrawal capabilities from the reservoir, and a 36-inch reservoir drain gate located at the bottom of the tower. The project record drawings show that the spillway consists of two 9-foot weirs near the top of the riser structure[1]. Large debris is excluded from flowing over the spillway by a set of trash racks. When the reservoir level falls below normal pool elevation, RWSA operates one of the four 10-inch sluice gates to keep water flowing to the pump station. As shown in Figures 1-3 and 1-5, these gates, which are numbered 1 through 4, have invert elevations of 5 feet, 10 feet, 15 feet, and 20 feet below the normal pool level, respectively. The invert elevation of gate 4 is at 518.7 feet, which corresponds to 3.3 feet above the designated sediment storage level. This leaves about 37 million gallons of water supply storage, or about 7.5% of the designated water supply pool capacity, inaccessible under the current infrastructure arrangement.

[1] As field verified in November 2017, in apparent conflict with the original SCS design notes, which indicate two 10.5-foot weirs.

Figure 1-5: Schematic of Beaver Creek Dam and Spillway Riser



1.1.2 Raw Water Pump Station

The raw water pump station is located at the end of the principal spillway conduit at the foot of the dam as shown in Figure 1-5 above and in Figures 1-6 and 1-7 on the following page. The pump station houses two 1 mgd vertical turbine pumps, which operate in a duty/stand-by mode and are furnished with back-up power generation. Raw water enters the pump station wet well at the spillway outlet chamber as shown in Figure 1-7. A weir at the downstream end of the chamber maintains at least a 4-foot depth in the wet well as long as there is sufficient flow into the chamber to supply the pumps. There is a flow meter near the discharge of the spillway conduit, which is tied to the SCADA system. Two 12-inch gated and screened conduits convey water from the spillway outlet chamber into the adjacent pump station wet well. The screens are equipped with an air-burst cleaning system.

When the reservoir level is below full pool, maximizing storage utilization is a challenge owing to the need to manually adjust the sluice gates on the outlet riser to release the precise amount of flow needed at the pump station. The pumps are not on VFDs and therefore operate at full speed when on. As such, neither the rate of release from the spillway riser nor the pumping rate to the WTP can be adjusted remotely by staff. Any additional flow released at the spillway riser in addition to the rate needed to supply the raw water pumps is released to the lower Beaver Creek as shown in Figure 1-7. Because releasing too little water from the riser causes the wet well to dewater and the pumps to run dry (or incur a float switch induced shutdown), staff tend to release more water than that needed to precisely maintain pump suction submergence.

Figure 1-6: Photo of Existing Raw Water Pump Station at foot of Beaver Creek Dam

Figure 1-7: Profile Schematic of Spillway Outlet and Raw Water Pump Station

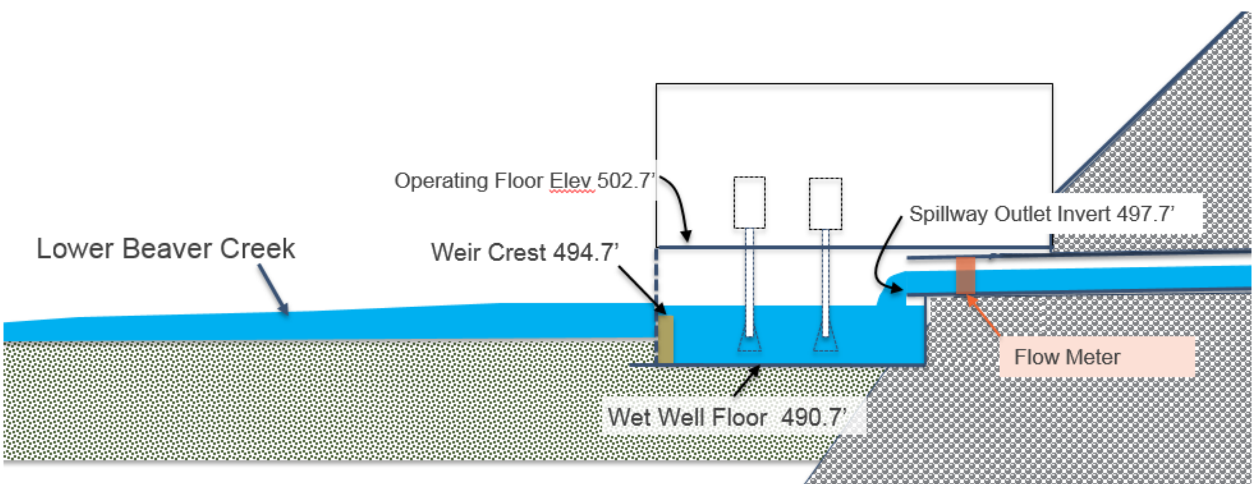
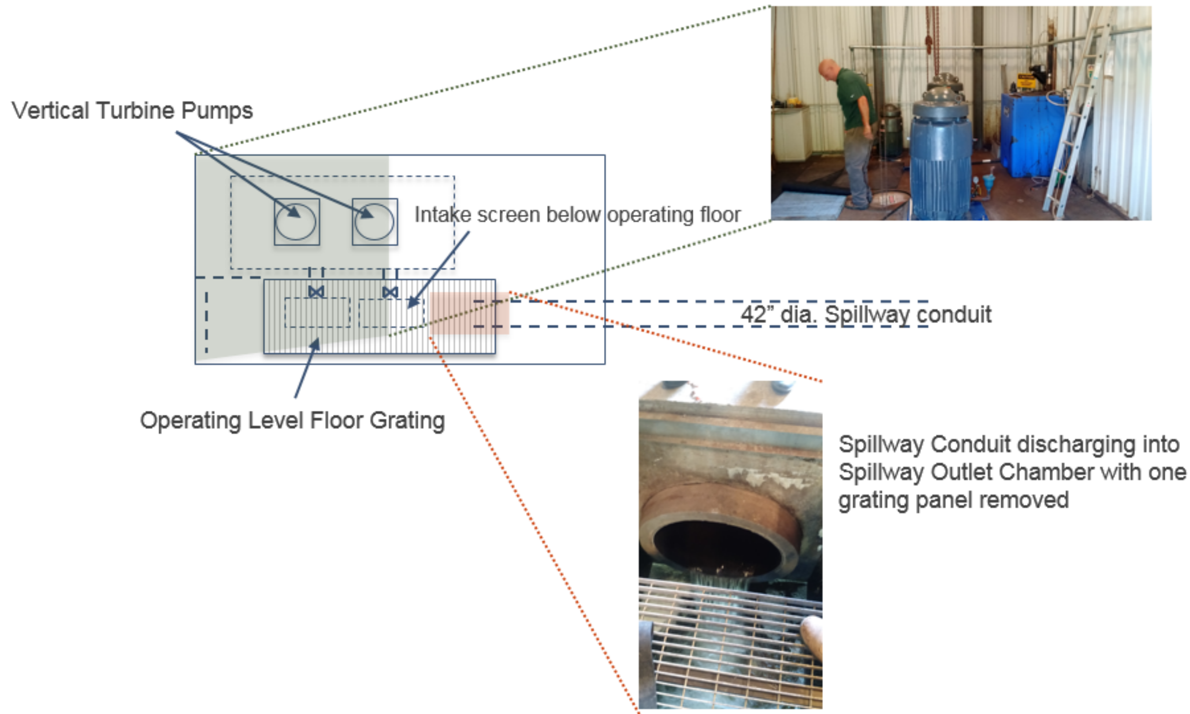


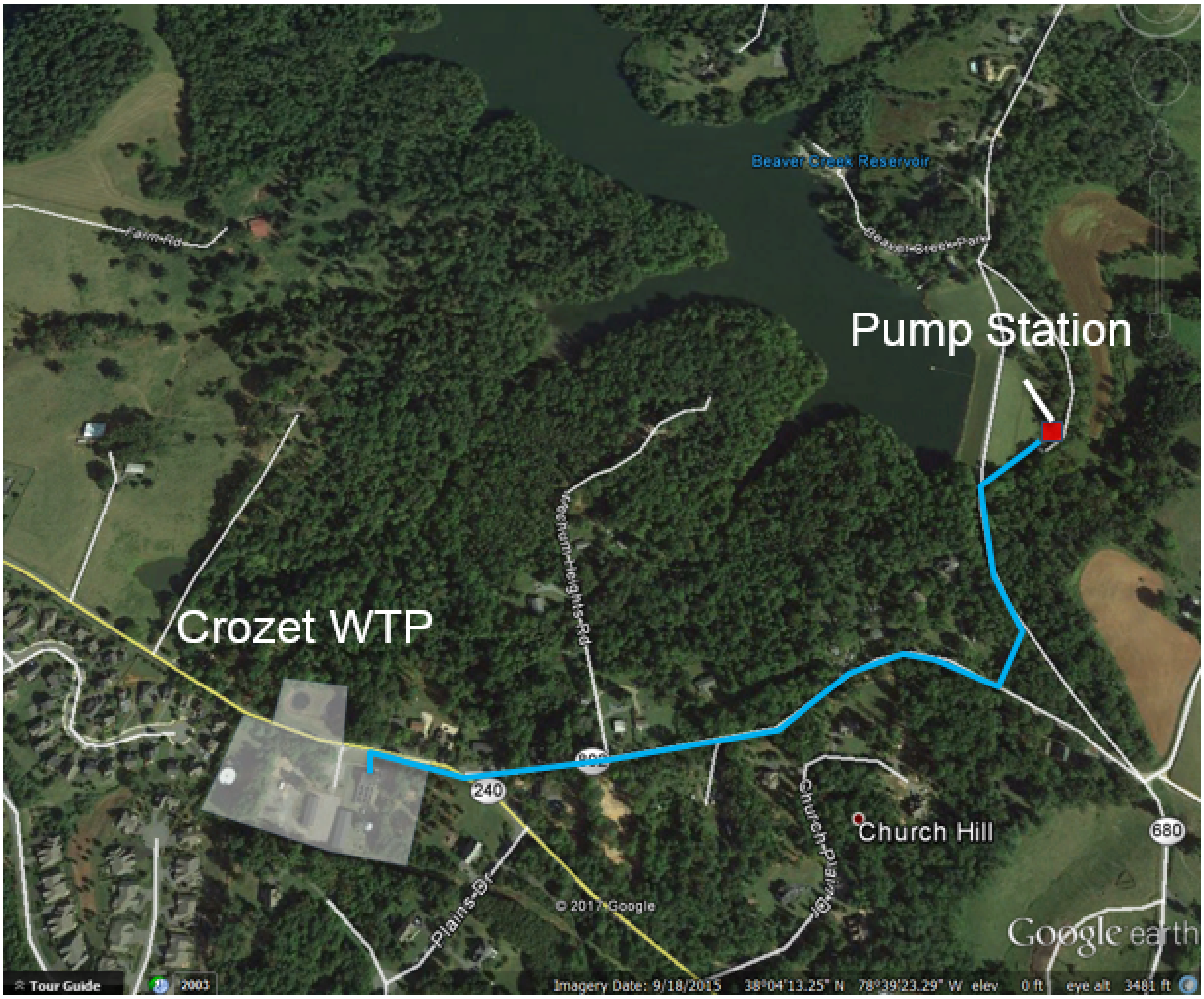
Figure 1-8: Plan View Schematic of Raw Water Pump Station with Photographic Aid



1.1.3 Raw Water Transmission Line

Raw water is conveyed to the Crozet WTP via a 12-inch asbestos cement pipeline. Installation of the pipeline dates to 1964 and follows the alignment of the blue line shown in Figure 1-9. The pipeline route generally follows highway 680 (Brown's Gap Turnpike) to highway 802 (Old Three Notched Road) to highway 240 (Three Notched Road) where it enters the Crozet WTP. The total length is approximately ¾ of a mile. Beginning at the pump station and for the first several hundred feet in the vicinity of the dam, the pipeline follows an old routing of highway 680 (Brown's Gap Turnpike) that was realigned sometime after the completion of the dam. The pipeline routing with the old location of highway 680 is shown in the drawing set for ACSA Project 64-3, Water Facility Improvements, dated March 20, 1964 by Langley and McDonald Consulting Engineers.

Figure 1-9: Raw Water Pipeline Alignment

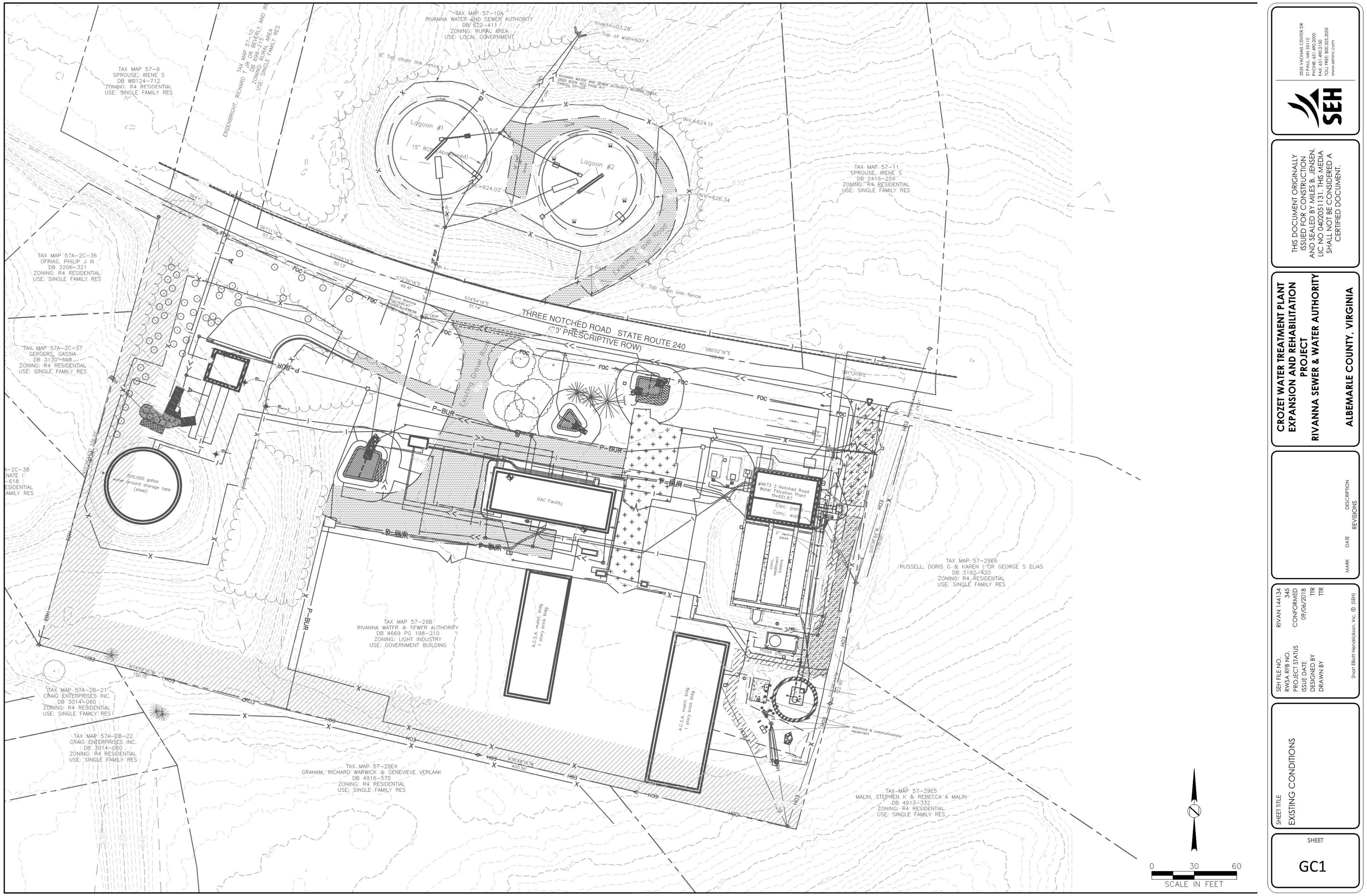


1.2 Water Treatment Facilities

The Crozet WTP was constructed in 1966 with a permitted treatment capacity of 1.0 mgd. The WTP typically produces 529,000 gallons per day on average, operating for an average of 14 hours a day, seven days a week. Due to demand characteristics, and system storage, intermittent operations provide for cost-effective staffing of the facility and ability to meet peak water demands.

The Crozet WTP treats raw water from the Beaver Creek Reservoir using conventional flocculation, sedimentation and filtration with post-filter granular activated carbon (GAC) adsorption treatment. The plant is located adjacent to Three Notched Road as shown above in Figure 1-9. Figure 1-10 on the following page provides an overall plan view of the WTP site and associated facilities.

Figure 1-10: Crozet WTP Site Plan (courtesy of SEH)



Raw water is pumped from the Beaver Creek Reservoir to the raw water channel where pre-treatment chemicals are added (alum, soda ash and PAC) prior to rapid mixing (overflow elevation of 665.50 ft, NAD83 datum). Coagulated water is conveyed from the rapid mix basin through a pipe to a two-stage flocculation basin. Flocculated water flows through a common channel to two sedimentation basins, which are equipped with a Claritrac residuals removal system. Two dual-media filters, currently permitted at 2 gpm/sf, provide final particle removal prior to GAC adsorption and disinfection.

Filtered water typically flows to the intermediate pump station at the new GAC facility; however, the site piping is configured so the GAC facility can be completely bypassed. The GAC facility includes one vertical turbine (can-type) pump with space for a redundant pump to be installed in the future. Two pressure vessels with a capacity for 20,000 pounds of granular activated carbon provide an empty bed contact time (EBCT) of 15.4 minutes at the current flow of 1.0 mgd. The vessels normally operate in parallel; however, a bypass flowmeter and control valve also allow for partial GAC treatment of the filtered water or for complete bypass if the vessels need to be taken out of service or if seasonal treatment only is desired.

From the GAC facility, treated water and bypass water combine and flow to the treated water vault where effluent water is metered, and chlorine, fluoride and corrosion inhibitor are fed. A static mixer provides for chemical mixing prior to the 500,000-gallon finished water ground storage tank (water surface elevation of 645 ft). Finished water is pumped from the ground storage tank to the distribution system, with flow metering provided by a magnetic flow meter installed downstream of the finished water pump station.

Backwash water is stored in a 50,000 gallon elevated tank on the treatment plant site (i.e., the “Waterball”). The filters are backwashed using pressure from this tank. The tank is filled by a supply pump located in the filter pipe gallery.

The existing finished water pump station was recently replaced with a new, canned vertical turbine-style pump station. The new station will continue to draw suction from the ground storage tank and will pump to the Buck’s Elbow Tank in the distribution system. Two 1.5 mgd pumps were installed with the replacement project with a TDH of 310 feet, and space was allotted for a future 1.5 mgd pump to be added, for an ultimate firm capacity at buildout of 3 mgd. All pumps are controlled by variable frequency drives (VFDs).

All of the settled solids and backwash waste is sent to two lagoons that are located on the north side of Three Notched Road across from the WTP. These lagoons are shown in Figure 1-10. Decant from the lagoons is returned to the Beaver Creek Reservoir through an existing discharge permit. Given the limited and difficult access to the lagoons for periodic removal of accumulated residuals, RWSA is working on replacing one of the lagoons with an equalization tank followed by a force main for sewer discharge.

Design documents have been prepared by an outside consultant (SEH) to expand the capacity of the Crozet WTP from 1 to 2.1 mgd, and the improvements are currently under construction.

1.3 Water Distribution Facilities

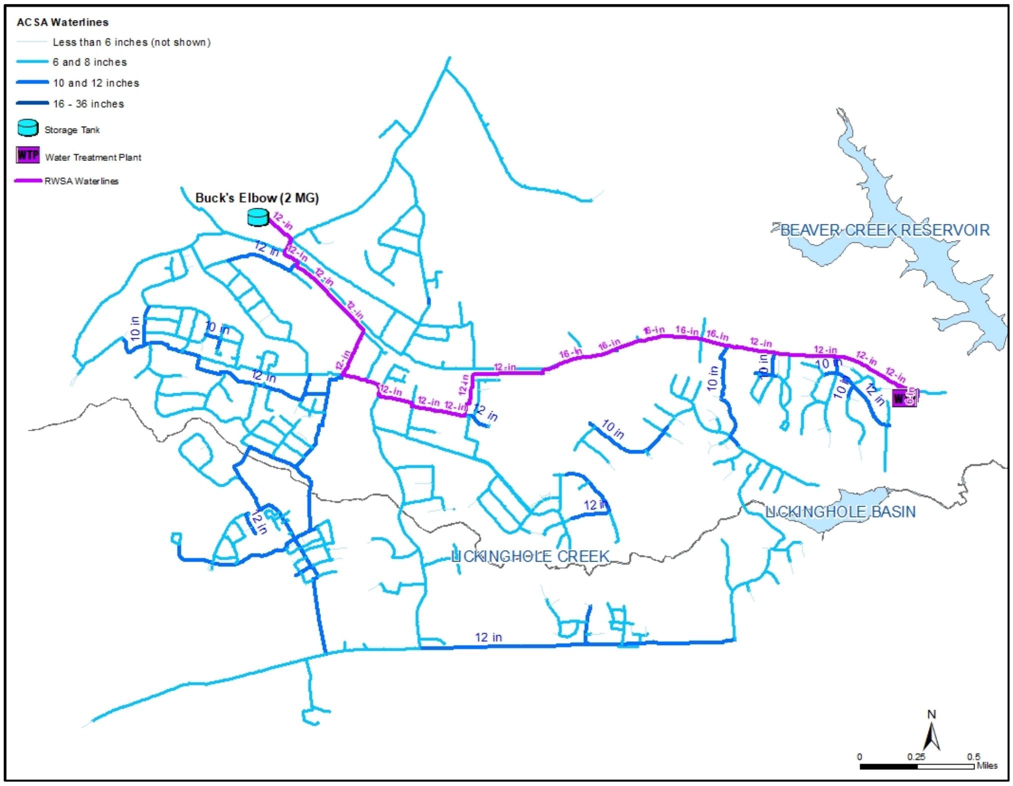
The Crozet distribution system consists of approximately 55 miles of distribution system pipes installed between 1955 and 2017. The breakdown by diameter is shown in Table 1-1. Of the 290,500 linear feet of installed distribution system piping, approximately 268,500 linear feet (92.4%) is owned by ACSA while the remaining 21,980 linear feet (7.6%) is owned by RWSA. The majority of the piping is ductile iron, with very limited areas of asbestos cement pipe and unlined cast iron pipe. The asbestos cement pipe and unlined cast iron pipe that remains in the system is being replaced with ductile iron in the near future.

Table 1-1: Crozet Water Distribution System Pipes

Diameter (inch)	Length (miles)
2	0.2
4	3.6
6	11.6
8	26.7
10	1.6
12	10.4
16	0.9
TOTAL:	**55.0**

The distribution system has one pressure zone (service level) with a nominal hydraulic grade of 905 feet. It is served by the Crozet WTP and high service pumps (as described in previous sections) with a 2-million-gallon ground storage tank called Buck’s Elbow Tank. The Buck’s Elbow Tank has a range of 37 feet with overflow elevation of 905 feet. A map of the distribution system is shown in Figure 1-11.

Figure 1-11: Crozet Water Distribution System



2. Summary of Prior Planning Documents

Prior to the initiation of the various analyses required for development of the DWIP for the Crozet Service Area, the project team reviewed available reports and other documentation associated with the drinking water infrastructure in Crozet. These documents were reviewed to provide a baseline understanding of the existing water supply, treatment and distribution system infrastructure as well as an understanding of prior projections for infrastructure needs. As described in the remainder of this section, these documents consisted of a combination of RWSA and Albemarle County planning documents and engineering study and design reports prepared by others. The principal focus of many of these documents was on water supply need and availability, with a select few focused on the treatment needs at the Crozet WTP. The specific documents reviewed and the primary findings and/or recommendations of these documents are described below.

2.1 Prior Demand Forecasting Reports

The Crozet Service Area has been the subject of numerous planning studies over the years, many of which were focused on understanding the current and future demands for finished drinking water for the service area. The earliest of these prior planning documents, dating back to 1997, were attempting to understand the demands for Crozet as they related to the total safe yield available from the Beaver Creek Reservoir. The 1997 demand projections report was prepared by VHB and O'Brien & Gere, while the next report was prepared by Gannett Fleming in May 2004. These analyses were done in the context of leveraging "excess" safe yield from the reservoir to augment the other available supplies to bolster the Urban System safe yield[2]. These early documents assumed a 93 gallon per capita per day (gpcd) finished drinking water usage rate.

Several more recent documents provided a specific focus on the Crozet Service Area itself, not in the context of supporting the Urban System. These documents were reviewed and utilized to support the estimation of future water demands described in Section 3 of this report, and included the following:

- Crozet Master Plan, Albemarle County Department of Community Development, October 2010
- RWSA Regional Water Demand Forecasts, AECOM, September 2011
- Comprehensive Sanitary Sewer Model Report, Greeley and Hansen, September 2016

These documents reflected the reduction in per capita usage seen in recent years compared to the late 1990s / early 2000s, noting an average usage across all categories of 68.3 gpcd. The documents noted that land use in the Crozet Service Area is primarily residential, with the majority (88%) of water usage coming from indoor uses. The build-out projection for total population was 18,000 people, with approximately 80% of buildout occurring by the year 2060. The County noted that the majority of future development would be commercial or mixed-use in nature keeping with the neighborhood model concept.

[2] The Urban System consists of the City of Charlottesville proper, the University of Virginia and the urban area around the City within Albemarle County.

Given these factors, the average day demand for finished drinking water was estimated to grow to 1.06 mgd by 2060, with a reduction to 0.99 mgd if additional conservation measures are put into place. For reference, the current average day demand is approximately 0.5 mgd with similar average per capita unit water usage.

2.2 Prior Water Supply Studies

As part of the Urban System Water Supply Alternatives Evaluation that was performed in response to the 2001-2002 drought, Gannett Fleming developed a series of reports and technical memoranda that touched on the water supply infrastructure and overall supply availability associated with the Beaver Creek Reservoir. These reports included the following:

- Safe Yield Study, January 2004
- Safe Yield Study Supplement No. 1, July 2004
- Water Supply Alternatives Supplemental Evaluation, July 2004
- Technical Memorandum on Beaver Creek Reservoir Pool Level Fluctuations, September 2004
- Flood Storage Allocation Memorandum, September 2004

These documents developed initial estimates of the available safe yield from the Beaver Creek Reservoir based on no voluntary conservation releases, a 9.55 mi^2 drainage area, and a storage volume at permanent pool of 520 million gallons (MG). The Pool Level Fluctuation memorandum touched on the amount of sedimentation that occurs in the reservoir and noted that the available sediment storage provided in the design of the dam below elevation 515.4 ft would last for nearly 120 years given sediment accumulation rates at the time of the study. Further, the Safe Yield Study Supplement adjusted the effective drainage area for safe yield calculations and clarified that the 520 MG of storage at normal pool is the usable storage compared to the total storage available of approximately 587 MGD.

In 2007 Gannett Fleming issued a safe yield study for the Beaver Creek Reservoir that complemented the 2004 reports. The safe yield study assumed a water supply pool capacity of 521 MG and used a drainage area adjustment to the flows recorded at the Mechums River gage to estimate inflows to the reservoir. Based on the work performed in support of the safe yield study, it was estimated that the safe yield of the Beaver Creek Reservoir is approximately 1.8 mgd. This estimate assumes no minimum release, no reserve storage and no demand reductions during drought conditions (for the sake of calculating a theoretical yield).

2.3 Prior Water Treatment Needs Analyses

The principal document reviewed as part of the water treatment plant capacity assessment was the 2017 Preliminary Engineering Report for the Crozet Water Treatment Plant Expansion and Rehabilitation Project prepared by SEH, Inc. This report assessed the need for facility rehabilitation and capacity expansion to 2.1 mgd. The Preliminary Engineering Report (PER) recommended completion of the upgrade and expansion of the Crozet WTP by the year 2019. Key recommendations coming from the upgrade and expansion PER included:

- Replace existing rapid mixer with larger unit
- Modification of the existing flocculation basins to provide two stages of powdered activated carbon (PAC) contact followed by a single train of three-stage, tapered energy flocculation
- Addition of plate settlers to the existing sedimentation basins to increase the capacity without needing to expand the sedimentation footprint
- Relocation of existing chemical feed systems from the filter building to the chemical feed additions at the GAC building.
- Re-rating of the existing dual-media filters to a loading rate of 4 gpm/sf to increase filtration capacity without the need to construct additional filters
- Modifications to existing chemical feed systems and construction of a new PAC system
- Demolition of existing Sludge Lagoon No. 1 and construction of a new 105,000 gallon concrete backwash waste equalization tank

As of the finalization of this DWIP report for the Crozet Area, the WTP expansion design is complete and construction of the improvements is underway.

3. Demand Forecast Development

3.1 Methodology and Sources of Data

Prior to beginning development of water demand forecasts, Hazen reviewed the following historical planning documents:

- 2010 Crozet Master Plan
- 2011 RWSA Regional Water Demand Forecasts
- 2015 Albemarle County Comprehensive Plan
- 2016 Sewer Model Report

The information gleaned from a review of these documents, which included a number of widely varying water demand projections, helped to guide the initial analysis and facilitate the final development of independent estimates of projected water demands for the Crozet Service Area over the 50-year planning horizon.

In an effort to better understand prior planning projections as well as current planning, Hazen met with RWSA and Albemarle County in November 2017 to review the aforementioned prior planning projections and discuss recent trends that could impact Hazen's independent demand forecasts. The intent of these meetings was to gather additional information on the following, which was then used to further inform the demand forecasting effort:

- Assumptions made in the development of prior demand forecasts
- Recent observations in per capita demand by sector
- Status of any conservation or other programs that may impact demands for finished drinking water
- Current breakdown in sectoral development
- Growth projections for the Crozet Service Area, including changes in zoning, land use patterns and/or buildout projections for each type of land use in the service area
- Average day and maximum day demands (both production flow data and finished water pump station flow data)
- Peak water use and the reasons for that peak use, taking into consideration the flow in and out of both the Ground Storage Tank at the Crozet WTP and the Buck's Elbow Storage Tank
- Status of any known major developments, new industries or commercial facilities that are planned for the service area

- Nature of existing customers to glean potential impacts of fixture replacement as part of a water conservation program

The 50-year projections presented herein have been made in 5-year increments for both average day and maximum day demands, with maximum day demands calculated from the projected average day using a peak factor derived from historical water production data.

This section outlines the detailed methodology that was used to develop the demand forecasts.

3.1.1 Projection Methodology

Below is the step by step process by which future demands were projected.

Step 1 - Review Yearly Historical Billing Data and Production Data
- Determine current average daily demand (ADD)
- Determine the current non-revenue water contribution
- Recommend a suitable maximum daily demand (MDD)/ADD ratio

Step 2 – Analyze Billing Data More Closely
- Identify current per capita usage, including distribution system non-revenue water
- Identify current per employee usage, including distribution system non-revenue water

Step 3 - Review Zoning Data to Identify Future Water Use
- Identify growth in population
- Identify new non-residential usage and growth in employees
- Estimate future ADD
- Estimate future MDD

Step 4 – Develop Growth Curve Envelope
- Perform a historical growth evaluation
- Assess impact of more rapid growth
- Assess impact of development outside of the current service area boundary
- Assess impact that conservation measures may have on demands

3.1.2 Industry Trends

The following general water industry trends were given consideration throughout the evaluation and in some cases (residential usage) data analysis indicated the Crozet Service Area is not following along the lines of every industry trend.

- Declining Gallons Per Capita Day (gpcd): Due to various factors such as changes in behavioral patterns, conservation measures, and more efficient plumbing fixtures.

- Declining Maximum Daily Demand (MDD)/Average Annual Daily Demand (ADD) Ratio: Increases in water rates and trends towards less irrigation have the effect of reducing the MDD/ADD ratio. It is also anticipated that as the system grows, demands and demographics will trend toward more multi-family dwellings, which further attenuate total finished water demand. The reduction can be offset somewhat due to factors such as improved SCADA control systems that assist utilities in distributing water efficiently and improved water meter data analysis, which reduces non-revenue water and increases the MDD/ADD ratio.

- Non-Revenue Water Reduction and Water Conservation Impacts: These impacts are captured by #1 and #2 above.
- Redevelopment: Can lead to lower gpcd with more efficient water fixtures but can also lead to higher demands where redevelopment involves increased development density.

Proper planning involves working to strike a balance between predicting the future and being conservative enough to avoid potential risks in not being able to meet future demands. Through sensitivity analyses, which were conducted for this effort, risks were assessed and addressed, as explained further within this section.

3.2 Projections

Each step of the demand forecast process, including the associated results, is detailed below.

3.2.1 Review of Yearly Historical Billing Data and Production Data

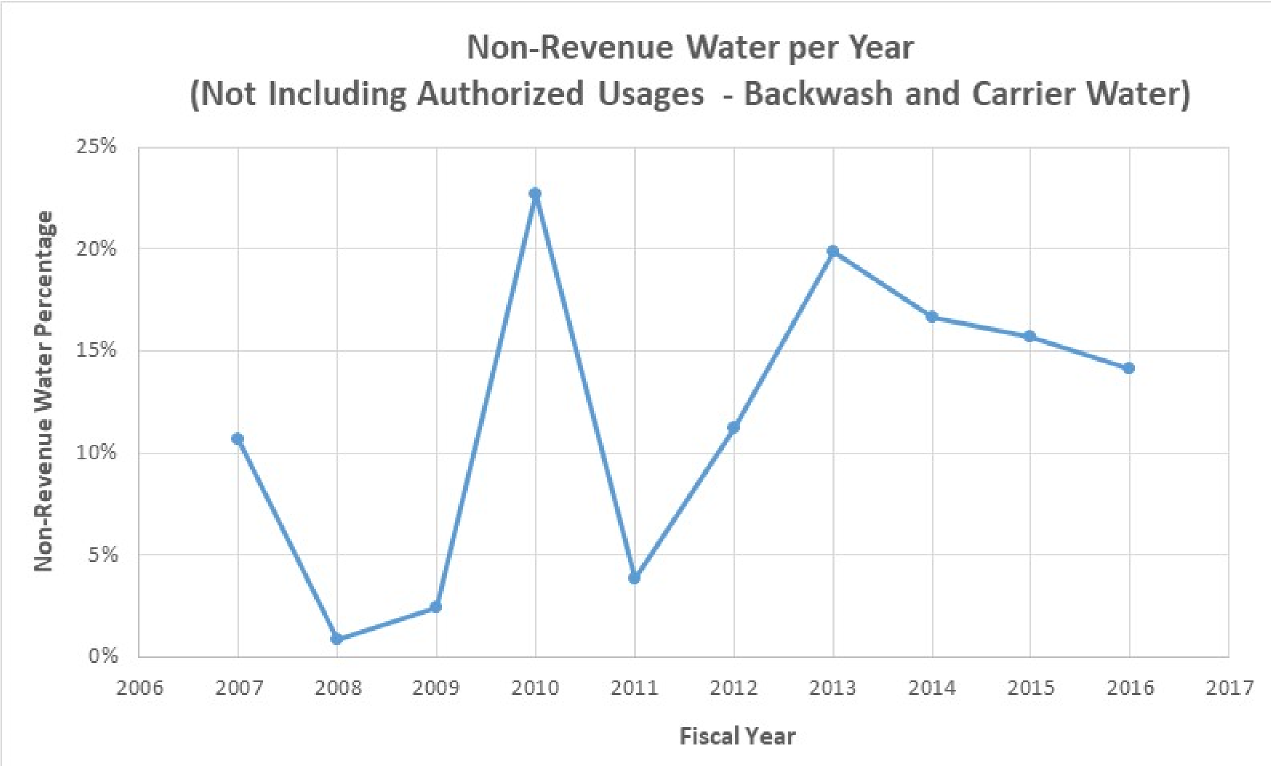
- Determine current ADD

  Billing data and production data were analyzed and the current ADD was determined to be 0.513 mgd. It is important to note this demand includes non-revenue water within the water distribution system, but does not include in-plant water usage (i.e. backwash water and carrier water). This demand served as the starting point for all future growth curves. In-plant water usage is estimated at approximately 10% of water demand. Therefore, 10% should be added to the demand projections included within this section to meet treatment and raw water needs.

- Determine/recommend non-revenue water (as a percentage, %)

  Similar to the analysis necessary to identify an ADD, billing data were compared to production data and SCADA data to determine a realistic non-revenue water percentage (%) for future demands. The average non-revenue water % has been 12% since 2007. However, for purposes of this planning analysis, 15% non-revenue water is recommended for future demands as non-revenue water has fluctuated over time (Figure 3-1). It is important to note that for demand projection purposes, non-revenue water % was not directly used in a calculation, rather it is used to assist in determining unit demands (described in Section 3.2.2). The unit demands used for future demand projections include a component for non-revenue water. While the recent downward trend in non-revenue water was recognized, the 15% value was used for planning purposes to provide a degree of conservatism in the projections.

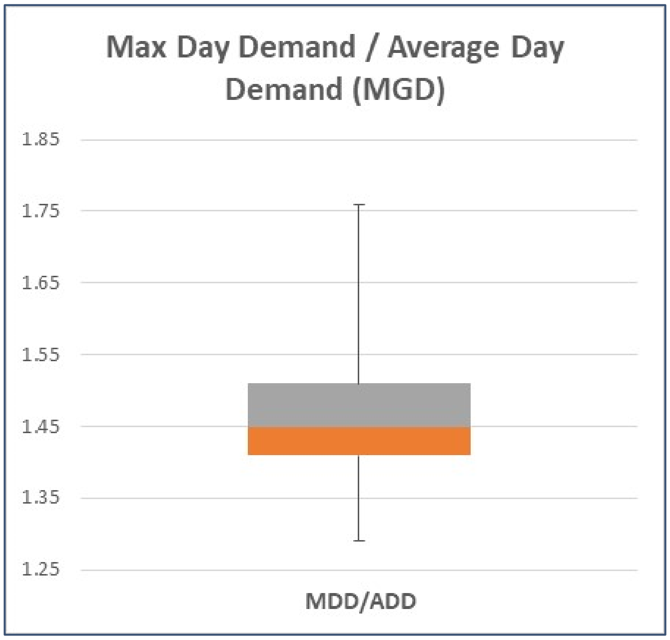
Figure 3-1: Non-Revenue Water Since 2007



- Determine/recommend an MDD/ADD ratio

Historical production data as well as billing data and SCADA data were used to perform a statistical analysis and recommend a reasonable MDD/ADD ratio for future planning purposes. The analysis completed assessed water that was consumed by customers within the distribution system, including non-revenue water, but excluding in-plant water usage (i.e. backwash water and carrier water). This data set took into consideration SCADA data to assess the impact that storage has on water production. Drought preparedness, balanced with operational changes to be implemented in the future to reduce the variation in daily demands and help to mitigate higher MDD/ADD ratios from occurring, were accounted for when recommending a 1.68 ratio. This ratio represents an approximate 95% confidence level over the past ten years (Figure 3-2). For reference purposes, the average MDD/ADD ratio is 1.47 since 2006.

Figure 3-2: MDD/ADD Statistical Analysis (2006 to Current)

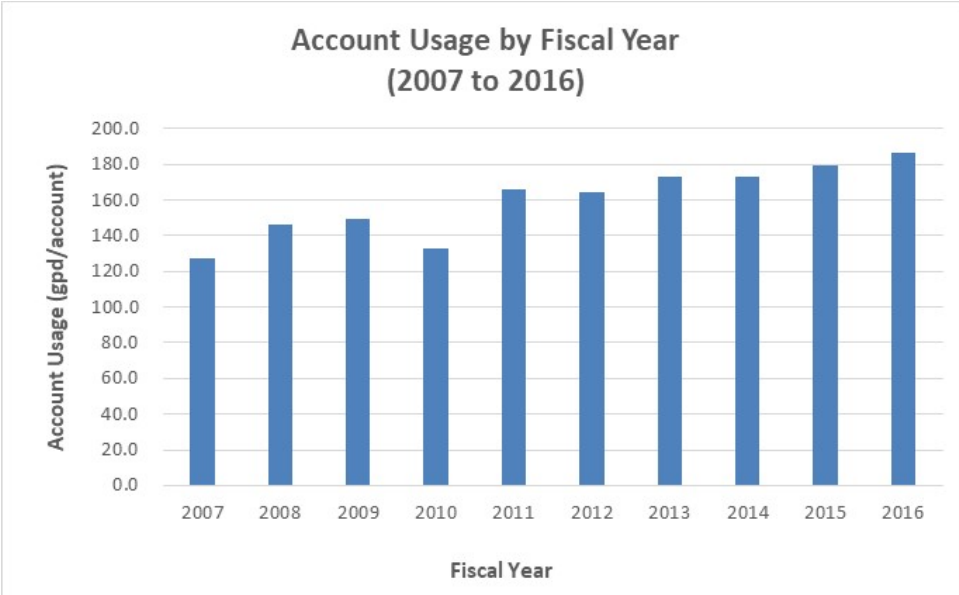


average	1.47
90%	1.60
95%	1.68
min	1.29
1st quartile	1.41
median	1.45
3rd quartile	1.51
max	1.76
Q1-min	0.12
Q1	1.41
Median - Q1	0.04
Q3-median	0.06
max-Q3	0.25

3.2.2 Analysis of Billing Data to Determine Per Capita Usage

Billing data, spanning July 1, 2016 through January 31, 2017, from the Albemarle County Service Authority (ACSA), were analyzed in relation to billing account type to assess how unit water demands have changed over time by sector. The unit water demand below is a blended unit water demand for both residential and employment uses. As can be seen in the figure, unit water demands have generally increased since 2007. However, it is apparent that variability in this blended unit water demand still exists.

Figure 3-3: Crozet System Wide Unit Water Demand (Includes Non-Revenue Water)



The billing data were then analyzed more closely using Microsoft Excel and Microsoft PowerBI to identify unit water demands for disaggregated sectors. Microsoft Excel was used initially to calculate water demands based on billing information and water connection information (separated by number of connections and connection type/sector). Distribution non-revenue water was then added to the calculated water demands to determine unit water demands for each sector. Microsoft PowerBI was used to centralize all the various data sources (i.e. Microsoft Excel spreadsheets) into one data location where additional analyses could be completed. The result of this analysis suggested unit water demands of 65 gpcd for new single-family residential customers, 55 gpcd for new multi-family residential customers and 65 gallons per employee per day (gped) for new employees. These unit water demands include distribution non-revenue water. ACSA billing data for Crozet were used with some caution, as ongoing ACSA meter work, including meter replacement, continues to improve data accounting and sorting. However, given the data's account level resolution, which provides information on connection type/sector, it was important to use the data to aid in general system understanding and planning. These data will continue to be a valuable resource for planning in the future. No adjustments were made to current customer demands, which are based on ACSA real meter billing data with a non-revenue water % added to match RWSA real production data.

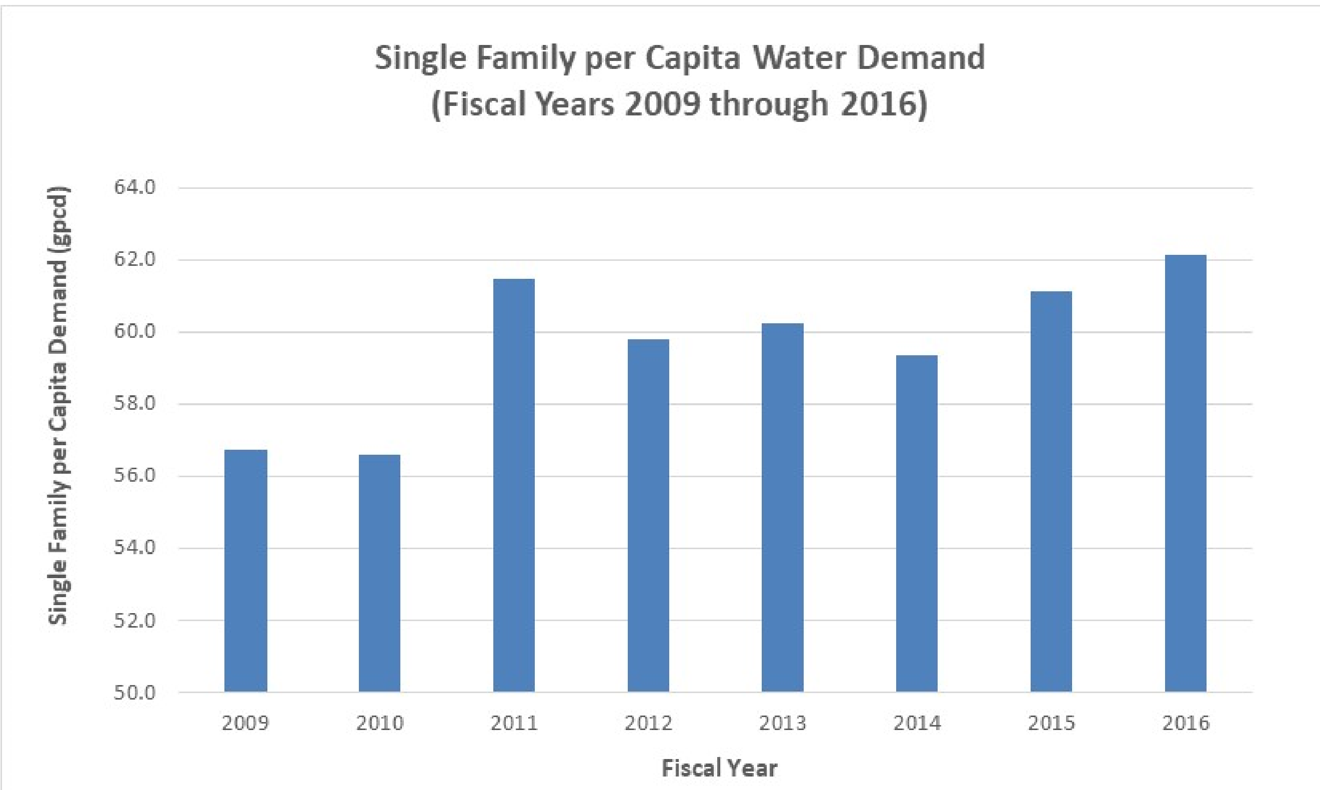
For comparison purposes, using 2010 census data, unit water demands were estimated to be 54 gpcd for new residential customers and 44 gped for new employees. Based on the analysis completed, there has not been much change in account type distribution since 2010 (~74% residential and ~26% non-residential), but there has been a slight increase in residential account usage and a substantial increase in non-residential account usage, which likely can be attributed to an increase in customers and employees per account. For analysis purposes, and based on discussions with Albemarle County, it was assumed that the number of customers per single family home has grown from 2.5 in 2010 to 2.8 in 2017.

- Identify Per Capita Usage, including Distribution Non-Revenue

A usage of 65 gpcd for new single-family residential customers and 55 gpcd for new multi-family residential customers is recommended for planning purposes. As more billing data is gathered, future consideration can be given to adjusting this unit water demand. Data analysis showed a slight increase in usage over the past few years, which is against the national trend. As noted above, this can likely be attributed to an increase in customers per account (customers per single family home increasing from 2.5 in 2010 to 2.8 in 2017).

Figure 3-4: Single Family Per Capita Water Demand by Year

Values based on 2.5 residents per home ca. 2010 and 2.8 residents per home ca. 2016 and beyond
Values also include 15% distribution system non-revenue water



- Identify Per Employee Usage, including Distribution Non-Revenue

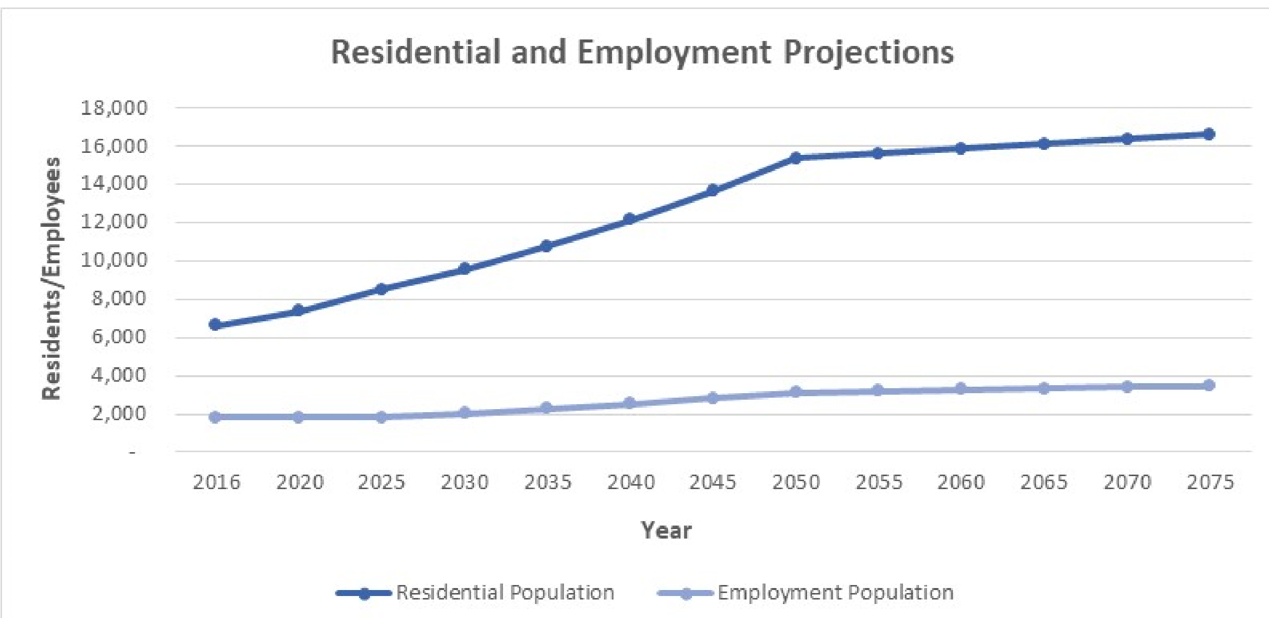
A usage of 65 gped has been recommended for current planning purposes. Similar to the residential unit water demands discussed above, as more billing data is gathered, future consideration can be given to adjusting this unit water demand. Employee demands vary widely, much more so than residential unit demands. This is not unusual, given the wide variation in employment types (i.e. institutional (schools), industrial, commercial businesses, etc.). Data analysis showed a substantial increase in account usage over the past few years, which can likely be attributed to an increase in employees per account.

3.2.3 Review of Zoning Data to Identify Future Water Use

Hazen utilized available land and current land use GIS data from the Albemarle County Office of Geographic Data Services (GDS) to assess the location of future developments. Based on this analysis it is estimated that residential growth will far outweigh non-residential growth over the 50-year planning horizon, with no non-residential growth projected to occur until after 2025. Population and employee projections were made using the following unit factors:

- 2.8 people per dwelling unit (Single Family)
- 2.0 people per dwelling unit (Multi-Family or Townhome)
- Non-residential to employees:
    - Institutional: 800 ft^2/Employee
    - Light Industrial: 300 ft^2/Employee
    - Mixed-Use (Retail and Commercial): 800 ft^2/Employee

Figure 3-5: GIS Based Residential and Employment Projections



- Estimate Growth in Population and Employees

  Growth in population and employees was assumed to be linear between the following planning horizons. Next to each planning horizon is the portion of the Crozet Service Area that is assumed to develop within that timeframe.

    - 2017 to 2025 – Approved residential dwelling units (RDU), under review RDUs, and near term planned RDUs
    - 2025 to 2050 – 80% of future developable land minus areas that are not developable as listed below:

    - 2025 planned growth
    - Parcels with existing buildings/water consumption
    - Land usages including greenspace, parks and green systems, river corridor, public open space, privately owned open space, and environmental features
  - 2050 to 2075 – Same as 2025 to 2050 – remaining 20% of developable land.

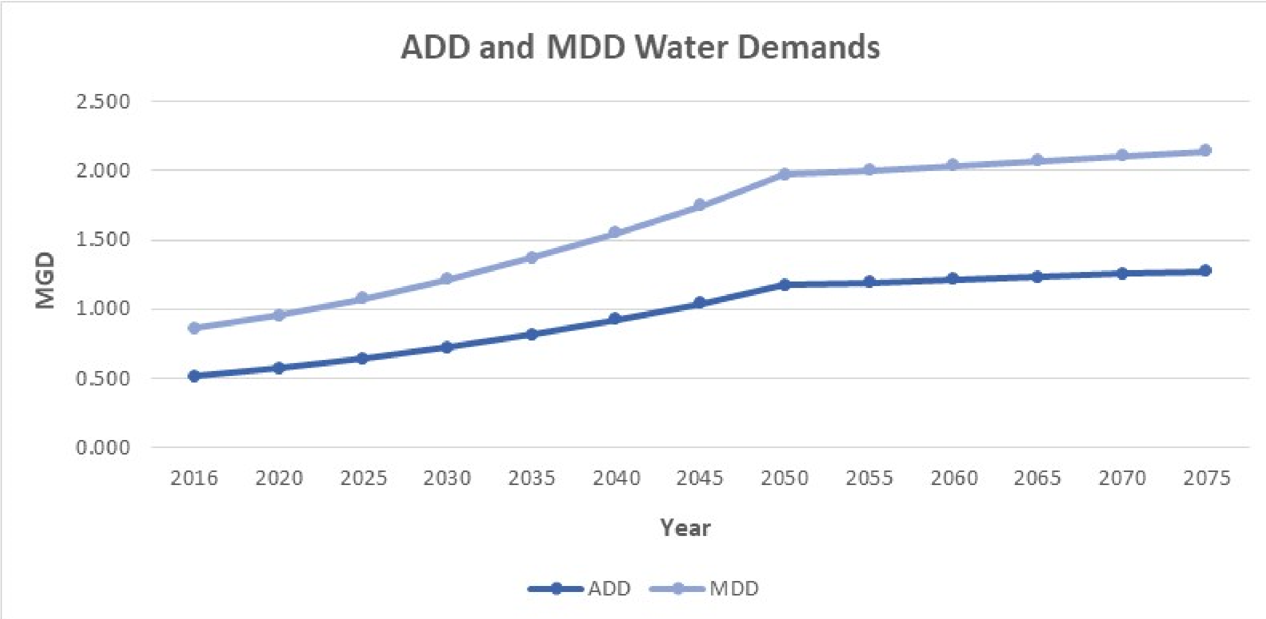
- Develop Baseline Demand Forecasts
  Unit water demands identified in Section 3.2.2 were multiplied by the changes in population described above and then added to current demands (0.513 mgd) to project annual average water demands in future planning years. The results of this analysis are shown below in Table 3-1 and represent the ADD.

  ADD at each future planning year was then multiplied by the MDD/ADD ratio identified in Section 3.2.1 to project maximum daily water demands in future planning years. The results of this analysis are shown below in Table 3-1 and represent the MDD. Figure 3-6 shows both the ADD and MDD growth in 5-year increments through the year 2075.

Table 3-1: ADD and MDD (in MGD) Water Demand Projections Through 2075

	Residential Projection	Growth Percentage	Water Demand (Residential + Employment) * *Does Not Include In-Plant Water Use	Additional Residential Flow (65 gpcd)	Employment Projection	Growth Percentage	Additional Employment Flow	Total ADD Demand (MGD)* * No 10% In-Plant Water Use	Total MDD Demand (MGD)
2016	6,603	2.82%	0.513		1,810	1.68%		0.513	0.862
2020	7,378	2.82%		0.056	1,810	0.00%	0.000	0.569	0.956
2025	8,477	2.82%		0.128	1,810	0.00%	0.000	0.641	1.076
2030	9,548	2.41%		0.197	2,019	2.21%	0.014	0.724	1.216
2035	10,754	2.41%		0.276	2,252	2.21%	0.029	0.817	1.373
2040	12,113	2.41%		0.364	2,512	2.21%	0.046	0.922	1.550
2045	13,643	2.41%		0.463	2,802	2.21%	0.065	1.041	1.749
2050	15,367	2.41%		0.575	3,126	2.21%	0.086	1.174	1.972
2055	15,603	0.31%		0.591	3,189	0.40%	0.090	1.193	2.005
2060	15,842	0.31%		0.606	3,254	0.40%	0.094	1.213	2.038
2065	16,085	0.31%		0.622	3,320	0.40%	0.098	1.233	2.072
2070	16,332	0.31%		0.638	3,387	0.40%	0.103	1.254	2.106
2075	16,583	0.31%		0.654	3,456	0.40%	0.107	1.274	2.141

Figure 3-6: Projected Water Demands (ADD and MDD) – through 2075



3.2.4 Develop Growth Curve Envelope

In addition to the growth curve developed in Section 3.2.3, various alternative growth projection methods were considered to develop a growth curve envelope. The approach for each alternative growth projection method evaluated is an attempt to answer the following questions:

Can a new standard residential and non-residential unit water demand be applied effectively across a water service area?

Can this new standard serve as an adequate representation of water demands in the near and distant future?

How does the new standard compare to the most recent demands represented by field data (billing data)?

Certain assumptions needed to be made prior to preparing the growth envelope, including the following:

- Unit water demands (gpcd and gped) would not decrease over time – if this were included, demands would be projected lower than presented herein. The residential unit water demands within the Crozet Service Area are generally in line with other systems in Virginia that have similar infrastructure age and home size and represent the water efficiency of new construction. For a reference point, historical standard industry practice often referenced 90 to 100 gallons per day as a suitable per person unit water demand for planning purposes.
- Non-revenue water % will remain fairly consistent over time – the Crozet Service Area has a relatively low non-revenue water % when compared to similarly sized systems. Some of this may likely be attributed to the relative age of Crozet's infrastructure.

- Non-revenue water is distributed throughout the Crozet service area based on demand.

3.2.4.1 Growth Projection Curves

The following growth projection methods were considered. As shown in Figures 3-7 and 3-8, some of these alternative growth curves fall above and some fall below the baseline growth curve from Section 3.2.3 (Zoning ADD and Zoning MDD on the figures). They represent the possible variability in demands that could occur. The recommended growth curve should balance water industry trends, Crozet billing data, and a level of conservatism that addresses the importance to a water system in meeting its obligation to provide drinking water to its customers. The additional growth projections used in preparing the growth curve envelope for ADD and MDD are as follows:

- Historical Usage – 5 year and 10 year linear regression
- Zoning Based Projection – From Section 3.2.3 – ADD of 1.27 MGD, MDD of 2.14 MGD
- More rapid growth – buildout reached by 2060
- More development outside of the current Crozet Service Area, within water only parcels (~0.2 MGD under MDD conditions)
- Modified sectoral usage – billing data analysis supports possible higher usage (75 gpcd single family, 60 gpcd multi-family, and 70 gped non-residential)

Figure 3-7: Project ADD Growth Envelope

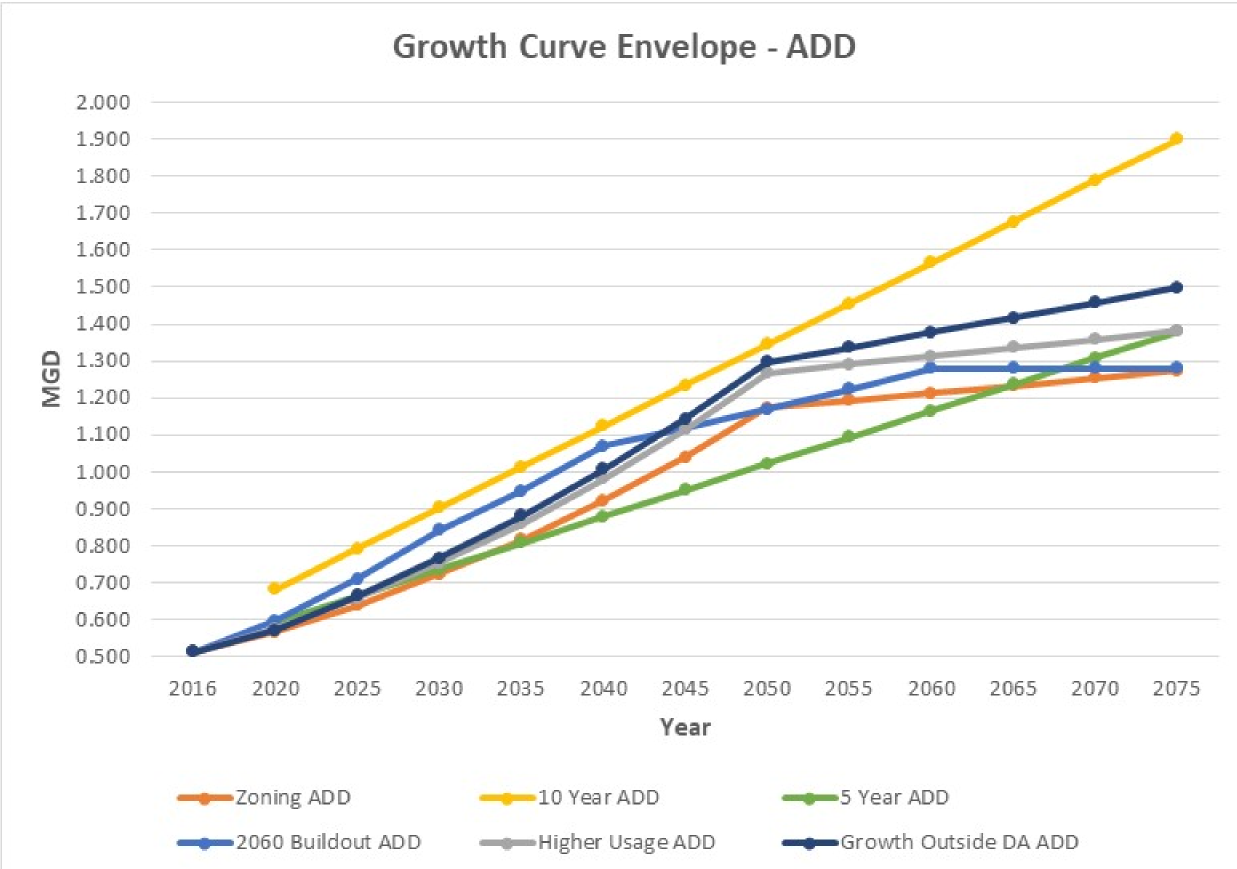
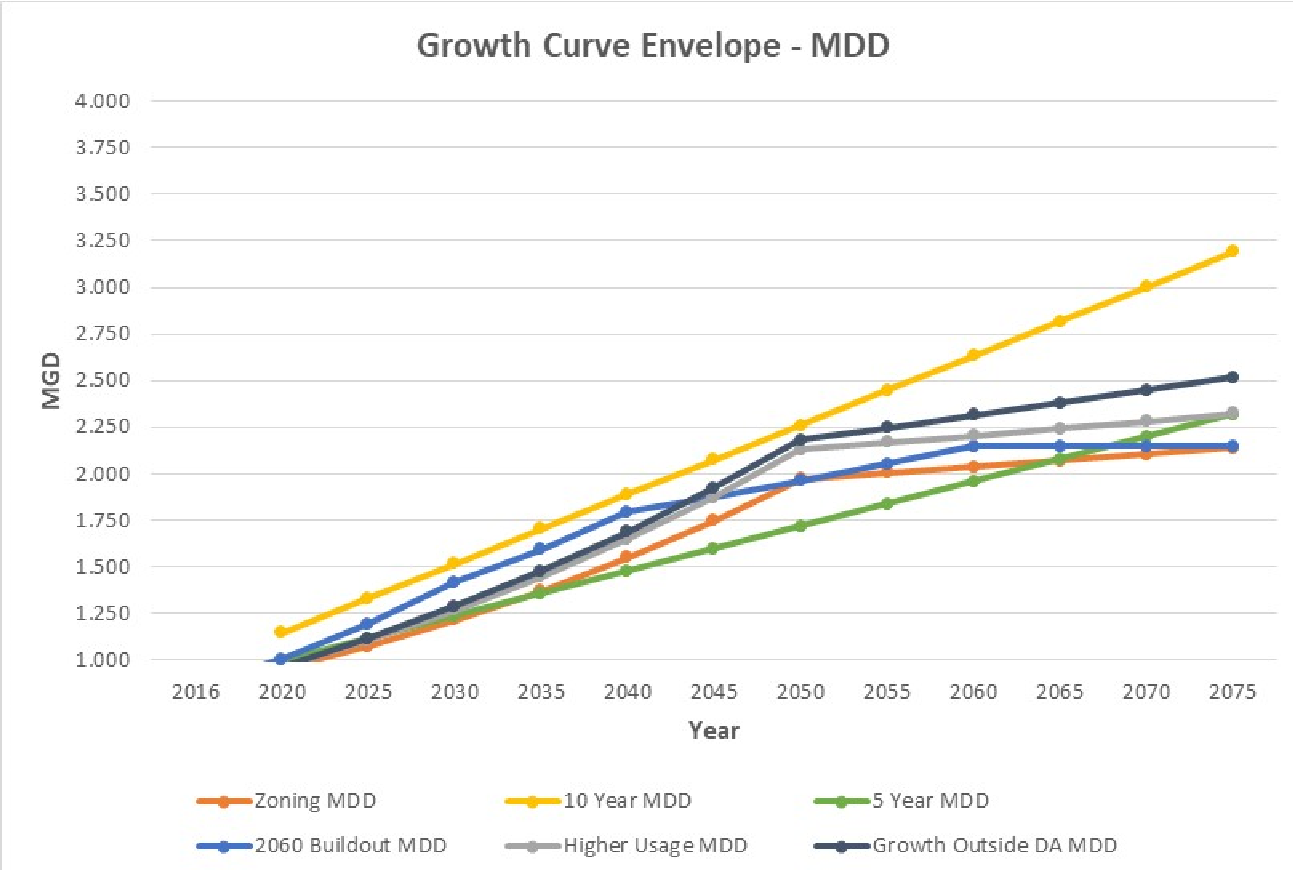


Figure 3-8: Project MDD Growth Envelope



Given the range of projections, for planning purposes it was recommended that the buildout ADD (in 2075) would be 1.5 mgd and the MDD would be 2.5 mgd. When in plant water use is considered (i.e., adding in an additional 10% of production), the required water withdrawal from Beaver Creek Reservoir and the required water treatment capacity at the Crozet WTP under average and maximum conditions would be 1.65 and 2.75 mgd, respectively.

4. Evaluation of Water Supply Capacity and Needs

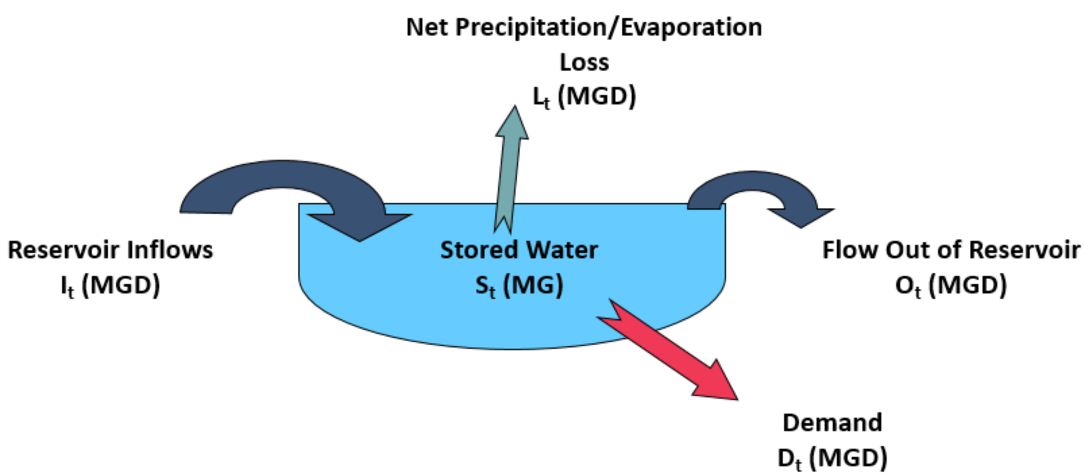
This section describes the identification and evaluation of potential water supply sources, including the existing Beaver Creek Reservoir, for meeting the needs identified in Section 3.

4.1 OASIS Hydrologic Model

Appendix A to this report, Modeling the Crozet Water Supply System with OASIS, is a detailed stand-alone report on the development and results of the Crozet OASIS hydrological model. The following is a summary of that report.

The existing OASIS model of the RWSA Urban System supply was extended to provide a detailed evaluation of the Beaver Creek Reservoir. The model functions on a daily time step and is built on the conceptual framework described in Figure 4-1 below.

Figure 4-1: Conceptual Reservoir Model

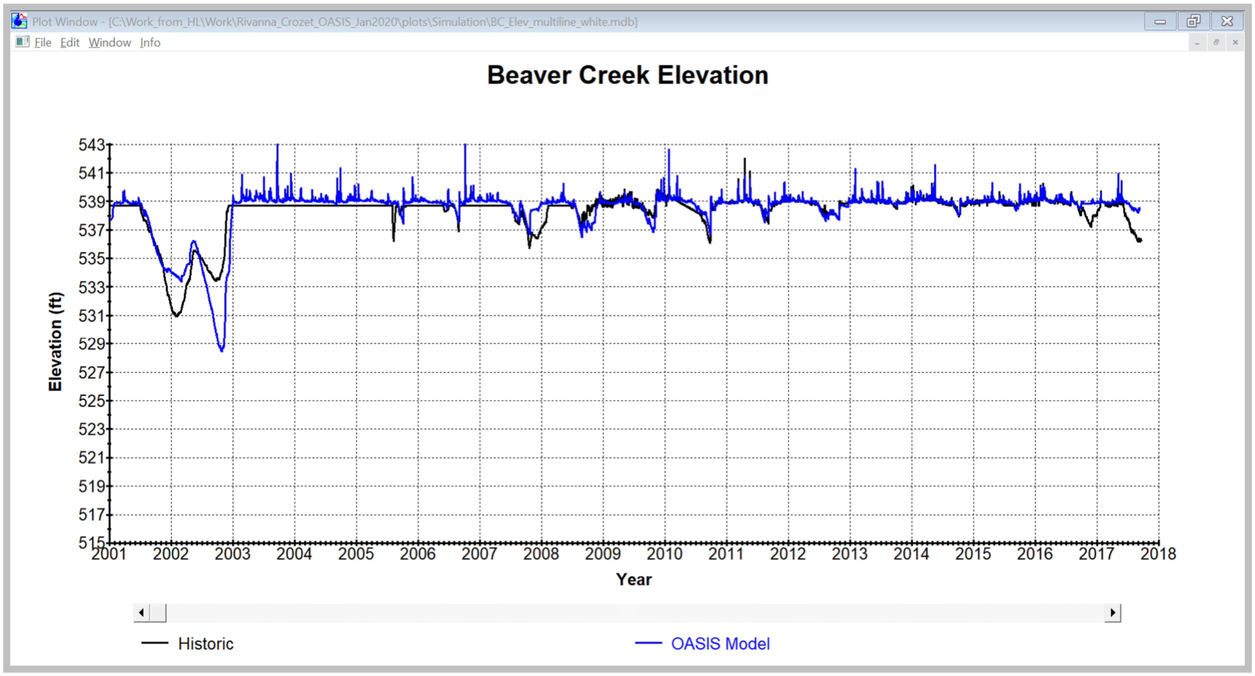


$S_t = S_{t-1} + I_t - D_t - L_t - O_t,$ where S_{t-1} = yesterday's storage

Based on an analysis of data from nearby USGS stream gages, daily flow records from USGS gage 02031000 on the Mechums River at Whitehall, VA, were selected to model inflow to the Beaver Creek Reservoir. Records from the flow meter in the spillway conduit were initially used to model the sum of the flow out of the reservoir and demand components of the reservoir model described in Figure 4-1. However, as shown in Figure 4-2 below, the gage data and spillway conduit flow records provided a poor match with records of actual reservoir levels from the sample 2013-2016 period when standard adjustments were made proportional to drainage area ratio (9.39 mi2 / 95.3 mi2 = 9.9% of gage flow). As a result, extensive additional modeling of reservoir operations were conducted along with model calibration refinements, with the ultimate objective of developing a model that provides accurate estimates of reservoir system yield. The results of this effort are reflected in the far more accurate match between the modeled reservoir elevation and historical reservoir elevation data shown in Figure 4-3. In this graph, significant deviations from standard reservoir operating procedures during the 2002 drought account for the poor match between actual and modeled reservoir levels for that period. In addition, reservoir elevations above normal pool elevation were not recorded until mid-2008. In prior years such elevations were noted in the record log as “full”. Overall, the model results are judged to provide a sound basis for calculating reservoir yield.

Figure 4-2: Original OASIS Model Verification with Area Adjusted Inflows and Ultrasonic Meter Outflow

Figure 4-3: Final OASIS Model Calibration Curve for Beaver Creek Reservoir



The model assumes no significant seepage loss from the Beaver Creek Reservoir through the dam. RWSA staff have indicated they believe this is an accurate assumption.

4.2 Capability of Existing Supply to Meet Projected Growth

Following the calibration phase, the model was then used to determine the reservoir operating yield (i.e. safe yield under a specific set of operating criteria). This set of analyses assumes that downstream losses as described in Section 1.1.2 have been addressed as described in Section 4.5 and that access to the full water supply pool volume is available (water surface elevations down to 515.4 feet). The Beaver Creek Reservoir operating yield estimate without consideration of a minimum release from the reservoir or reductions in demand during periods of drought is 1.71 mgd based on the 1925 – 2017 hydrology with adjustments described in Section 4.1 and Appendix A. The projected annual average finished water demand for the Crozet service area in 2075 is between 1.3 and 1.5 mgd as described in Section 3.

For planning purposes, the VA DEQ standard is to require that the equivalent of 60 days of demand be held in reserve when calculating safe yield. It is also assumed that as a part of the VWP permitting process, accommodations will have to be made for a minimum in-stream flow (MIF). A proposed MIF protocol was presented to the VA DEQ in November 2017 and again in March 2018. Assuming this MIF is accepted along with the proposed conservation demand reductions of 5% and 20% in stages 3 and 4 of the MIF protocol, respectively, the operating yield of the existing reservoir with a 60-day reserve storage is estimated at 1.65 MGD. This is sufficient to meet the upper bound projected finished water average day demand in 2075 including a 10% greater raw water withdrawal rate above the finished water demand to satisfy the process water needs at the Crozet WTP. However, the raw water management system at the reservoir and the Crozet WTP capacity will have to be upgraded to realize the stated water supply potential and continue to meet the projected service area demands. The needs in these two areas are discussed further in Sections 4.5 and Section 5.3.

4.3 Development of Alternatives

Eight alternatives for meeting the projected demand described in Section 3 have been identified. As discussed below, Alternative 1, increase withdrawals from the Existing Beaver Creek Reservoir, is the preferred alternative and meets not only the reservoir withdrawal requirements for the regulatory 15-year VWP permit term but the project planning requirements through year 2075. Seven alternatives to the preferred alternative are also outlined consistent with the Joint Permit Application (JPA) requirements for an alternatives analysis.

4.3.1 Alternative 1 (Preferred Alternative): Permit Increased Withdrawals From the Existing Beaver Creek Reservoir

Currently, the Beaver Creek Reservoir has excluded status in the VWP permit system managed by the VADEQ. To receive regulatory approval to increase withdrawals from the Beaver Creek Reservoir above 1 mgd, RWSA will need to submit a JPA. The concept for this alternative involves moving the Beaver Creek Reservoir from excluded status in the VWP permit system into the VWP permit system (additional discussion in Section 7.1). Based on the analyses and facility improvements, **this alternative involves no augmentation of reservoir storage in Beaver Creek Reservoir**. It is therefore the preferred alternative and fully meets the needs for (a) projected finished water demand through 2075, (b) process water needs at the Crozet WTP, and (c) a four-tiered MIF protocol as outlined in Table 4-1 below, which would be capped at 1.5 cfs when the reservoir is below normal pool elevation (538.7').

Table 4-1: Proposed Minimum In-Stream Flow

Tier	Reservoir Storage	Downstream Release (MIF)	Associated Demand Reductions (Conservation)
1	>= 90%	50% of inflow*	none
2	80% – 90%	40% of inflow*	none
3	70% - 80%	20% of inflow*	5% reduction
4	< 70%	10% of inflow*	20% reduction

* - up to 1 mgd (1.5 cfs)

Section 1 provided a description of the existing spillway riser and raw water pump station infrastructure. This infrastructure will need to be improved for RWSA to meet the requirements of the proposed MIF, or any other MIF likely to be accepted by the regulatory agencies, and simultaneously manage water supply storage in the reservoir in a manner that will maximize water supply operating yield. One concept for meeting these needs involves constructing a dedicated water supply intake in the reservoir that does not rely on the existing spillway and providing for automated adjustment of the sluice gates with the frequency and precision necessary to meet the MIF and maximize reservoir storage within the constraints of the MIF. A concept for providing this functionality is described in more detail in Section 4.5. However, these modifications will need to be coordinated with the dam safety improvements project that is also currently in the planning phases. Assuming this is the selected alternative, the ultimate development of these infrastructure improvement concepts is highly dependent upon the spillway modification alternative selected to meet current dam safety criteria. Finally, the raw water transmission line between the reservoir and the Crozet WTP may need to be relocated in conjunction with the construction of a new supply intake.

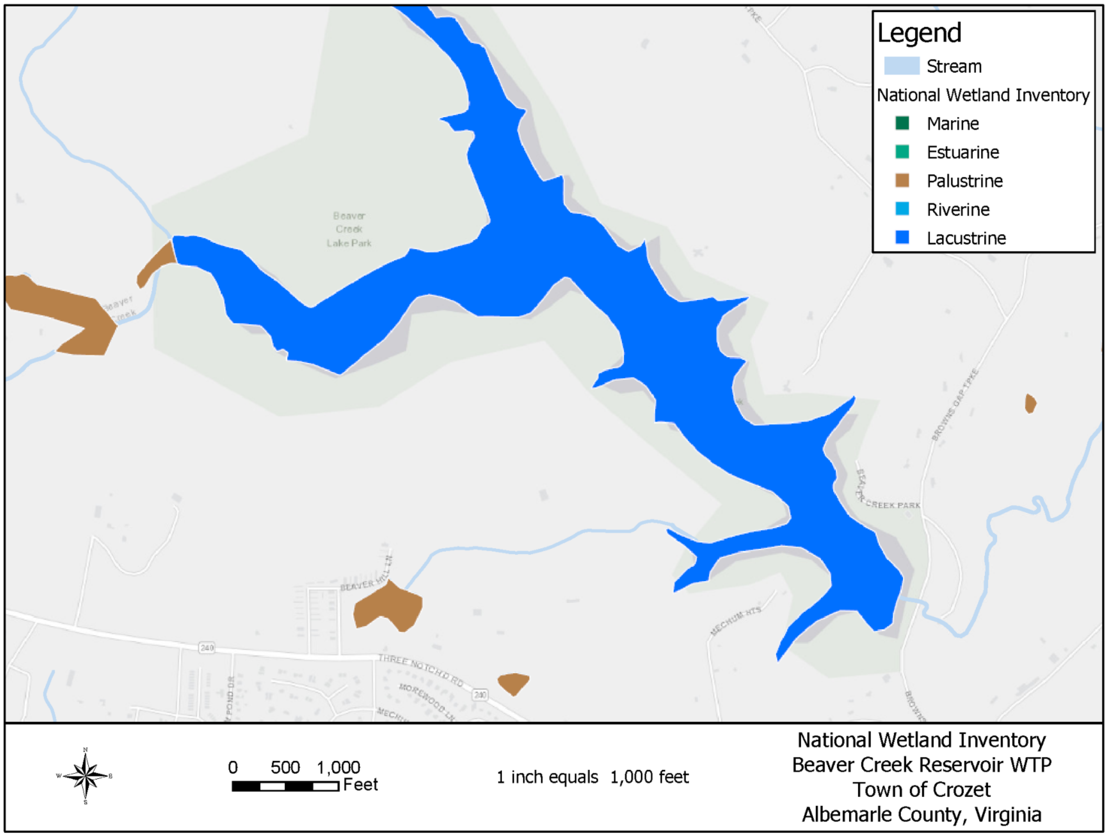
Despite some remaining uncertainty in the dam safety improvements and approval of this alternative, the JPA-VWP process was begun in November 2017 with a Pre-Application Meeting with the VADEQ and followed up in March 2018 with a Pre-Application Panel Meeting organized by the VADEQ to which other stakeholder regulatory agencies were invited. Initiating the permitting process was considered necessary to advance the VWP portion of the JPA to the point that required infrastructure improvements can be concurrently designed with the spillway and emergency spillway improvements.

4.3.2 Alternative 2: Raise the Beaver Creek Reservoir Dam to Increase Storage Capacity

This alternative involves increasing the height of the Beaver Creek Dam to accommodate a larger water supply pool, should such be ultimately required for the VWP permit or long-term planning purposes. This alternative would involve major costs, dam safety permitting, and environmental impacts that would accompany the inundation of additional land around the reservoir. There are some wetlands at the western upstream end of the reservoir that could be inundated depending upon the additional height added to the dam as shown in Figure 4-4.

Additional stream reaches would be inundated as well. The advantage of this alternative would be that the operating yield could be increased and additional storage provided to meet a greater MIF during dry periods than is possible within the constraints of the existing water supply storage volume.

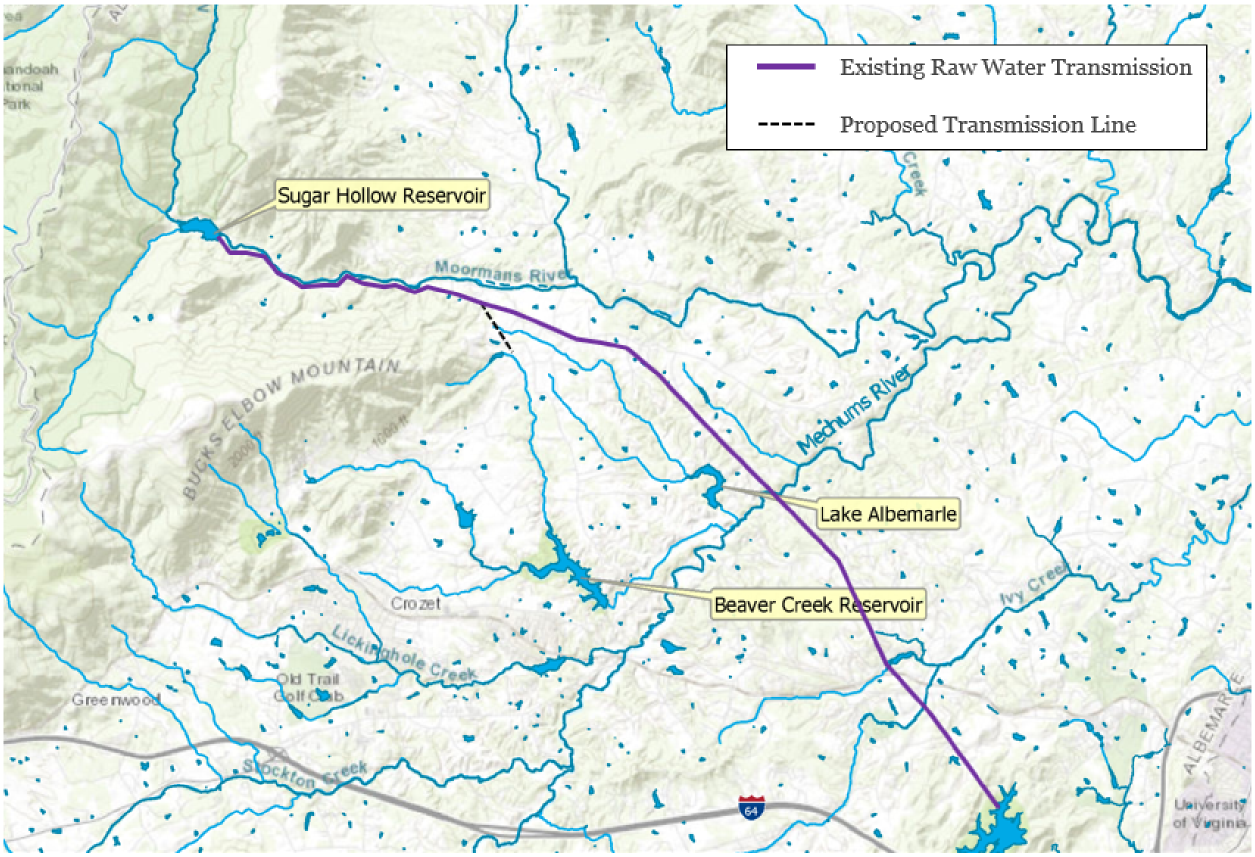
Figure 4-4 National Wetland Inventory Map for the vicinity around Beaver Creek Reservoir



4.3.3 Alternative 3: Transferring Water from the Sugar Hollow Reservoir to the Beaver Creek Reservoir

This alternative involves making use of the Sugar Hollow Reservoir storage to augment the natural runoff into the Beaver Creek Reservoir so that sufficient yield is available to meet the Crozet Service Area demands throughout the planning period. An existing raw water pipeline running from Sugar Hollow Reservoir to the Ragged Mountain Reservoir passes nearby to the headwaters of the Watts Branch of the Beaver Creek. An additional section of pipeline ½ mile to 1 mile in length, depending on the route chosen, could be installed from the existing transmission line allowing for gravity delivery of water into the Watts Branch which would then flow into the Beaver Creek Reservoir as shown in Figure 4-5 on the following page. This alternative would involve all the permitting required for increasing withdrawals from the existing reservoir plus approval for the additional pipeline and study the hydrologic impacts of the extra flow discharged into the Watts Branch to supplement Beaver Creek Reservoir storage. There are areas of freshwater forested/shrub wetland along the Watts Branch that may be impacted by this alternative[3] in addition to the need for the additional leg of pipeline to make stream crossings. The operation and management of the alternative would need additional study to determine impacts to the Urban System water supply yield and impacts due to hydrologic alteration in the Beaver Creek and Moormans River basins.

Figure 4-5: Orientation to Sugar Hollow Pipeline and Lake Albemarle Alternatives



4.3.4 Alternative 4: Connecting to the Charlottesville Urban Water System

This alternative would involve extending a finished water pipeline from the RWSA Urban Supply System near the town of Ivy to Crozet, a length of approximately 5 miles[3]. The pipeline would likely have several stream crossings including most significantly the Mechums River. It would add to the burden placed on the Urban System to meet growing demand, but likely could be accommodated within the range of future supply options available to RWSA. However, this alternative would create additional development pressure along the finished water pipeline route and such development would be at odds with

[3] Emily Steinweg, SEH memorandum "Rivanna Task Update SHE No. RIVAN 138263 14.00" dated October 24, 2017

longstanding policy by Albemarle County to maintain the rural character of the County and direct growth to designated areas, including the Community of Crozet.

4.3.5 Alternative 5: Developing Groundwater Wells

This alternative would involve searching for an area where wells could be installed that would yield a sufficient supply to meet the additional projected need for the Crozet service area. Well production in most parts of Albemarle County is not sufficient to meet the needs of a municipal supply of the magnitude anticipated in Crozet, but there are similarities between the geology and topography around Crozet and a few other areas in Virginia that are reported to have high yield groundwater production with Waynesboro, VA being the closest site noted in a USGS Report from 1972 on this subject. Since locating high yielding wells is not guaranteed, this alternative would involve risk in the exploration phase and could not be relied upon as a viable alternative until/unless such investigations come to an affirmative conclusion. The quality of any groundwater would need to be evaluated for drinking water suitability as well. Even if locating productive wells is successful, this alternative may have impacts on wetlands and streams in the area depending on the location, the extraction rate at the well and the interaction between groundwater and surface waters in the area of the well or well field.

4.3.6 Alternative 6: Divert Water from Lake Albemarle

This alternative would involve diverting water from nearby Lake Albemarle to the Beaver Creek Reservoir or directly to the Crozet WTP. Lake Albemarle is approximately 2 miles to the northeast of Beaver Creek Reservoir and 2.3 miles northeast of the Crozet WTP as shown above in Figure 4-5. It was built by the Civilian Conservation Corps in 1938 and is owned and operated by the Virginia Department of Game and Inland Fisheries (DGIF). It impounds a drainage area of just over 3.6 square miles which is about 38% of the area impounded by the Beaver Creek Reservoir. As such, it is not likely to have a safe yield sufficient to satisfy all the additional demand for the Crozet Service Area projected between now and 2075. Nevertheless, utilizing its supply could potentially take some of the burden off of the Beaver Creek Reservoir. Its use would require the same permitting studies required for Beaver Creek Reservoir, plus the construction of an intake and pipeline to Beaver Creek Reservoir or the Crozet WTP. However, perhaps the biggest obstacle to its use for Crozet's water supply would be reaching an agreement with DGIF for withdrawing water from the reservoir while still meeting DGIF's recreational and environmental goals for the facility.

4.3.7 Alternative 7: Build a New Reservoir in an Alternate Location

This alternative would involve identifying a new site to build a dam and impoundment either as an in-stream reservoir or a pumped storage reservoir supplied by a nearby river. Compared with the preferred alternative, the required construction, including related pipelines, would certainly have far higher construction costs and environmental impacts. The project would also face formidable permitting difficulties in the current socio-political landscape.

4.3.8 Alternative 8: Demand Conservation

This alternative consists of implementing various water conservation/efficiency measures within the service area to lessen current and future water demand. As indicated in Section 3.2.2, single family residential demand comprises about ¾ of the total billed volume in the Crozet Service area. While per capita residential demand appears to have risen slightly in recent years, likely owing to the increasingly affluent demographic makeup of the community, the overall estimated per capita demand for single family homes of about 65 gpcd (including non-revenue portion) is still 20% less than the average for all housing types of 75 gpcd for the State of Virginia in 2010[4] and presumably the state-wide average for single family dwellings would be higher than the cited state-wide figure for all housing types. While reducing per capita demand may have some effect on curbing the overall growth of demand in the service area, it will not be possible to satisfy demand over the planning period without expanding the Crozet WTP and permitting increased withdrawals from Beaver Creek Reservoir or obtaining it from one of the other alternative sources cited above.

4.4 Evaluation of Alternatives and Recommendations

Alternatives are typically evaluated on the merits of technical feasibility, environmental impacts, and cost to implement though other criteria may be considered as well depending on the priorities of the water authority, served communities, and regulators. In this case, Alternatives 1, 2, 4, and 7 are very likely to prove technically feasible for meeting demands from the present through 2075, though Alternative 7 may not be able to meet an implementation schedule needed to meet the more immediate need to expand the Crozet WTP to satisfy peak day demands that presently approach the rated capacity of the facility. Alternatives 3, 5, and 6 would require further evaluation to develop a more confident opinion of their feasibility for satisfying the additional demand growth projected between now and 2075. However, given the infrequency of tapping large groundwater supplies in the area there are significant doubts about the ability to develop wells with the production rate needed to make Alternative 5 feasible and the size of Lake Albemarle and its watershed makes the technical feasibility of Alternative 6 unlikely. The water conservation alternative (Alternative 8) is not deemed technically feasible as a stand-alone alternative though RWSA already has a strong conservation program which is likely to become part of the selected long-range water supply plan moving forward and paired with a feasible supply expansion alternative.

Alternatives 2 and 7, both of which involve dam building and the inundation of stream reaches and wetlands, are likely to have the highest negative environmental impacts of the identified set of alternatives. They would also likely have among the highest cost to implement of any of the alternatives under consideration and for the combination of these reasons will not be selected as the recommended alternative. The other six alternatives would have more manageable environmental impacts with the possible exception of the groundwater alternative (Alternative 5), which could have highly variable impacts on local and regional hydrology depending on the location, the extraction rate at the well, and the interaction between groundwater and surface waters in the area of the well or well field. The environmental impacts from Alternative 4 would be most acute during construction of the pipeline, but nevertheless would exert some impact on regional hydrology over the full planning period. Alternatives 1,

[4] Maupin et. al., 2014, Estimated Use of Water in the United States in 2010; USGS, Circular 1405, p. 22.

3, and 6 would also have some impact to long-term hydrology as demand grows. Alternative 3 would have some additional impacts from building the pipeline over to the Watts Branch of the Beaver Creek plus potentially some negative impacts on the hydrology in the Watts Branch from the extra flow received and on the Moorman's River, which would see a reduction in flow. However, the magnitude and nature of the impacts would depend upon the operating procedure for the interconnection which has not been developed. Alternative 6 would have both pipeline construction impacts and impacts from building a new water supply intake.

Alternative 4, connecting Crozet to the Charlottesville Urban Water System, is both technically feasible and would not have serious environmental impacts, though constructing the approximately 5 miles of pipeline connecting the systems would have short-term impacts including several stream crossings. However, as mentioned in Section 4.3.4, the major concern here is that it would create development pressure along the pipeline corridor that is counter to the stated wishes of the County.

Based on the technical feasibility and estimated environmental impacts, permitting increased withdrawals from the existing Beaver Creek Reservoir is the leading alternative. Costs have not been developed for the alternatives presented in this section except for Alternative 1. However, among the potentially technically feasible stand-alone alternatives (Alternatives 1 – 5 and 7), the leading candidates for the lowest cost to implement include Alternatives 1, 4, and 5 (if Alternative 5 is technically feasible). Alternatives 4 and 5 have significant drawbacks among the other criteria discussed above. Alternative 3 may have only a marginally higher cost to implement than Alternative 1. Nevertheless, Alternative 1, increasing withdrawals from the Beaver Creek Reservoir, would appear to be the least environmentally damaging, practicable alternative. The Beaver Creek Reservoir is the current supply source for Crozet, is located very near to the community, and modeling has demonstrated that sufficient supply exists to meet the demand growth projected over the planning horizon to 2075. Therefore, based on all the information at hand, Alternative 1 is the recommended alternative for meeting the projected demand growth in the Crozet Service Area through 2075. While this alternative will have construction impacts to address, the greater concern may be the reduction in outflows from the reservoir toward the end of the planning window.

4.5 Conceptual Development of Preferred Alternative

Hydrologic modeling has demonstrated that the water supply pool portion of the Beaver Creek Reservoir can meet the projected demand through 2075. However, successfully utilizing this resource to meet that demand will require upgrading some of the infrastructure used to operate the reservoir. The concept described in Section 4.3.1 for increasing withdrawals from the Beaver Creek Reservoir will require implementing a MIF protocol to assure that the increased withdrawals cause as little hydrologic stress on the lower Beaver Creek and Mechums River as possible while still meeting the water supply needs of the community. As noted in Section 1.1.2, the current infrastructure arrangement creates a challenge to maximizing storage in the reservoir when the reservoir is below full pool because of the need to manually adjust the sluice gates on the outlet riser. Releasing too much water from storage during a severe drought would jeopardize the ability to achieve the required operating yield, whereas releasing too little water could result in pump shutdown on low wet well level and disruption of WTP operations.

One solution to these challenges is to separate the raw water withdrawal facilities from the MIF and other releases. This approach would involve (a) building a dedicated raw water intake and pump station that withdraws water directly from the reservoir and delivers it directly to the WTP, and (b) installing motorized actuators on the spillway riser sluice gates to provide gate modulation for MIF control. In addition to the benefits of decoupling and simplifying the raw water delivery and MIF control functions, energy requirements for raw water pumping would be reduced by 40 to 45 feet of head normally lost at the existing intake. The next sections discuss the development of a new raw water intake and provisions for MIF modulation.

4.5.1 Site Selection for New Raw Water Pump Station

Selecting a site for a new intake and pump station involves balancing a number of factors including the following:

1. Access to deep water.
2. Proximity to the CZWTP.
3. Placing critical infrastructure above flood elevation.
4. Minimizing environmental impact.
5. Minimizing impacts to private property.
6. Aesthetic factors.
7. Minimizing impacts to other uses of the reservoir, including recreation.
8. Minimizing cost to build the intake.

In order to maximize reservoir operating yield, it is necessary to provide access for water withdrawal over the full depth of the water supply pool, from the normal pool EL 538.7 to the top of the sediment pool EL 515.4. For intake design purposes, it is desirable to provide access to storage at the upper levels of the sediment pool to serve as a buffer during a water supply emergency. As shown in Figure 4-6, along the western reservoir bank for a distance of approximately a half mile directly upstream of Beaver Creek Dam, access to deep water (as low as the reservoir bottom at EL 500) typically lies within a hundred to two hundred feet from shore.

Figure 4-6: Topographic Map of Reservoir and Surroundings Near to the Dam

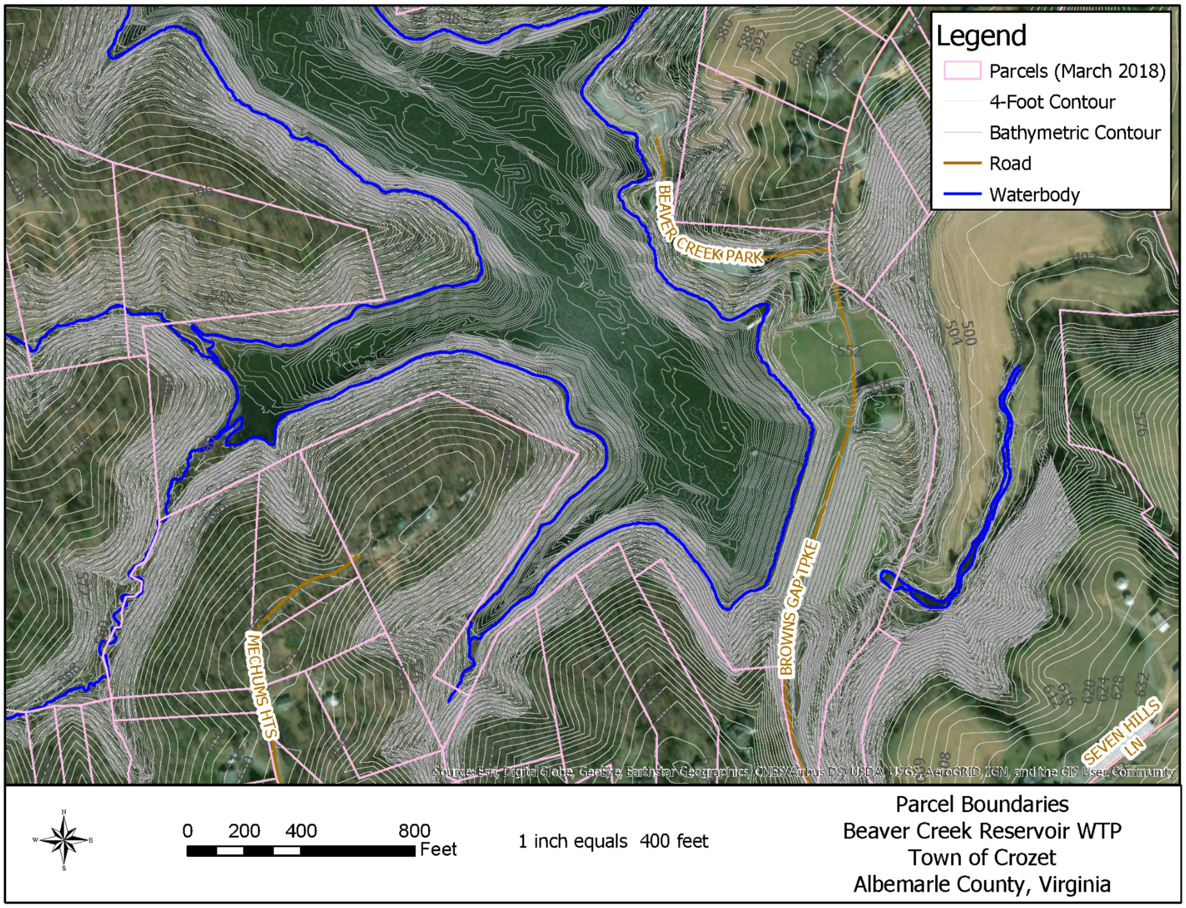


Figure 4-7 shows 5 sites that appear well-situated to satisfy the criteria listed above. All parcels not immediately contacting the edge of the reservoir are privately owned (parcel boundaries shown in pink).

The following is a list of general and site-specific comments associated with the alternative sites under consideration:

- Each site has land situated above the design high inundation level of the reservoir (~557 feet) with a relatively rapid drop off toward the deepest parts of the reservoir which would keep the length of intake pipes out to the reservoir as short as possible.
- Sites 3, 4a, and 4b would require obtaining an easement across private property to provide roadway access to the facility and for locating the raw water line from the pump station to the Crozet WTP as shown in Figure 4-8. Site 1 might require acquiring a small portion of one or two private lots adjacent to the site to keep critical infrastructure above flood level, but this would not adversely affect any buildings or improvements visible via satellite on those parcels.
- Sites along the southern shore of the reservoir are most likely to allow for easily locating a raw water transmission line to the Crozet WTP. Sites north of the dam were excluded from

consideration because they would require a pipeline to be laid across the reservoir or completely around the downstream end of the dam. Pursuing either solution for sites north of the dam would be costlier and inflict greater environmental harmful than utilizing sites on southwestern side of the reservoir. Routing a pipeline from a site on the northeastern side of the reservoir along Brown's Gap Turnpike back across the dam is also not an option because pressurized pipes cannot be safely located within dams.

- Building an intake at Site 2 may require some additional consultation with the dam safety engineer to confirm that the location is suitable, and that construction of the pump station wet well and intake lines would not pose any risk to the integrity of the dam. A desktop analysis indicates it should not be problematic in this respect and the length of road needed for access is the least of the five sites. However, for Site 2 the pump station would be located rather near to the dwelling on the adjacent parcel.

Figure 4-7: Sites under consideration for a new Raw Water Intake and Pump Station

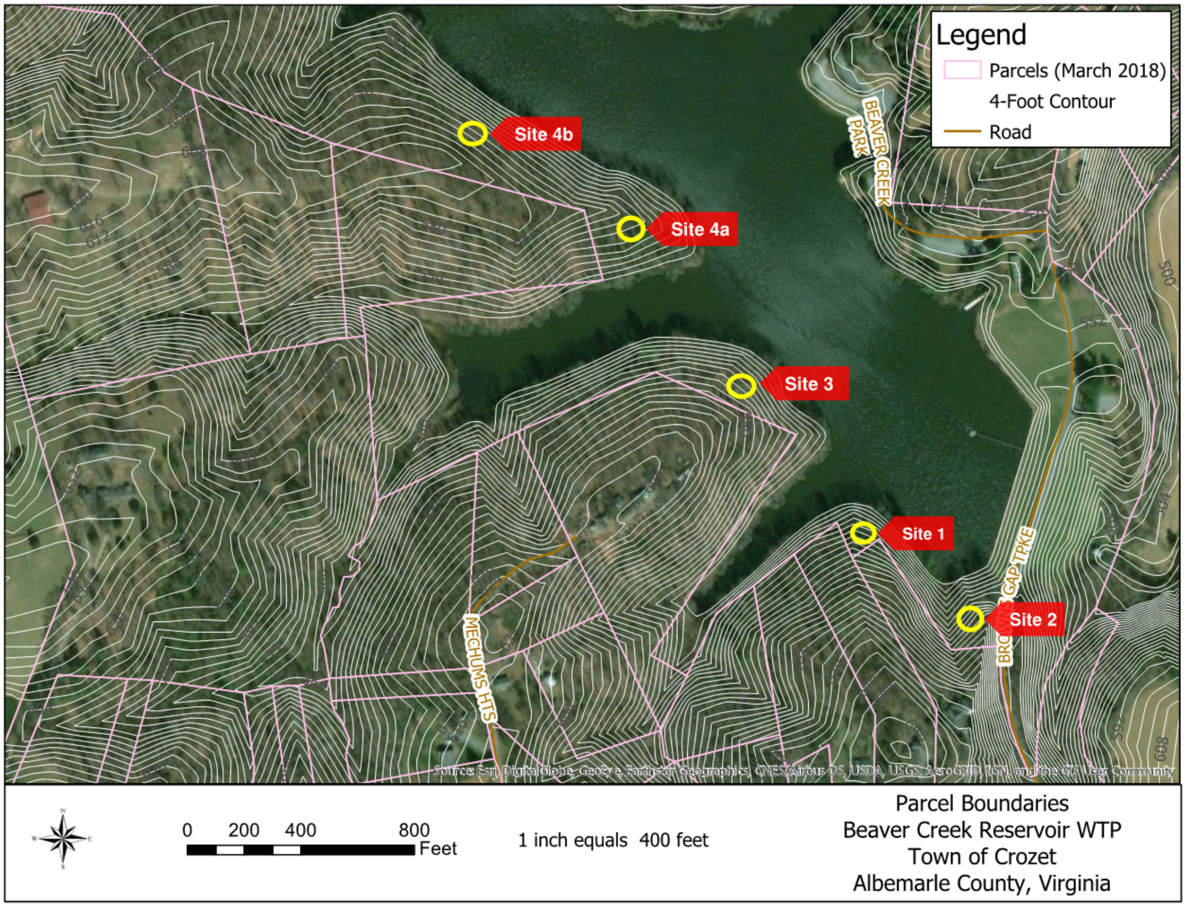


Figure 4-8: Pump Station Sites with Conceptual Roadway/Waterline Easements

Table 4-2 below describes the estimated length of road access needed to be constructed to access the five proposed sites as well as the length of raw water transmission line to convey raw water to the Crozet WTP. Road length is total length from the nearest public roadway to the pump station site.

Table 4-2: Access Road and Pipeline Lengths by Site

Site	Access Roadway Length	Raw Water Pipeline to CZWTP	Easement Needed on Private Property
1	500 ft	4,450 ft	0 ft
2	100 ft	3,950 ft	0 ft
3	1,050 ft	3,350 ft	350 ft
4a	3,650 ft	6,900 ft	2,400 ft
4b	3,150 ft	6,400 ft	2,400 ft

RWSA will review the project objectives with community residents, either in conjunction with or following a meeting on the same subject with the ACSA and the County. Assuming the community is receptive with respect to the general concept and one or more of the potential sites, the next engineering steps would involve site reconnaissance to field verify specific site issues and help narrow site selection. This effort would be following by a desktop/field geological survey and a preliminary geotechnical evaluation, which would provide a preliminary basis for site evaluation and selection and further engineering evaluations.

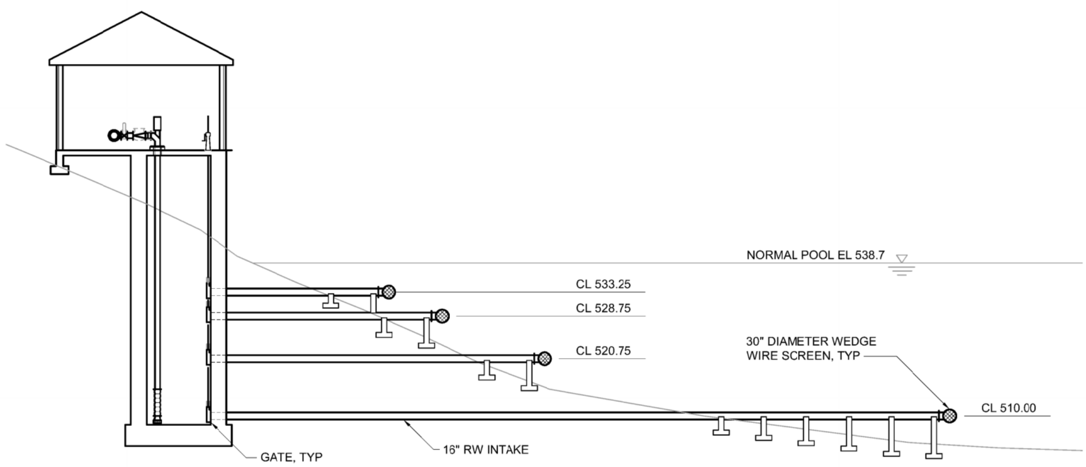
4.5.2 Intake and Pump Station Concept

The need for a dedicated in-reservoir raw water intake and pump station have been described in Sections 1.1.2, 4.3.1., and 4.5. In summary, providing a dedicated intake and pump station will allow for more efficient management of storage in the reservoir, assure that proper pump bowl submergence is maintained, and remove an obstacle to providing a precise MIF downstream of the dam. The intake and pump station concept presented herein involves constructing an on-shore pump station with multiple levels of withdrawal capability that will allow RWSA to draw from a reservoir stratum with desirable water quality characteristics and allow RWSA to access the entire water supply pool volume.

A schematic of one intake and pump station concept is presented in Figure 4-9 which is based on the bathymetry profile at Site 1 (Figure 4-7). Each intake level would be provided with cylindrical wedge-wire tee screens meeting DGIF intake design guidance criteria of 1-mm mesh and a maximum of 0.25 ft/s approach velocity at a 3 mgd maximum withdrawal rate[5]. Screens would need to be 30-inches in diameter for the extended tee dimensions offered commercially to meet this criterion or 36-inch diameter if standard tee screen dimensions are used. It is recommended to include an automated air-burst screen cleaning or other automated screen cleaning method in the intake design.

[5] Gowan, Charles; Garman, Greg; Shuart, Will. Design Criteria for Fish Screens in Virginia: Recommendations Based on A review of Literature, DGIF, 1999

Figure 4-9: Conceptual Intake and Pump Station Profile

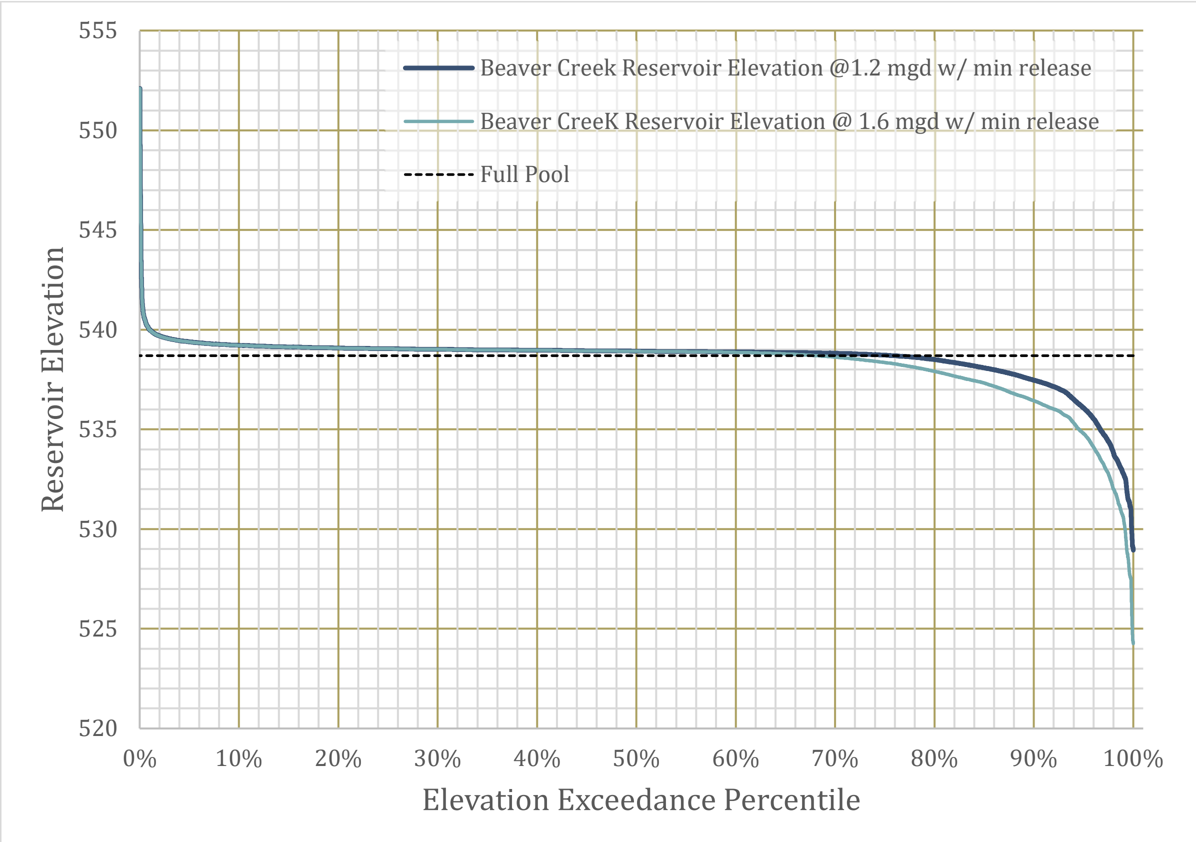


Generally, water quality is better at higher levels within the reservoir during periods of reservoir stratification and therefore the intake levels have been preliminarily set to allow for withdrawals near to the prevailing water surface elevations. Wedge-wire screen intakes are generally designed to operate with one full diameter of submergence from the center-line elevation (1/2 D from the crest) so each intake level was conceptually chosen to operate at water surface elevations as low as 30-inches above the indicated centerline elevation in Figure 4-9.

Figure 4-10 describes the frequency of encountering the full range of reservoir elevations expected at future demand levels of 1.2 mgd and 1.6 mgd. Therefore, even at the 2075 average raw water demand of 1.6 mgd, it is expected that the top intake level would have proper submergence (water elevations > 535.5 ft) 93% of the time. The third intake elevation would have proper submergence 100% of the time based on historical hydrologic conditions at a 1.6 mgd demand level.

The SCS design criteria for the reservoir allocated a volume for sediment accumulation over the life of the project equivalent to the volume in the reservoir below 515.4 feet at the time of design. However, the reality is that sediment is distributed across the full reservoir profile and not just in areas with a bottom depth below 515.4 feet. Therefore, for RWSA to retain access to the full water supply pool volume of about 500 MG over the entire planning period, it is recommended that the lowest intake (fourth level) be able to access water when surface elevations are somewhat below 515.4 feet. Based on evidence indicating a slow accumulation of sediment in the reservoir and the desire to provide RWSA with maximum storage access in the reservoir, the lowest intake is conceptually placed at a centerline of 510 feet. The recent bathymetric study indicating a low rate of sediment accumulation points to an intake at this elevation remaining serviceable and unobstructed by accumulated sediment for the full planning period and likely beyond that time. Nevertheless, specific intake elevations should undergo additional evaluation during the design phase.

Figure 4-10: Reservoir Elevation Exceedance Probability Chart for Future Demands

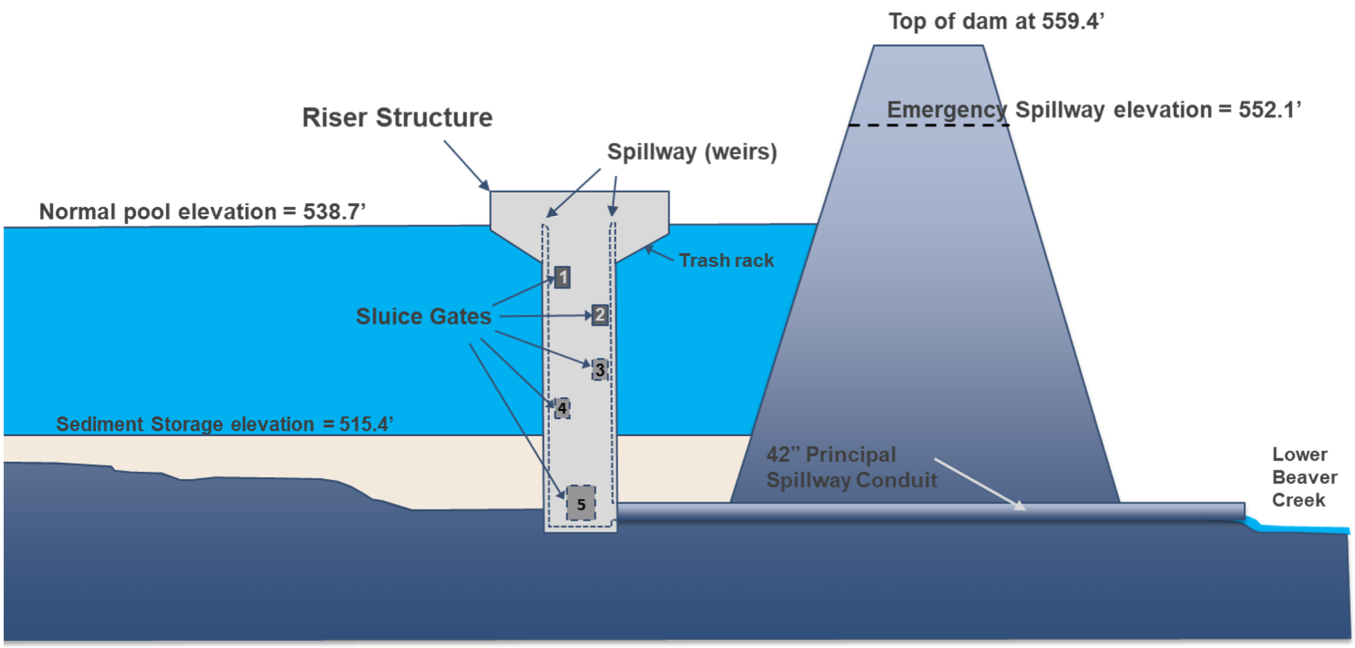


A geotechnical evaluation of the sites described in 4.5.1 is recommended prior to selecting a site and certainly before pump station design commences. It is expected that a good portion of the pump station wet well will require excavating through rock. The raw water intake lines between the intake screens and the pump station wet well may also require excavation or tunneling through rock.

4.5.3 Facility Improvements to make Minimum In-Stream Flow

Once the Beaver Creek Reservoir is brought into the VWP permit system it will be necessary to provide a MIF from the reservoir into the lower Beaver Creek below the dam. The provisions needed to accomplish this will include remote operation of the flow release infrastructure (gates or throttling valves) and an accurate flow meter. The particular arrangement to achieve this is highly dependent upon the design of the dam safety modifications currently under evaluation. If the existing spillway riser remains, then it is likely this could be accomplished by adding motorized sluice gate actuators on gates 1-4 shown in Figure 4-11. Providing PLCs for the gate actuators and control from the Crozet WTP via SCADA will aid in monitoring and control of the MIF.

Figure 4-11: Schematic of Dam, Spillway Riser, and Spillway Conduit



If there is a need to provide provisions for providing a MIF at reservoir elevations below the invert of Gate 4, then it may be necessary to core drill the spillway riser and install a valve or additional sluice gate capable of throttling flow to the MIF criteria.

5. Evaluation of Water Treatment Needs

5.1 Capability of Existing Facility to Meet Projected Growth

With the proposed upgrades outlined in the Expansion and Rehabilitation Project PER, treatment capacity of most Crozet WTP facilities would be 2.1 mgd. The capacity of the GAC Facility is currently 1.0 mgd based on an empty bed contact time of approximately 15 min. The capacity of the finished water pump station will be 3.0 mgd total, with a firm capacity of 1.5 mgd, at the completion of construction.

The demand projections provided in Section 3 predict maximum day water demands will exceed 2.0 mgd by the year 2045 and reach 2.5 mgd by the year 2075. Additional expansion and upgrades to the Crozet WTP will be required to provide the expected buildout water demands for the Crozet Service Area. This section of the report describes the evaluation of the existing unit processes and recommended facility upgrades to be implemented over time to meet the increasing finished water demands.

5.2 Evaluation of Unit Processes and Related Facilities

The discussion below involves a comparison of the current treatment unit capacity to the capacity required at buildout (i.e., 2.75 mgd). The 2.75 mgd treatment rate is based on the maximum day demand projection in the year 2075, 2.5 mgd, with an additional 10% for in-plant uses (backwash water supply, carrier water, etc.). Given the modular nature of the unit processes at the plant, and the per-unit capacity as planned for in the current expansion and upgrade, it is recommended that one final upgrade and expansion be performed when the plant capacity nears the 2 mgd threshold in the year 2045. The discussion of expansion concepts below was developed with that in mind.

5.2.1 Rapid Mixing and Flocculation

The current plant expansion PER recommends maintaining the existing, single rapid mix basin and replacing the rapid mixer itself to meet mixing needs for a 2 mgd plant flowrate. At the future flow rate of 2.75 mgd, the existing rapid mix basin would have a hydraulic detention time of 39 seconds, which meets the minimum of 10 seconds required by VDH. Given that the volume of the rapid mix basin will not change, the mixer being installed under the current plant expansion will be sufficient for future flows. The primary issue will be to evaluate the condition of the mixer, as by the year 2045, it will likely need to be replaced.

Regarding the flocculation basin capacity, the current expansion will reconfigure the structure to provide two PAC contact basins followed by a second rapid mixing basin and then a single train of three-stage, tapered energy flocculation. The current PER notes that the proposed configuration will provide 23 minutes of detention time at 2.1 mgd; however, this appears to be based on a basin side water depth of 14 feet. The existing basins operate at a side water depth of 12 feet, which means the 14 foot operating depth assumption would not allow for any freeboard in the structures. Table 5-1 compares detention times presented in the PER for the rapid mix, PAC contact, and flocculation basins at a sidewater depth of 14 feet with detention times based on a 12 ft sidewater depth.

Table 5-1: Hydraulic Detention Times for Flash Mix, PAC Contact, and Flocculation Basins

Process Unit	Hydraulic Detention Time at 2.1 mgd (min)	
	HRT from PER (based on 14 ft. sidewater depth)	HRT based on 12 ft. sidewater depth
Flash Mix Chamber 1	2.2	0.8*
PAC Contact Basin 1	7.8	6.8
PAC Contact Basin 2	7.8	6.8
Flash Mix Chamber 2	2.2	1.9
Flocculation Basin 1	7.8	6.8
Flocculation Basin 2	7.8	6.8
Flocculation Basin 3	7.8	6.8

* Assumes existing flash mix chamber is not re-configured

The re-configured flocculation basins would provide only 15.8 minutes hydraulic detention time at 2.75 mgd, which does not comply with the 20 minute detention time required by VDH. Therefore, the PAC contact and flocculation basins would need to re-configured in the expansion from 2.1 mgd to 2.75 mgd to provide the required flocculation detention time.

Table 5-2: Modified Hydraulic Detention Times for Flash Mix, PAC Contact, and Flocculation Basins

Process Unit	Hydraulic Detention Time at 2.75 mgd	
Flash Mix Chamber 1	39 sec*	Exceeds 10 sec minimum
PAC Contact Basin 1	5.3 min	
Flash Mix Chamber 2	90 sec	Exceeds 10 sec minimum
Flocculation Basin 1	5.3 min	
Flocculation Basin 2	5.3 min	
Flocculation Basin 3	5.3 min	
Flocculation Basin 4	5.3 min	

* Assumes existing flash mix chamber is not re-configured

5.2.2 Sedimentation

An additional sedimentation basin with inclined plates is needed to treat 2.75 mgd. The new basin would be located to the west of the existing basins as shown on Figure 5-1. The basin should be equipped with inclined plates with equivalent projected area to the existing basins. With all basins in service the loading rate would be lower than the existing rate, providing for enhanced treatment. With one basin out of service, the firm capacity would be 2.1 mgd at a loading rate of 0.2 gpm/sf.

5.2.3 Filtration

One additional filter is needed to treat 2.75 mgd. The new filter should have similar area as the existing filters to maintain consistent backwash rates across all filters. With 3 filters operating at 2.75 mgd, the filter loading rate would be 3.4 gpm/sf. With one filter out of service for backwshing, the firm filtration capacity would be 2.1 mgd (with the filters operating at 4.0 gpm/sf).

5.2.4 GAC Adsorption

An expansion of the GAC Facility may be required to ensure sufficient NOM removal and DBP control at 2.75 mgd. The existing site configuration cannot accommodate an expansion of the GAC Facility. RWSA would need to utilize the portion of the site currently occupied by ACSA for expansion of the GAC Facility. In the agreement between RWSA and ACSA, RWSA could provide 5-year notice that the site is needed for water treatment infrastructure and ACSA would vacate the property. Figure 5-1 illustrates an additional GAC Facility located to the south of the existing facility. The GAC Facility is sized to accommodate 3 pressure vessel type contactors to treat up to 2.5 mgd at a design empty contact time of approximately 15 minutes. Under most flow conditions, this facility could provide full GAC treatment. At the maximum capacity of 2.75 mgd, blending of the filtered water would be required. RWSA intends to utilize a blending approach for the existing GAC Facility and will have a feel for the level of GAC treatment required well before the Crozet WTP needs to expand beyond 2.1 mgd.

5.2.5 Finished Water Pumping and Storage

The current finished water pump station design was laid out to accommodate installation of a third pump in the future. An additional 1.5-mgd pump would provide a total nominal pumping capacity of 3.0 mgd to meet future needs.

At a future flow of 2.75 mgd, the existing 500,000 gallon clearwell can meet disinfection CT requirements assuming the clearwell is at least 50 percent full and a minimum chlorine residual is 1.5 mg/L is provided. Equalization volume would be less than the desired for a 2.75 mgd WTP given typical goals for 25% of finished water production provided as storage. However, the available volume in the Buck's Elbow elevated storage tank is anticipated to provide sufficient storage for equalization and emergencies in conjunction with the ground storage at the WTP.

5.2.6 Chemical Systems

At a future flow of 2.75 mgd, additional bulk storage would be required for sodium hypochlorite. The design intent for the GAC Facility was to replace the day tank in the future with a new tank of at least

2,000 gallons. Storage for the fluoride and corrosion inhibitor systems is sufficient for the future WTP production. We understand that the new liquid lime and PAC systems being constructed are sufficient to meet future needs at 2.75 mgd.

5.2.7 Residuals Handling

The sizing of the equalization tank proposed for the expansion to 2 mgd is based on process residuals volumes of 35,000 gallons per filter backwash (2 filters), 5,400 gpd settled solids from the Trac-Vacs, and allowances for basin drains.

Settled solids estimates were developed for the Crozet WTP based on the assumptions for average and maximum water quality conditions shown in Table 5-3. Settled solids volumes will be higher at the buildout capacity of 2.75 mgd. Based on the proposed waste equalization tank volume of 105,000 gallons, an adjustment of sludge pumping rate or operational practices may be needed to ensure peak sludge production is adequately addressed,.

Table 5-3: Crozet WTP Solids Production Estimates

	2.1 mgd WTP Capacity		2.75 mgd WTP Capacity	
Treatment Parameter	Average Values	Max Values	Average Values	Max Values
Raw Water Turbidity (NTU)	5	10	5	10
Alum Dose (mg/L)	45	65	45	65
PAC Dose (mg/L)	5	15	5	15
Total Solids Generated (lbs/day)	575	1,045	740	1,340
Total Settled Solids Volume (gals/day)	13,800	25,000	17,700	32,200

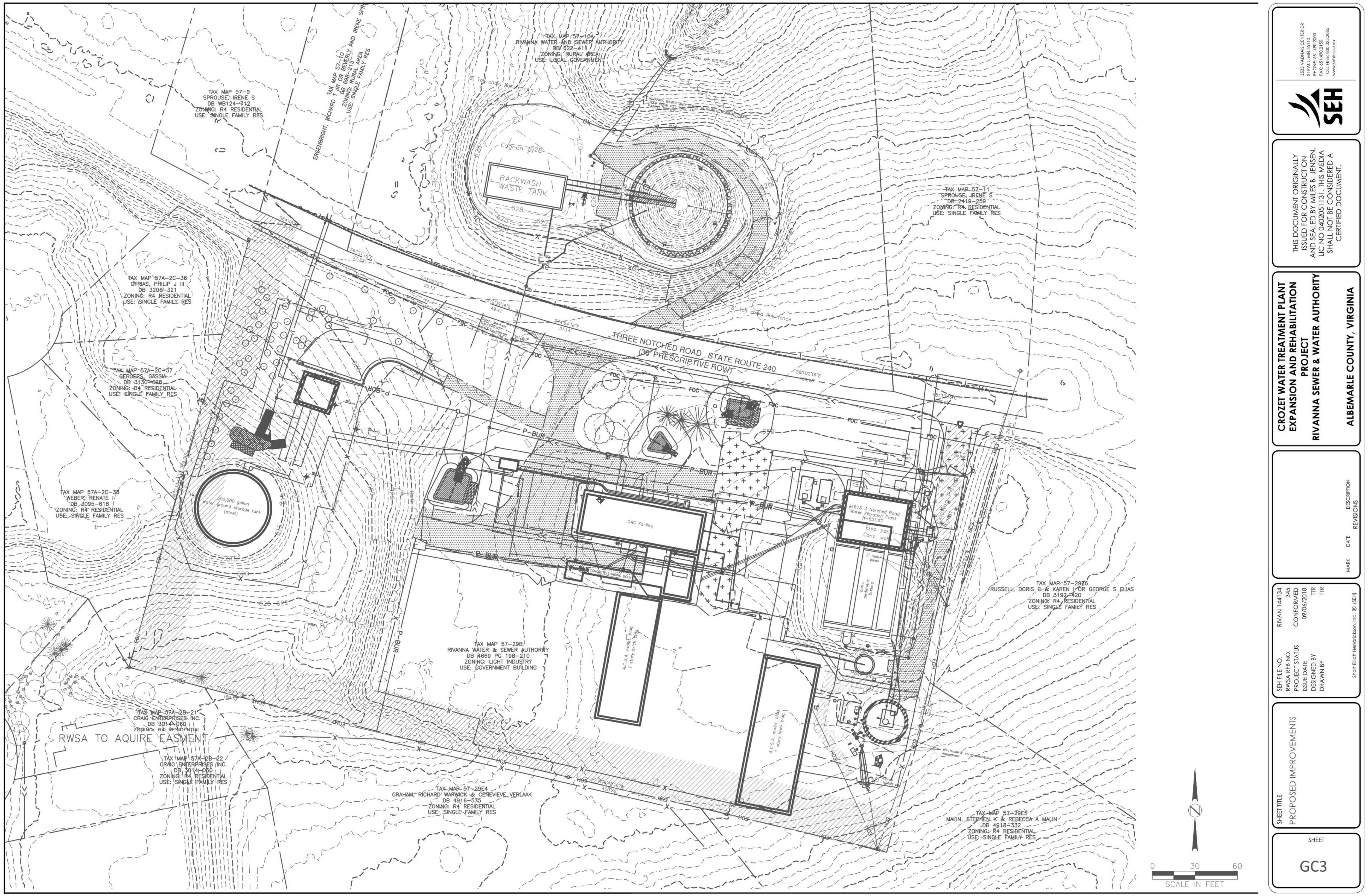
5.3 Recommendations

At 2.75 mgd, an additional flocculation basin is needed to meet VDH requirements. One of the new PAC contact basins should be re-configured to provide sufficient detention time required. Flash Mix Chamber No. 2 would need to be re-configured upstream of the converted PAC contact tank.

One additional sedimentation basin with inclined plates located to the west of the existing basins would be required at 2.75 mgd. An additional filter is needed to meet production needs at acceptable filter loading rates.

A portion of the site utilized by ACSA would be required to expand the GAC Facility if it is needed for NOM removal and DBP control. The proposed GAC Facility is sized to accommodate 3 pressure vessel type contactors to treat up to 2.5 mgd. It is anticipated that RWSA will have a better understanding of the GAC usage to meet Stage 2 DBP rules well before the Crozet WTP needs to expand beyond 2.1 mgd. This information will inform the level of GAC treatment needed at the future capacity of 2.75 mgd.

Figure 5-1: Proposed Expanded Crozet WTP Site Plan



As noted in Section 5.2.5, an additional pump will need to be added to the finished water pump station to meet future needs. From a chemical feed perspective, as noted in Section 5.2.6, additional bulk storage for sodium hypochlorite will be needed for the ultimate plant expansion.

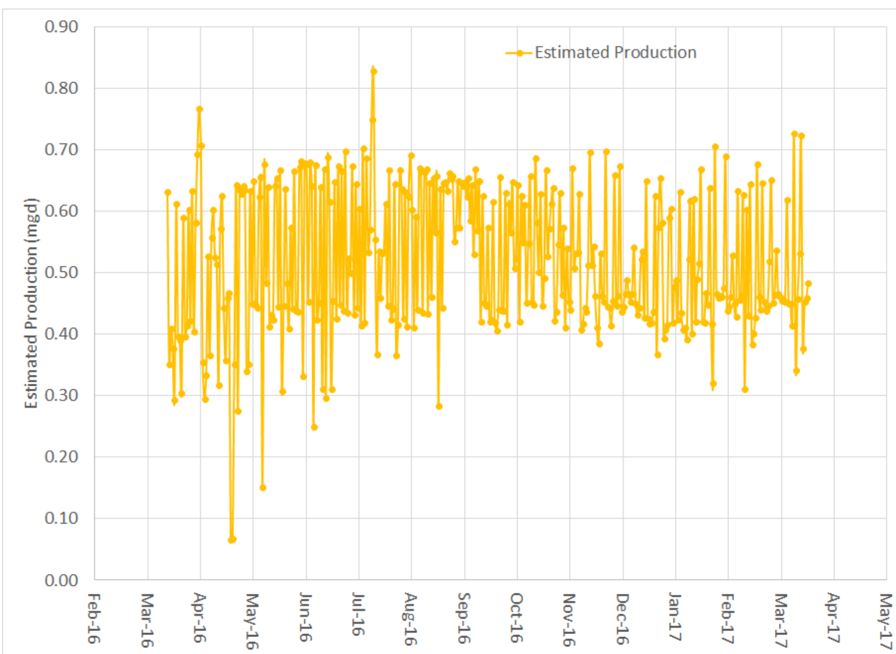
6. Evaluation of Water Distribution Needs

6.1 Hydraulic Model Update and Calibration

ACSA built the existing Crozet water distribution system model from GIS and assigned demands to the model nodes from customer billing records. The model included all ACSA and RWSA distribution system pipes, hydrants, and the Buck's Elbow Tank. ACSA computed an annual average water usage for each customer using billing records from April 1, 2016 to March 31, 2017. The billing totaled 0.45 mgd for this time period. The model from ACSA also included 12-month average flushing rates for two automatic flushers, one on Thurston Drive and one on Hillsboro Lane.

The estimated average production from that same time period was 0.52 mgd. Non-revenue water was therefore 0.07 mgd (14%). A demand scaling pattern was used in the model to increase water demand to match production records by assuming non-revenue water was proportional to the billed consumption assigned to each demand node. Average estimated production is shown in Figure 6-1, which also illustrates the maximum day demand for the study period of 0.83 mgd.

Figure 6-1: Estimated Crozet WTP Production; April 1, 2016 through March 31, 2017



Hazen further updated the hydraulic model by adding performance curves for the high service pumps at the water treatment plant. One set of curves represented the high service pumps that were operational during the calibration period (spring 2018). These pumps were used for calibrating the model only. The other set of curves simulated the new pumps that RWSA installed in the summer of 2018. These pumps were used for all system evaluation purposes beyond the calibration of the model.

For the old pumps, pump curves were not available from the manufacturer or from previous testing. Therefore, the first step for the calibration process was to conduct a draw-down test to estimate pump operating points during the calibration period. During the draw-down test, the finished water storage tank was isolated from the treatment train such that changes in tank levels reflected high service pumping rates only.

A summary of the pump draw-down test calculations is shown in Appendix B. Table 6-1 summarizes the measured operating points. Pumps 1 and 2 are identical. Pump 3 is larger. The generic pump curves developed from these points were used only for model calibration purposes.

A second set of performance curves represented the two new high service pumps (installed summer 2018). The design curves for these pumps are shown in Figure 6-2. These curves were used to evaluate the distribution system under current operations as well as future conditions.

Table 6-1: Old CZFWPS Draw-Down Pump Test Results

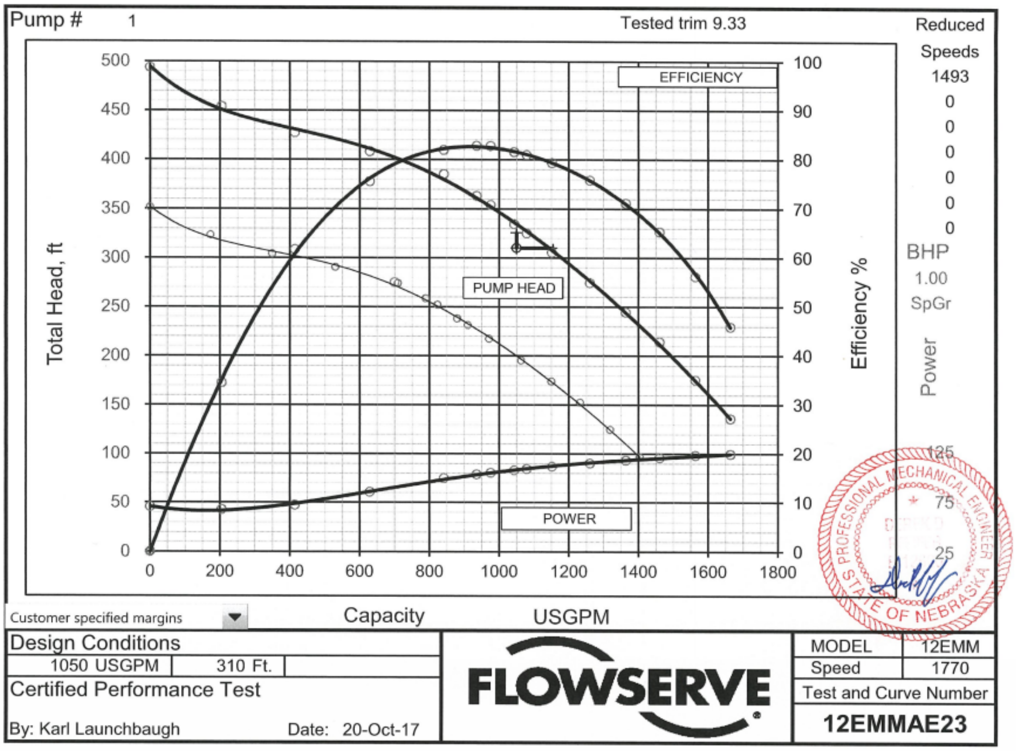
	Pump 1		Pumps 1 + 2		Pump 3	
	Flow (gpm)	TDH (ft)	Flow (gpm)	TDH (ft)	Flow (gpm)	TDH (ft)
Average	497	259	990	278	1118	281

Calibration efforts also included three loss of head tests, one hydraulic grade line test, and four fire flow tests. SCADA tank levels were compared to model predictions to calibrate extended period simulations.

The results of the loss-of-head tests and calculated roughness coefficients (C factors) are shown in Table 6-2. Locations of the three tests are shown in Figure 6-3. The asbestos cement (AC) pipes, which had been in service more than 50 years, had an average C-factor of 127, so the model used this value for any pipe where GIS indicated pipe material was AC. The ductile iron (DI) pipe on Thurston Drive had a roughness coefficient of 149. This is within the range expected for ductile iron pipe (125-155). This particular pipe was assigned a C-factor of 149, but where GIS indicated DI for other pipes, we conservatively assigned a C-factor of 140.

According to ACSA staff, nearly all of the unlined cast iron pipe in the system has been replaced or is scheduled to be replaced soon. Unlined pipe can have significantly lower C-factors, greatly reducing hydraulic capacity. Model calibration assumed that all unlined cast iron pipes indicated in GIS would have a C-factor of 140 because of near-term replacement plans.

Figure 6-2: New High Service Pump Curves



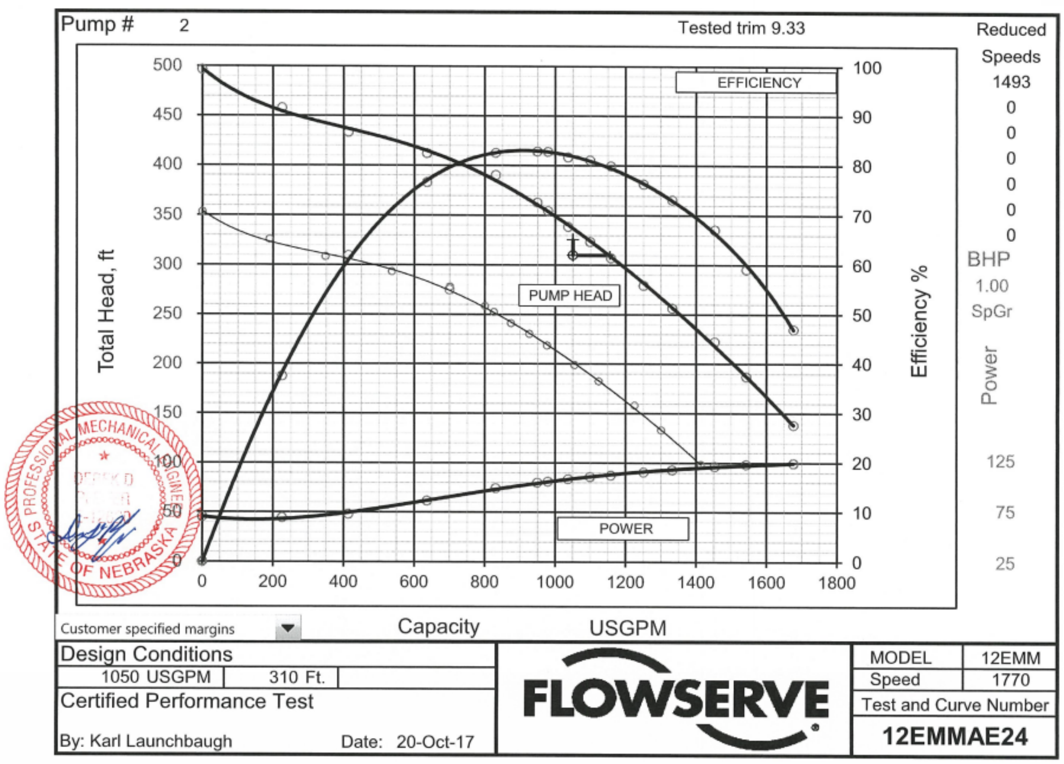


Table 6-2: Loss of Head Test Results

Location		Year	Type	Size	Length ft	Flow mgd	Loss ft	C-Factor
Thurston Drive -	White Hall Rd to 1737 Thurston Dr	1977	DI	8	2,400	1.50	36.9	149
Hillsboro Lane -	Rockfich Gap Tnp to Yancey Mill Ln	1965	AC	6	1,640	0.93	57.6	126
Three Notch'd Rd -	4720 Three Notch'd to 4790 Three Notch'd	1960	AC	12	768	1.61	2.46*	128

* Head loss measured using parallel hose and differential pressure recorder

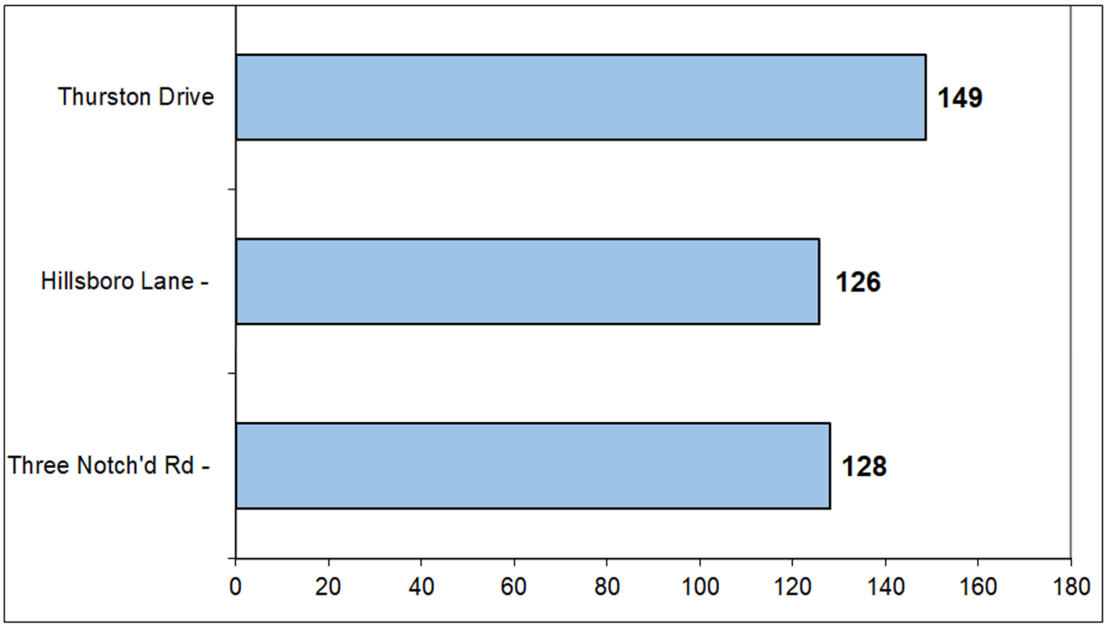


Figure 6-3: Loss of Head Test Locations

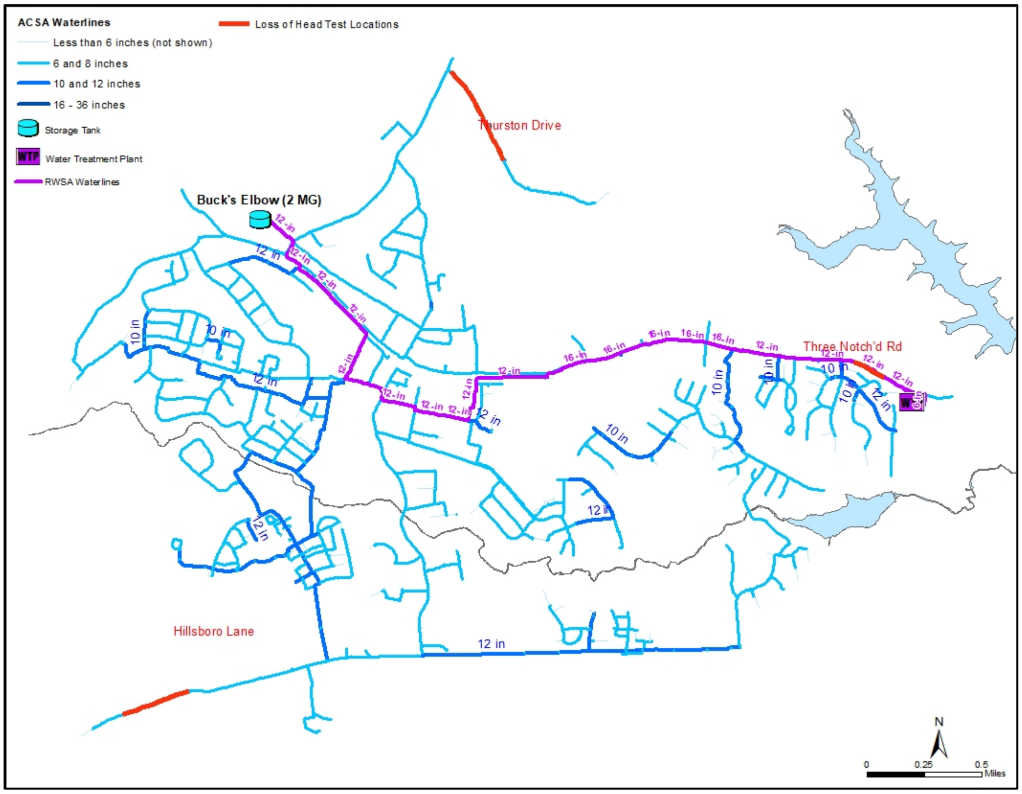


Figure 6-4 shows the location and Figure 6-5 shows the results of the hydraulic grade line (HGL) test. Model predictions of flow rate matched within 5% and HGLs matched within 3 feet with relatively minor adjustments in the model. The HGL test did not show any unexpectedly high head losses along the major transmission mains of the system.

Fire flow tests were used to check localized calibration in the model. The four test locations are shown in Figure 6-6, and Table 6-3 shows the test results. Static pressures in the model matched those measured in the field within 1 psi, and residual pressures predicted by the model were within 2 psi of those measured. However, to match the residual pressures from Test 1 and Test 2, the model assumed closed valves because measured pressures were much lower than predicted when the model assumed all valves were open.

For Test 1, the model predicted pressure was within 1 psi of the measured residual pressure by assuming a valve was closed on the 8-inch pipe along Crozet Ave between Buck Road and Ballard Drive. Alternatively, assuming a valve is closed on the 8-inch along Buck Road between Crozet Ave and St. George Ave also achieves a pressure drop that approaches what was measured.

Similarly, for Test 2, the model matched the measured residual pressure by assuming a valve was closed on Grayrock Drive (8-inch) north of Jarmans Gap Rd.

Both locations were inspected by ACSA to identify closed valves; however, none were reported to be found. While autoflushers were also in-place and active during the time of the tests, those units were incorporated into the modeling analysis and are not considered to be related to the pressure discrepancy.

Further assessment is recommended to identify the cause of the hydraulic restriction suggested by the modeling and field testing results.

Figure 6-4: HGL Test Paths

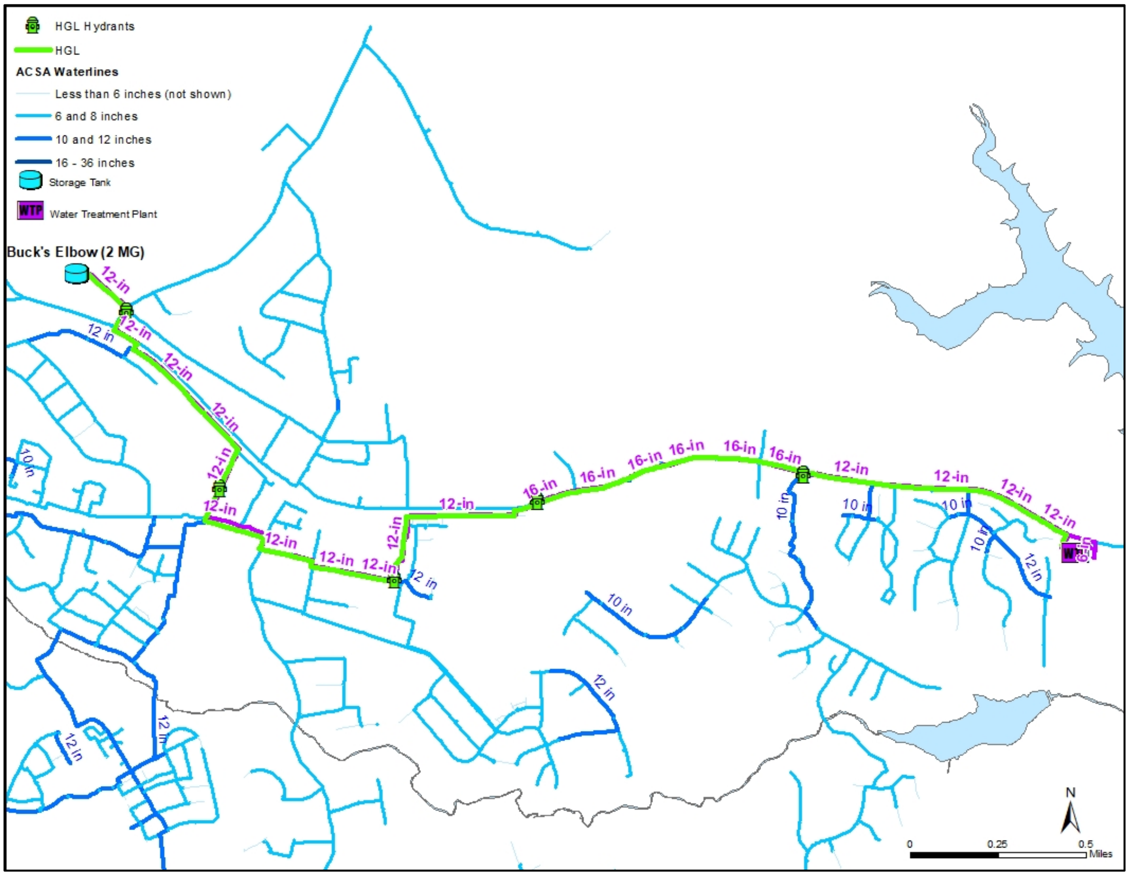


Figure 6-5: HGL Test Results

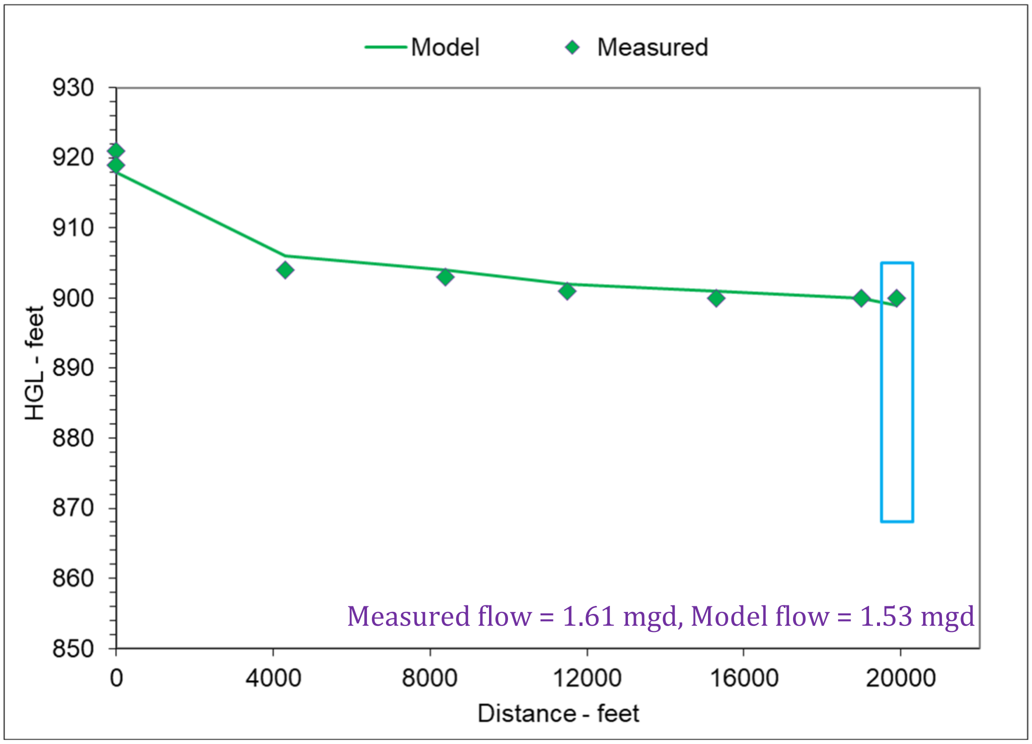


Figure 6-6: Fire Flow Test Locations

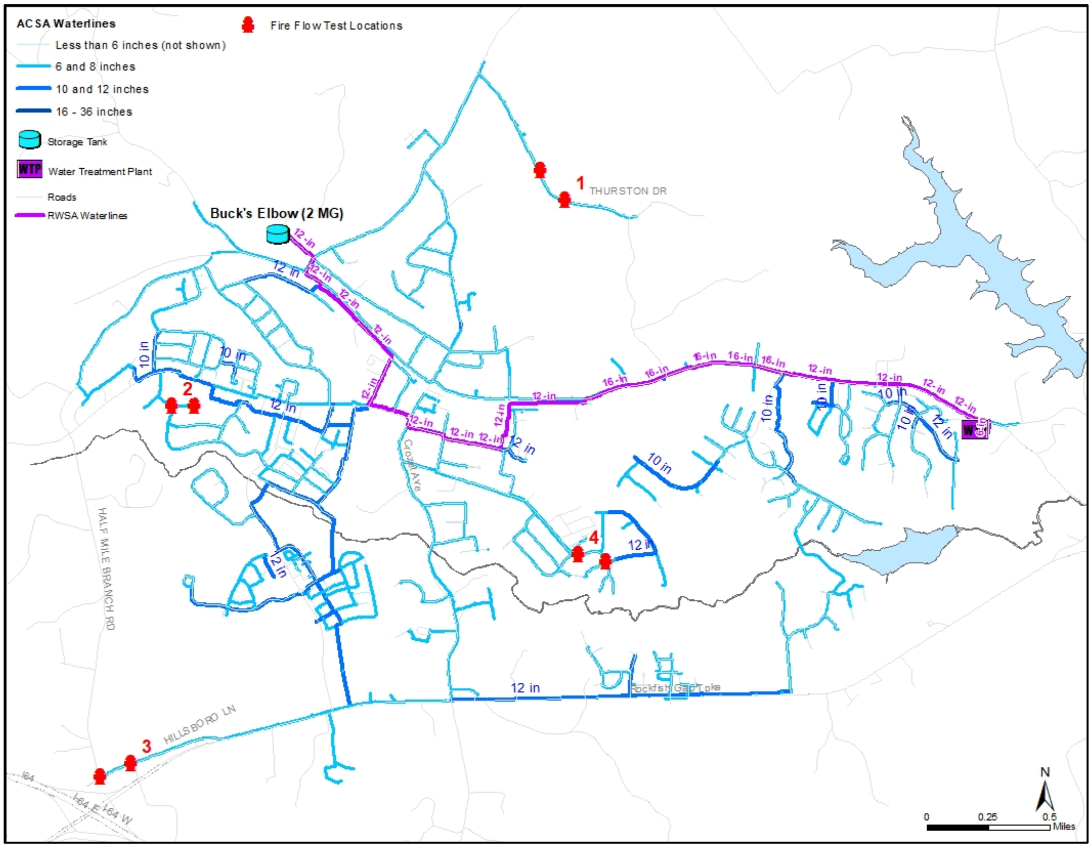


Table 6-3: Fire Flow Test Results

Test No. and Location		Static Pressure		Residual Pressure		Test Flow	Flow at 20 psi	
		Measured	Model	Measured	Model	gpm	Test	Model
		psi	psi	psi	psi		gpm	gpm
1	1737 Thurston Dr	92	93	46	45*	920	1,200	1,200
2	Birchwood Hill & Wellbourne	64	64	55	56*	1,120	2,600	2,800
3	Hillsboro & Yancey Mill	72	72	26	28	640	700	700
4	Park & Brookwood	111	112	100	100	920	2,900	2,800

* Assuming closed valves

The final step of calibration was to compare Buck's Elbow Tank levels recorded by SCADA to predictions from an extended period simulation (EPS). A diurnal curve was calculated to reflect daily fluctuations in demand using the estimated flows from the pump station draw-down test and the change in volume of Buck's Elbow Tank. The diurnal curve is shown in Figure 6-7. The peak hour of the day is 160% of the average and the minimum night rate is 35% of the average.

Figure 6-7: Crozet Diurnal Curve Developed from April 18, 2018 Flow and Tank Levels

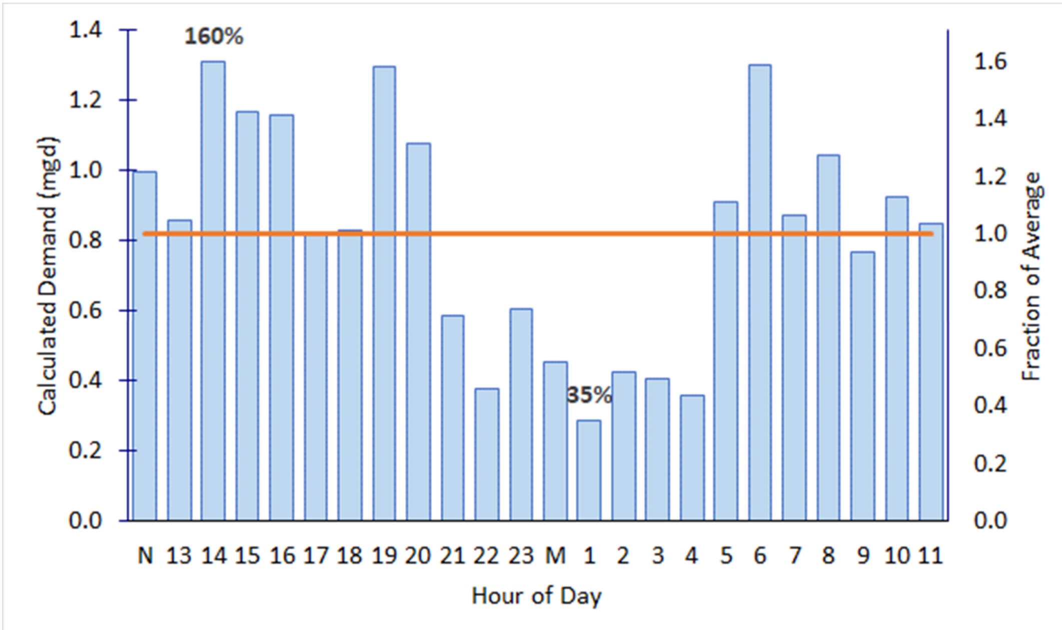
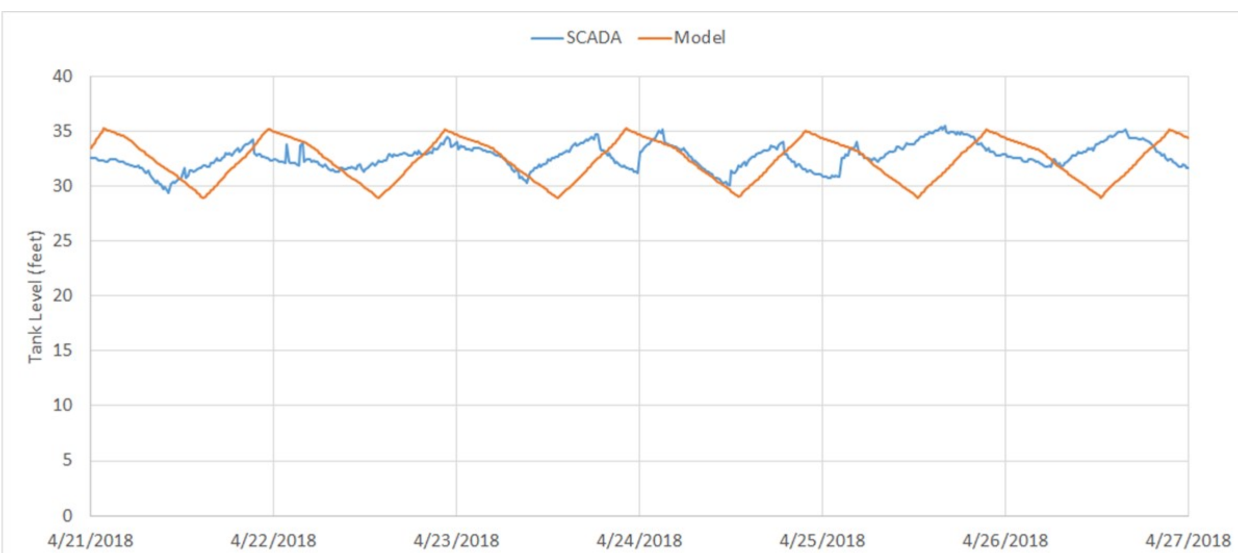


Figure 6-8 shows the EPS comparison between SCADA readings and model results. For an EPS in the model, controls were set on the high service pumps to reflect the general range and frequency of Buck's Elbow Tank cycles recorded by SCADA. This included running Pump 1 when Buck's Elbow Tank reached 29 feet and turning off when the tank level reached 35 feet.

Figure 6-8: Extended Period Simulation Model Calibration



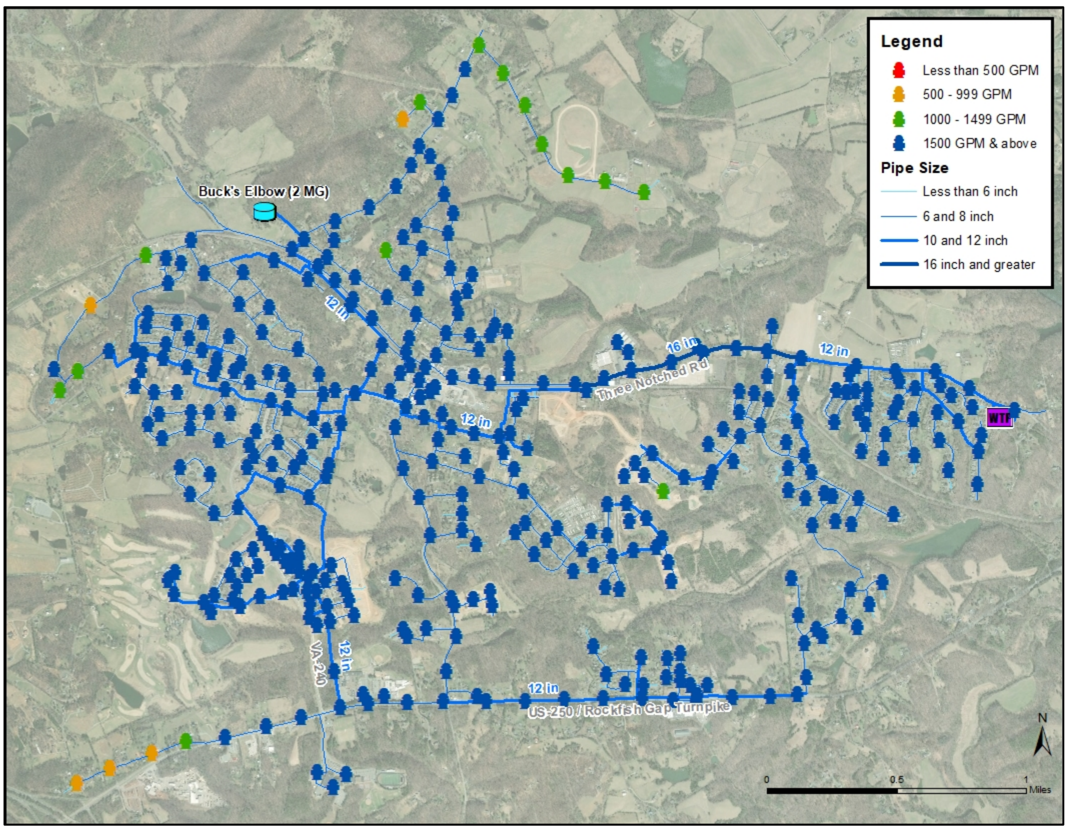
6.2 Existing Water Distribution System Evaluation

To evaluate the current distribution system, the model checked pressures, flows, and water age. Design criteria employed for this evaluation included the following:

- lower pressure limit of 20 psi during fire flows
- lower pressure limit of 30 psi for the peak hour demand of maximum day demand
- upper pressure limit of 150 psi during average day demand
- upper velocity limit of 5 feet per second to prevent excessive head loss
- water age less than 14 days to maintain chlorine residual and prevent water quality complaints

A map showing available fire flows is provided in Figure 6-9. These fire flows shown on the map are the flows the water system can deliver to the hydrant connection at a residual pressure of 20 psi. These results assume all unlined cast iron pipes are replaced with new pipes of the same size. Hydrant losses generally prohibit fire flows in excess of 1,500 gpm, so multiple hydrants may be needed to achieve higher flows.

Figure 6-9: Available Fire Flow



The existing distribution system was evaluated by comparing available fire flows to our estimates of needed fire flows. Needed fire flows were based on ISO requirements for Crozet, the type of structures near each hydrant, and guidelines from the American Water Works Association (AWWA) Manual of Practice on Fire Protection (M31). Appendix D includes estimated needed fire flows at every hydrant.

Figure 6-10 shows which hydrants have deficiencies relative to the above standards and requirements for the type of structures.

Figure 6-11 shows peak hour pressures. Peak hour pressures represent the lowest pressure during the peak demand hour of the maximum day with the storage tank at its lowest typical operating level. Only one location falls below the 30 psi design criteria on the north dead-end on Mint Springs Road, where the predicted pressure is 29.5 psi.

Many customers on the west side of the distribution system have pressures between 30 and 40 psi. These pressures are limited by the range and overflow elevation of the Buck's Elbow Tank. Alternatives for increasing pressures include:

- a new pressure zone with a booster pump and hydropneumatic tank
- a new pressure zone with a booster pump and new elevated tank with higher overflow elevation
- raise the overflow of the existing Buck's Elbow Tank
- replace Buck's Elbow Tank completely with a new smaller tank with higher overflow elevation

Figure 6-10: Deficient Fire Flows for Existing Distribution System

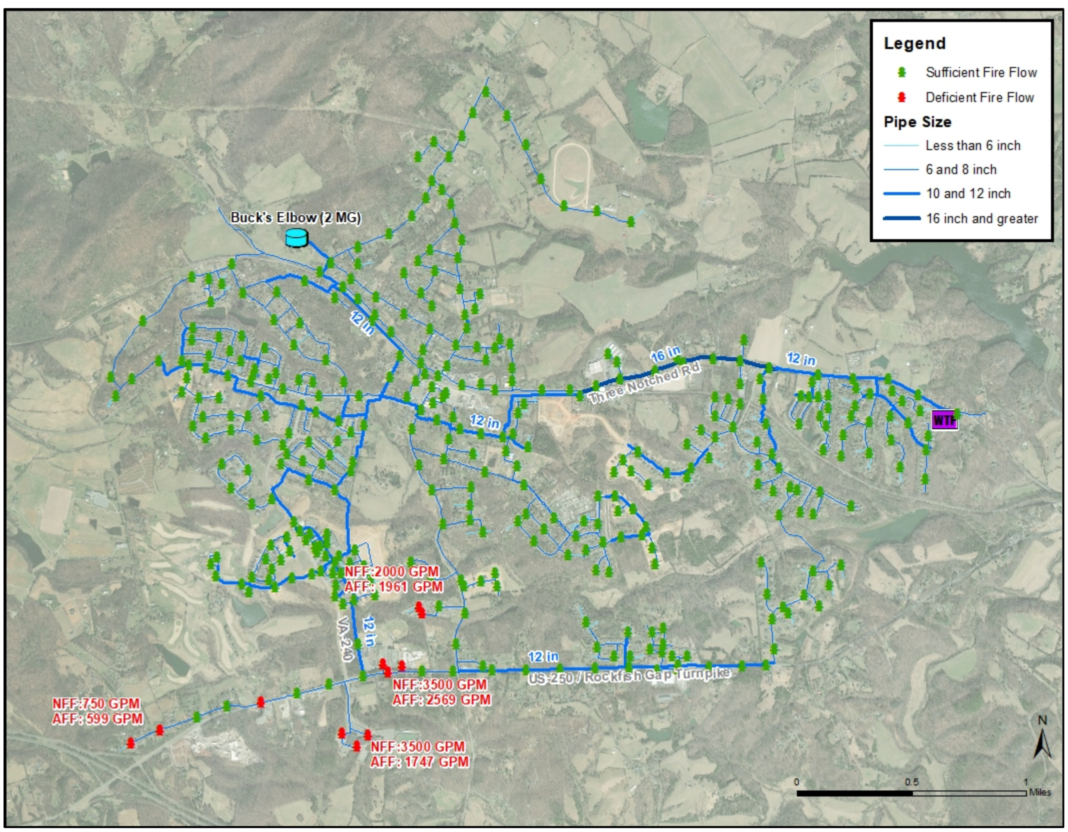
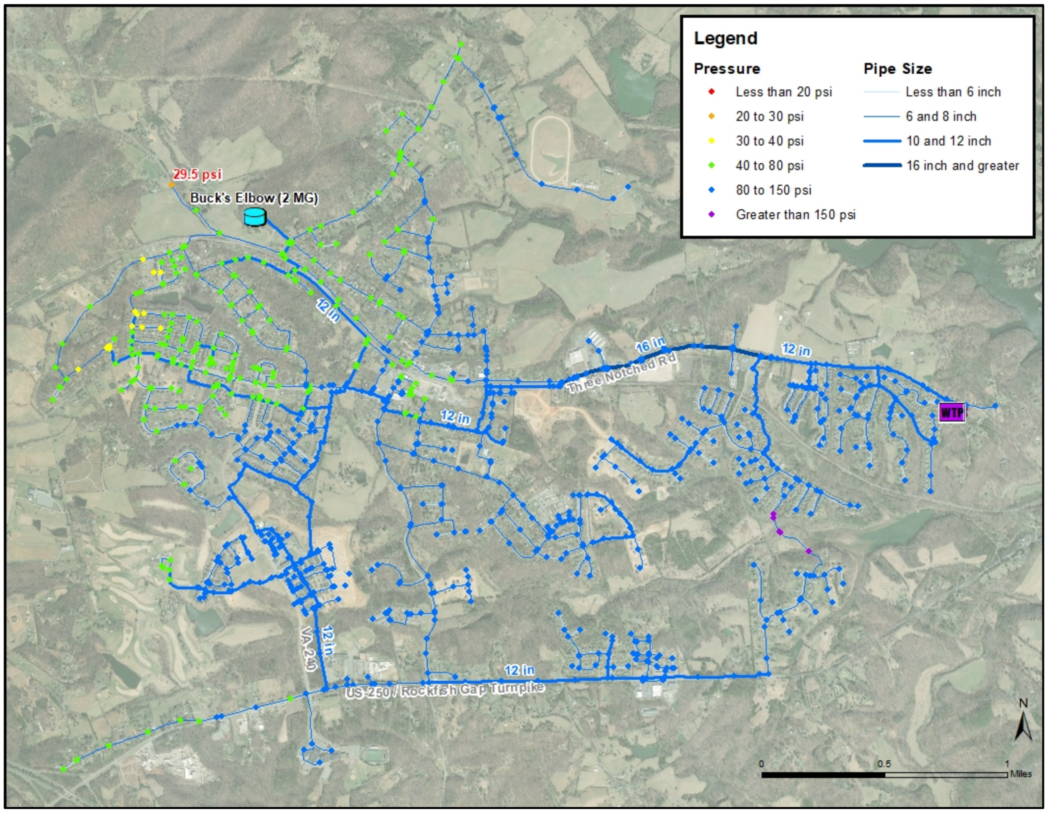


Figure 6-11: Peak Hour Pressures for Existing Distribution System



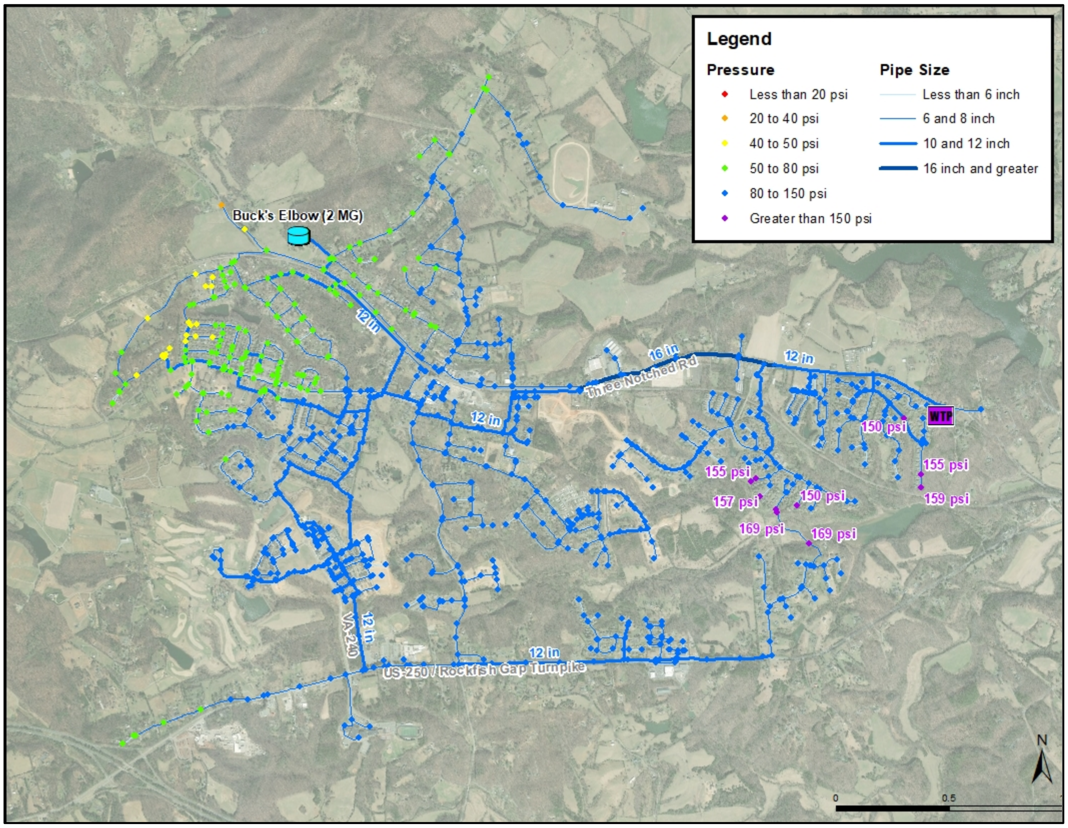
There are advantages and disadvantages to each of these options. Replacing Buck's Elbow Tank with a smaller tank, for example, would improve water quality, but a higher overflow elevation would cause excessive pressures on the east side of the system. After discussion with RWSA and ACSA staff, the design criterion of 30 psi was maintained, and these alternatives were not investigated further.

Figure 6-12 shows maximum pressures during average day demands. A few areas close to Lickinghole Creek, including the south side of Highlands and Western Ridge and north side of Foxchase experience pressures over 150 psi (as high as 169 psi near the creek crossing). All other locations remain below the high pressure design criterion. Pipes in this area are all ductile iron except for the segment under the creek which is HDPE (high-density polyethylene).

ACSA has a standard for ductile iron pipe whereby the thickness must be a minimum of 0.31 inches for pipes 8 inches and smaller or 0.34 inches for pipes 10 inches and larger (manufactured in accordance with AWWA/ANSI C151/A21.51). The working pressure for ductile iron pipes of this thickness is 350 psi, so pressures over 169 psi would be well within the rated working pressure. For HDPE, the standard for ACSA installations is DR 11 which is rated for a working pressure of 200 psi.

Assuming there are not surge pressures more than 100 psi higher than the rated working pressures for the ductile iron and HDPE pipes in the area, acute incidence of failure is unlikely. The existing pressures are also well under the rated working pressures of the pipes; therefore, we do not recommend any improvements to address high pressures.

Figure 6-12: Maximum Pressures during Average Day Demand



Water age is shown in Figure 6-13. The average water age in the Buck's Elbow Tank as modeled is approximately 23 days, and the area southwest of the Buck's Elbow Tank shows water age in excess of the 2-week design criterion. Figure 6-14 shows the area of influence of the Buck's Elbow Tank which significantly increases water age and has the potential to create water quality problems. It should be noted that the model was based on reported tank level controls and finished water pumping operations at the time of the modeling effort; RWSA has since made changes to the operation of the finished water pumps and Buck's Elbow Tank to address low chlorine residuals and water age. In addition, subsequent to the modeling effort, the Grayrock autoflusher was installed and operated on a limited basis to further reduce water age and improve chlorine residuals. Figure 6-13 depicts the locations of the permanent and temporary autoflushing devices.

Figure 6-13: Average Water Age for Existing Distribution System with Modeled Operations

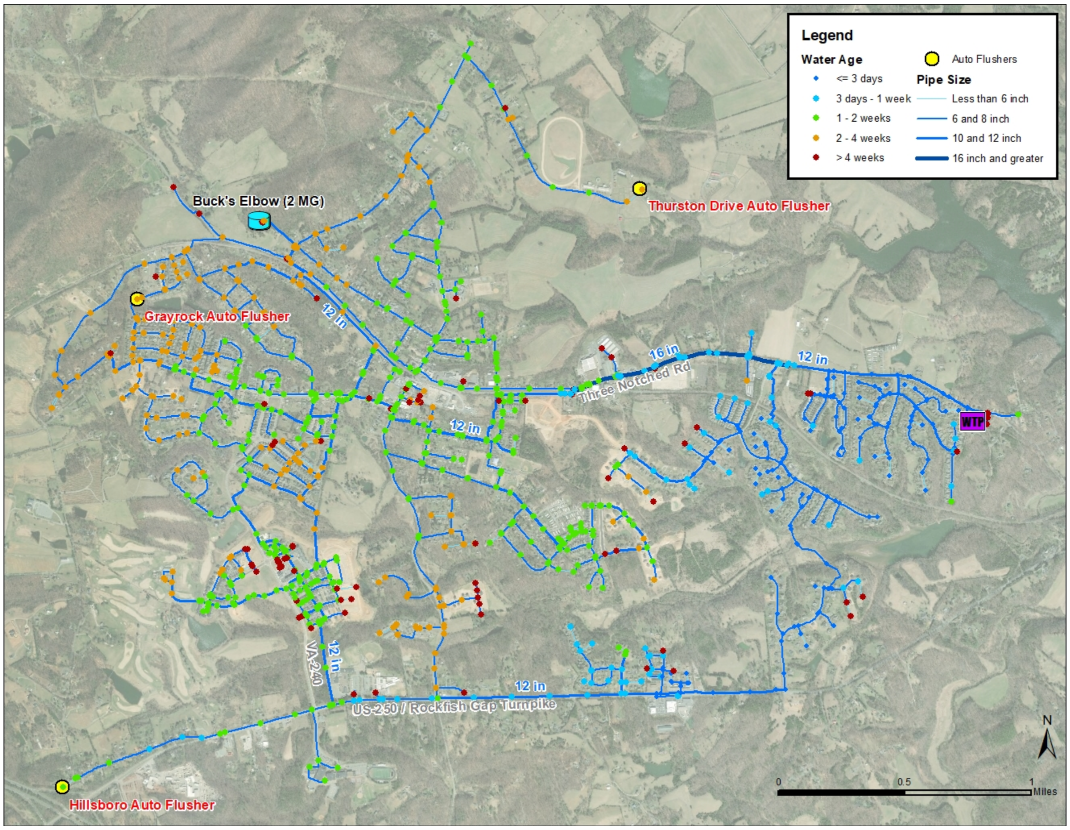
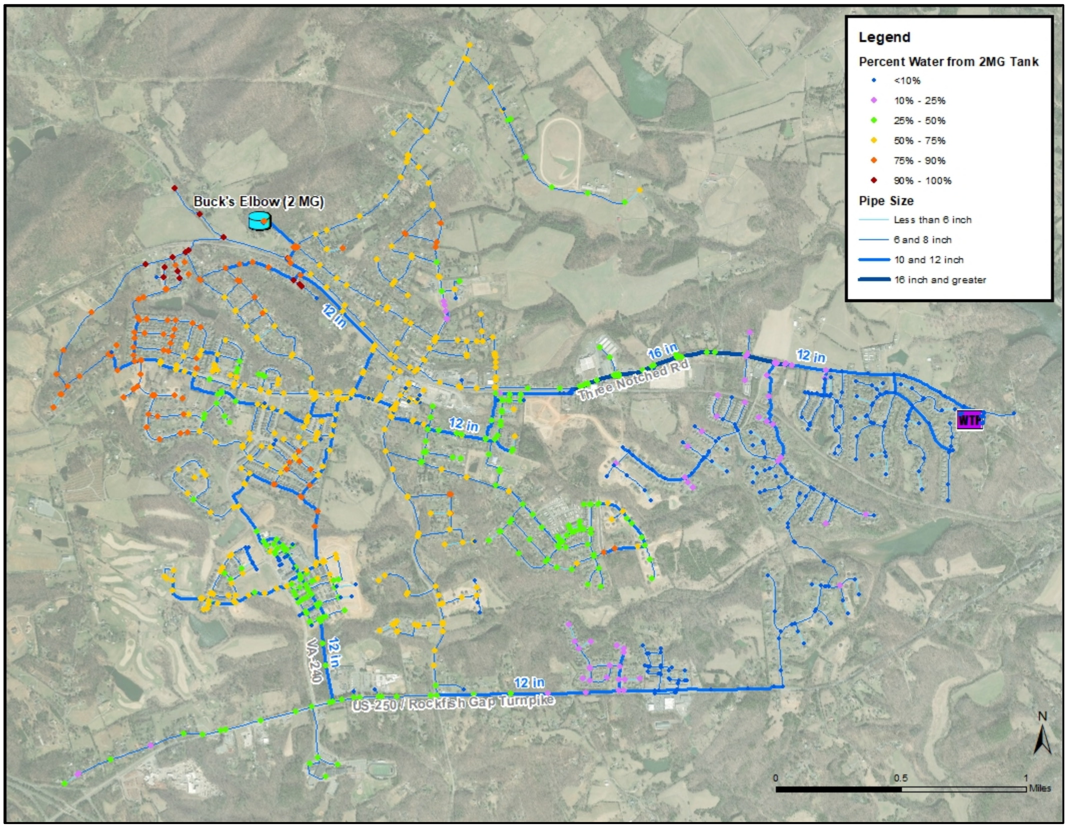


Figure 6-14: Buck's Elbow Tank Source Trace for Existing System with Modeled Operations



6.3 Capacity of Existing Infrastructure to Meet Projected Growth

The Crozet service area's buildout water requirements, as described in Section 3.2.4, are average day demand (ADD) of 1.5 mgd and maximum day demand (MDD) of 2.5 mgd.

6.3.1 Water Storage Assessment

The volume of storage that a distribution system needs has three components: equalization storage, fire storage, and emergency storage. Equalization storage optimally allows water to be pumped into the system at a constant rate equal to the 24-hour average demand rate. This approach minimizes the required capacity of transmission mains and finished water pumps. The amount of storage required depends on the amount of variation in hourly demand. The diurnal demand pattern (Figure 6-7) was used to calculate the storage requirement, which was found to be 16% of the total demand.

For buildout in 2075, 16% of 2.5 mgd maximum day demand is 0.4 million gallons for equalizing storage.

Fire storage requirements were calculated by multiplying the maximum fire flow by the required duration. The maximum flow rate is 3,500 gpm, which requires a duration of 3 hours according to AWWA guidelines. Therefore, the recommended fire storage volume is 0.63 MG.

Emergency storage capacity supplies the system in the event the normal supply is out of service during an unforeseen event (equipment failures, power outages, etc.). The VDH requirement for emergency storage is half an average day's demand. For the 2075 demand projections, the emergency storage requirement is 0.75 MG. The volume of the ground storage tank at the WTP (0.5 MG) can be included for emergency storage, which means 0.25 MG of additional storage is required in 2075.

For the Crozet distribution system, equalizing plus fire storage volumes are larger than emergency storage requirements, so those volumes were used to assess storage capacity.

Table 6-4 summarizes the existing system and the 2075 storage requirements. The distribution system currently has 1.24 MG of excess capacity and will still have a surplus of 0.97 MG in 2075.

Table 6-4: Distribution System Storage Capacity Evaluation

	Existing	2075
Average Day Demand (mgd)	0.52	1.5
Maximum Day Demand (mgd)	0.83	2.5
Equalizing Storage Needed (MG)	0.13	0.4
Fire Storage Needed (MG)	0.63	0.63
Total Storage Needed (MG)	0.76	1.03
Total Existing Storage (MG)	2.00	2.00
Storage Surplus (MG)	**1.24**	**0.97**
% Surplus Storage	**62%**	**49%**

In addition to volume, the height of the water in the tank is important to maintain adequate pressures in the system.

Minimum pressures when demands are highest and storage is at the lower limit of the normal operating range should be no less than the design criterion of 30 psi. The lowest operating level to maintain 30 psi in the distribution system corresponds to approximately 23 feet of water in Buck's Elbow Tank. The effective storage volume above that level is 0.76 MG. Therefore, the Buck's Elbow Tank has sufficient volume at a sufficient height to meet equalizing requirements through 2075.

Water quality is also an important consideration for storage. Excess storage increases water age, which may result in water quality problems in and around tanks.

6.3.2 2075 Demand Evaluation

Pressure, available fire flow, and water age were evaluated for buildout demand as shown in Figures 6-15, 6-16, and 6-17, respectively. Pressures and available fire flows decreased only marginally with future demands. Water age improves with the increased demand, so the automatic flushers will no longer be needed.

Discharge pressures and velocities were evaluated at the Finished Water Pump Station (FWPS) for a 2,100 gpm (3 mgd) future capacity under maximum day conditions. The existing pipes result in discharge pressures of 142 psi, which exceed the design criteria provided by RWSA of 125 to 130 psi.

Figure 6-15: Peak Hour Pressure with Existing Infrastructure and 2075 Peak Hour Demand

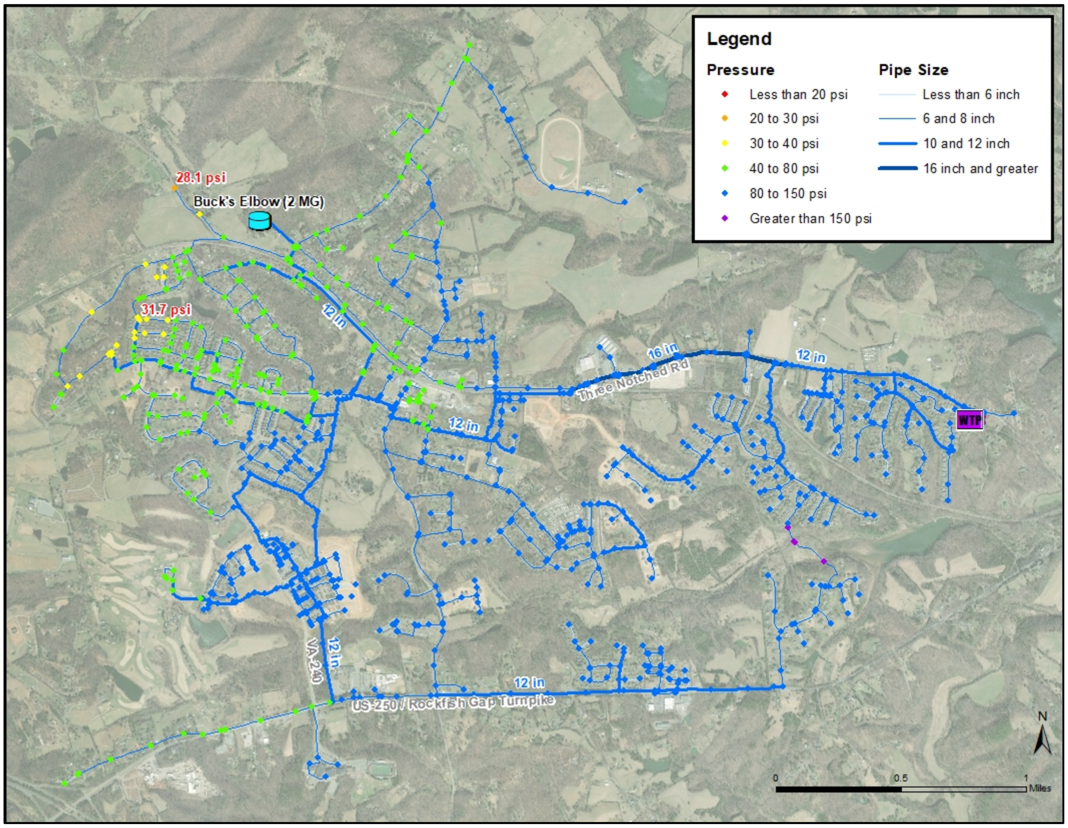


Figure 6-16: Deficient Fire Flows with Existing Infrastructure and 2075 Maximum Day Demand

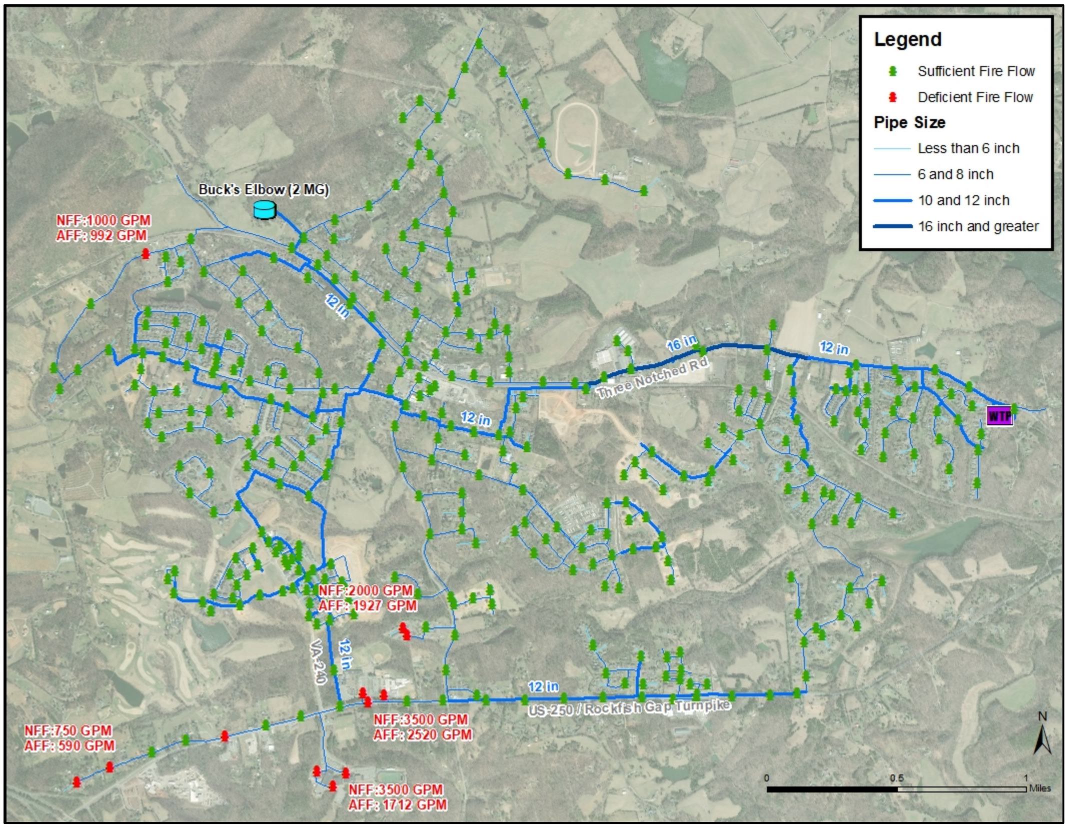
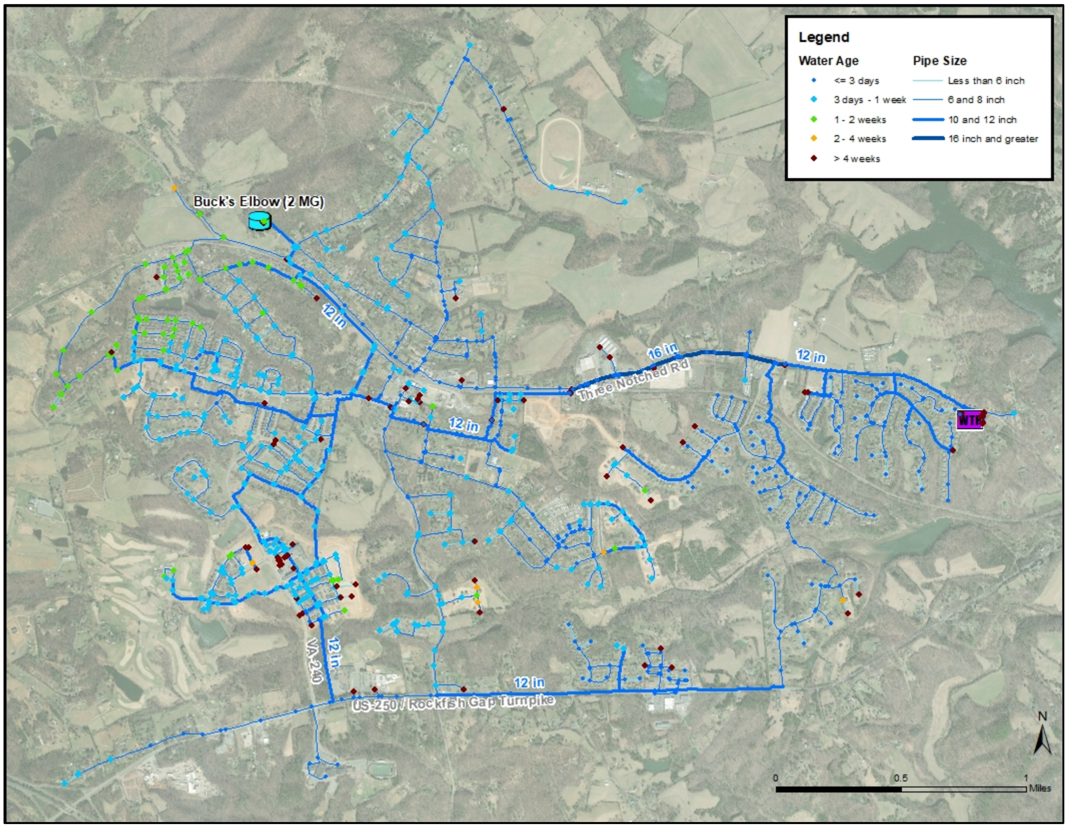


Figure 6-17: Water Age with Existing Infrastructure and 2075 Average Day Demand

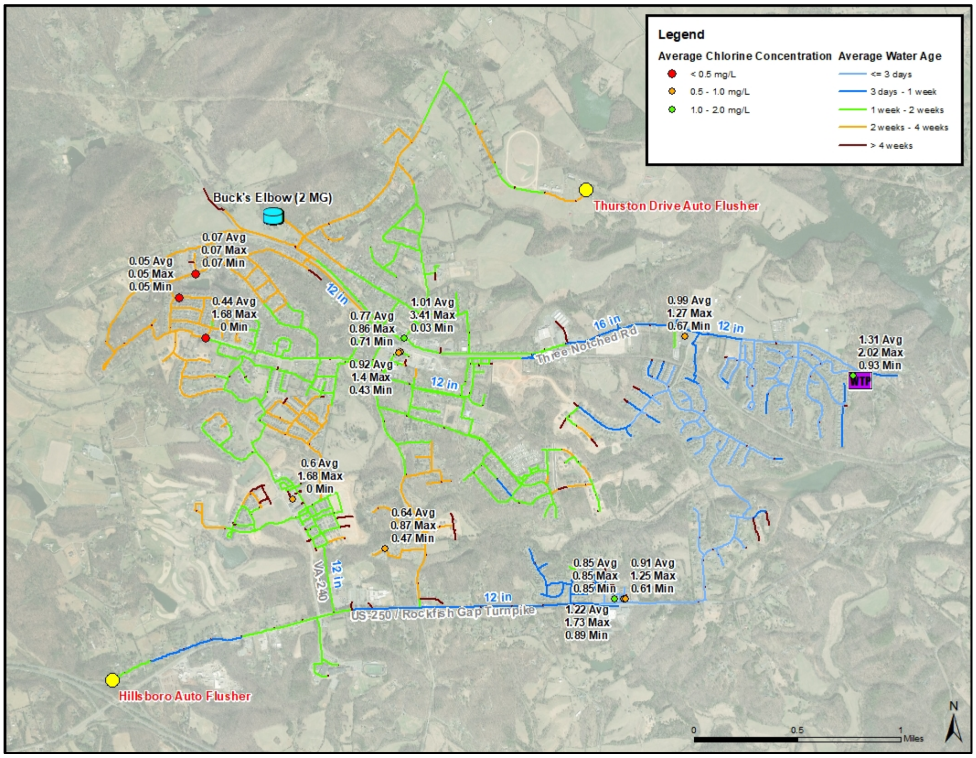


6.4 Water Quality Evaluation

The Crozet service area experiences low chlorine residuals at several locations. VDH requires a "detectable" level of chlorine at 95% of the Total Coliform Rule monitoring site samples. Figure 6-18 shows the minimum, average, and maximum chlorine concentrations in Crozet between January 2016 and May 2018. Modeled water age based on EPS calibration with SCADA is shown for comparison.

Sites on the western side of the system experience lower chlorine residuals. Buck's Elbow Tank influences water age and is the likely cause of the low disinfectant residuals. Despite the high water quality leaving the water treatment plant, free chlorine decays in the tank.

Figure 6-18: Chlorine Residual Concentrations (Jan 2016 – May 2018) and Water Age



To investigate ways to reduce water age, initial efforts with the model simulated different operating scenarios that involved controlling the finished water pumps based on Buck's Elbow Tank levels to cause different degrees of tank drawdown/turnover. While this would theoretically improve water age, the corresponding operating requirements at the water treatment plant to cause this turnover are unrealistic. Additionally, a March 2018 memorandum provided by RWSA (prepared by SEH) indicates that the level in Bulk's Elbow Tank cannot drop below 23.5 ft, which would limit the effectiveness of the tank cycling strategies considered.

Since the time of the initial modeling analysis, and with the addition of the new finished water pumps at the Crozet WTP, RWSA has made operational improvements that have been reported to successfully resolve questions surrounding water age and low chlorine residuals. These operational improvements include:

- Filling the Buck's Elbow Tank during periods of low system demand, such that the tank can be turned over during periods of higher demand.
- Changing the timing of the autoflusher operation from periods of low demand to periods of high demand. In this way, the autoflusher operation can work synergistically with the high system demands to pull the older water out of the Buck's Elbow Tank, and then turn off when the newer water is pumped into the tank from the water treatment plant.
- Providing the ability to monitor chlorine residuals into and out of the Buck's Elbow Tank using an on-line CL17 analyzer and to add chlorine as needed to boost the residual if the analyzer shows a decrease in residual. The chlorine boost is currently a manual operation requiring an

operator to visit the tank; plans are in place for a future addition of a permanent rechlorination station and associated mixer to automate the chlorine boost operation.

Accordingly, modifications to Buck's Elbow Tank operations and or finished water pumping strategies are not necessary at this time. Potential system changes to address future water quality/water age situations, were they to arise, are discussed in the next section.

In addition to the above, initial water quality modeling also predicted a problem area in the Grayrock subdivision (near 738 Russett Road) where average water age under modeled operating scenarios exceeded 18 days without flushing. An autoflusher in this area set for an average flushing rate of 50 gpm was shown by the model to reduce water age in the area to 12 days. Simultaneous with the modeling effort, ACSA installed an autoflusher in the Grayrock neighborhood, on an 8-inch distribution line approximately between 400 and 412 Grayrock Drive along the path to Lanetown Way. Correspondence with ACSA indicated that the autoflusher was in operation between July 3 and November 6 of 2018, and provided satisfactory water quality benefits while in operation. It is recommended that the autoflusher be made a permanent installation to continue to provide future water age reduction benefits.

6.5 Recommendations

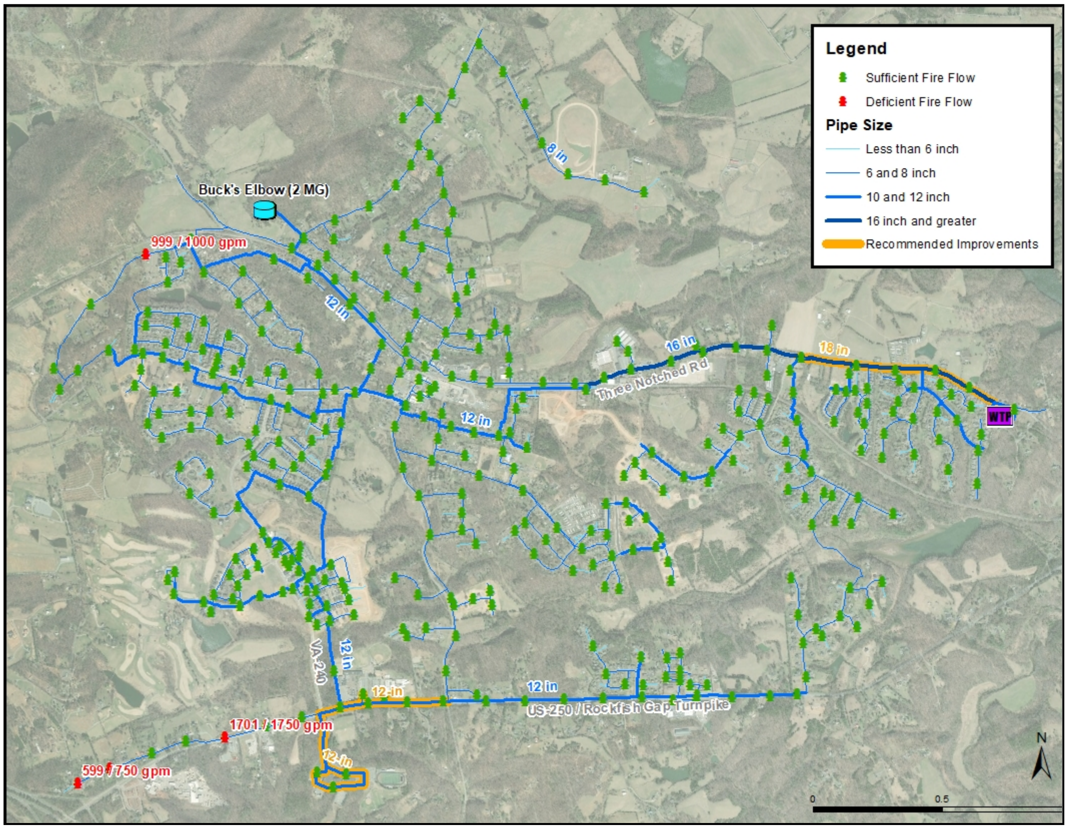
Peak hour pressures are marginal along the west side of the system, but the cost and operational complexity of capital improvements are high. Therefore, infrastructure improvements to increase peak hour pressures are not recommended at this time. Temporary pressure monitoring should be performed for the limited number of customers where daily pressures fall near the 30 psi criterion.

Future pumping requirements at the Crozet WTP FWPS necessitate improvements to reduce discharge pressures. Based on the October 2016 memorandum from SEH and our modeling results, we recommend installing approximately 4,200 ft of 18-inch pipe on Three Notched Road from the FWPS discharge to Park Ridge Drive. This improvement will reduce the maximum day discharge pressure from 142 psi to 128 psi to meet the desired discharge pressure range of 125 to 130 psi.

The existing distribution system has fire flow deficiencies that become worse as demand increases. Improvements to increase fire flows were tested with future demands for the year 2075 to ensure any improvements installed near-term will not be obsolete in the future. Fire flow improvement recommendations include 3,030 ft of 12-inch pipe on US-250 and 3,750 ft of 12-inch pipe for Western Albemarle High School. These improvements largely address the fire flow limitations noted (shown previously in Figures 6-10 and 6-16). While there remain a few hydrants that provide less than the target fire flow, they are marginally below the target value and additional capital expenditures to make up this difference do not appear to be warranted. The fire flow improvements for WAHS may not be necessary depending on additional fire flow analysis, building investigations and further requirements review by ACSA, Albemarle County Schools, and ACFR. The project is being retained in the DWIP for planning purposes until those future projects are performed and updated decisions made.

Recommended pipe improvements are shown with available fire flows assuming 2075 maximum day demand in Figure 6-19. Cost estimates for these piping improvements are described in Section 8.

Figure 6-19: Available Fire Flows with Recommended Distribution System Improvements and 2075 Demand



7. Permitting Analysis and Environmental Review

7.1 Water Supply

As previously noted, under the existing VDH permit, production of finished drinking water for the Crozet Service Area is limited to a maximum of 1 million gallons per day (mgd). Currently the Beaver Creek Reservoir has excluded status in the VWP permit system managed by the VADEQ based on the fact that it was in operation prior to the implementation of the VWP permit system program. However, a VWP permit is required to withdraw more than 1 mgd from the reservoir.

The VWP application is made through the JPA process for concurrent federal and state project review. The JPA is submitted to the VMRC, but the application is administered by the VADEQ Office of Water Supply (OWS). RWSA held a pre-application meeting with VADEQ OWS in November 2017 and participated in a Pre-Application Panel Meeting organized by VADEQ and attended by other stakeholder regulatory agencies in March 2018. Formal comments from the participating agencies have not been received but are expected to be forwarded by VADEQ to RWSA following a 60-day agency comment period. The following is a preliminary summary of comments received during the pre-application meeting:

1. VADEQ anticipates that the entire JPA process is expected to take on the order of 6 months to a year to complete, though this comment may have been in respect to the VWP portion of the JPA. It is uncertain if RWSA's need to bundle the VWP permit with the dam safety improvements will result in a lengthier review.
2. VADEQ has indicated the primary concerns for stakeholder agencies with respect to the VWP permit application will be modifications to the hydrology below the dam and any construction related impacts to wetlands and protected species habitat.
3. A mussel survey in the lower Beaver Creek for the James River Spinymussel will be required prior to commencing construction activities that could impair water quality below the dam and any significant population may need to be relocated.
4. Permits for construction on land may come with time-of-year limitations for clearing trees depending on evidence of the presence of the Northern Long-Eared Bat or breeding grounds for birds of conservation concern.

7.2 Water Treatment

The Virginia Department of Health (VDH) Plant Operating Permit allows for treatment capacity at the Crozet WTP up to 1.0 MGD. A VDH Waterworks Construction Permit was issued on 8/9/18 for the construction of the plant expansion, and an updated operating permit will be required prior to the completion of construction. Similar permits will be required for the future expansion to 2.75 mgd.

7.3 Water Distribution

Given the limited water distribution projects recommended in this DWIP, significant permitting hurdles are not anticipated. The three pipeline projects follow existing roads, and coordination/permitting with the Virginia Department of Transportation (VDOT) will be necessary.

8. Program Costs

8.1 Capital Costs

Preliminary planning level cost estimates were prepared for the water supply, treatment and distribution system improvements described previously in Sections 4, 5 and 6, respectively. These estimates were developed considering equipment prices received from manufacturers, the labor costs for the current construction market, construction information from recent projects, and recent experience with drinking water infrastructure costs. The cost estimates (in 2018 dollars) include markups for contractor overhead and profit of 20 percent, a 5 percent markup for "general conditions", a markup for contingency of 30 percent (to reflect the very preliminary nature of the concepts), and an 18 percent markup for engineering fees, program management fees, construction inspection and miscellaneous legal costs. The "general conditions" include costs to the contractor for items such as mobilization/demobilization, bonding, onsite office expenses, expenses incurred due to construction sequencing constraints, etc. The contingency is intended to cover the unknowns associated with geotechnical conditions, small piping and miscellaneous equipment, future materials prices and market conditions (hot or cold contractor market), and other unknowns.

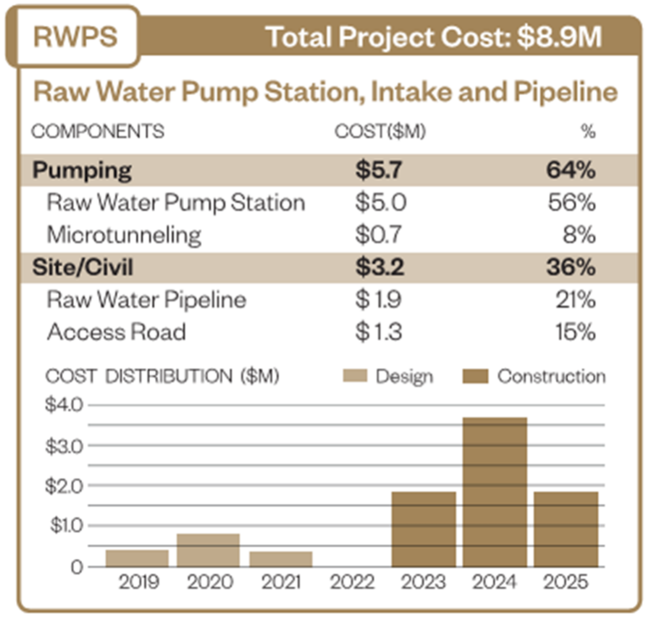
8.2 Project Cards

Each of the discrete projects proposed for implementation as part of the Crozet Service Area DWIP is associated with one of the three areas of water infrastructure through color coding and, for convenient reference, each project has a summary table, which is referred to as a project card. The DWIP for the Crozet Service Area is expressed via 6 project cards. The cost graph on the project cards shows the expected expenditures for construction and engineering costs over the life of the project. An S-curve type distribution is used to show the initial smaller expenditures on planning and design, followed by the larger construction expenditures and the trailing construction completion and commissioning expenditures.

Each project card provides:

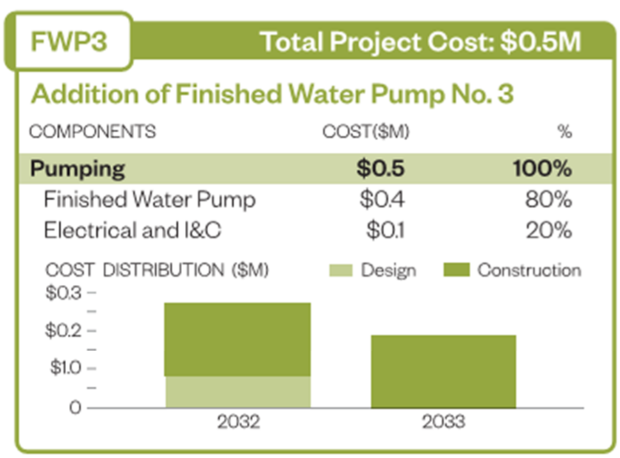
- The overall project cost, including capital costs as well as engineering costs associated with planning, design and construction, in 2018 dollars.
- Project components and sub components, with cost and percentage of total cost provided for each component and sub-component.
- Projected distribution of costs, on an annual basis, over the life cycle of the project.

Figures 8-1 to 8-3 include all of the project cards, grouped according to the water infrastructure component. It should be noted that the pipeline projects would be designed and constructed by ACSA rather than RWSA; however, they are included here to provide the full picture of infrastructure needs for the Crozet Area over the planning horizon.

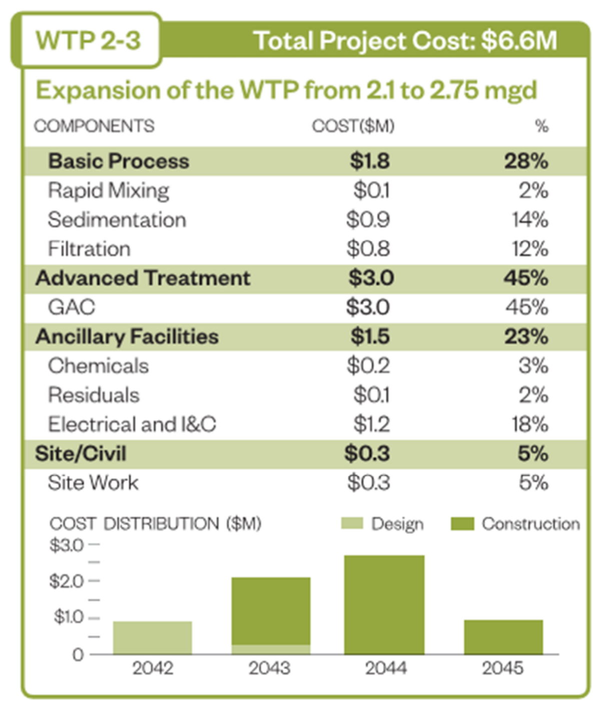


COMPONENTS	COST($M)	%
Pumping	**$5.7**	**64%**
Raw Water Pump Station	$5.0	56%
Microtunneling	$0.7	8%
Site/Civil	**$3.2**	**36%**
Raw Water Pipeline	$1.9	21%
Access Road	$1.3	15%

Figure 8-1: Water Supply Infrastructure Project Card



COMPONENTS	COST($M)	%
Pumping	**$0.5**	**100%**
Finished Water Pump	$0.4	80%
Electrical and I&C	$0.1	20%



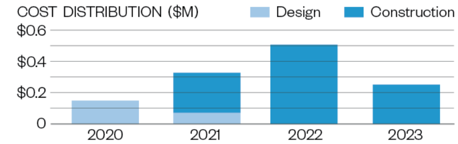
COMPONENTS	COST($M)	%
Basic Process	**$1.8**	**28%**
Rapid Mixing	$0.1	2%
Sedimentation	$0.9	14%
Filtration	$0.8	12%
Advanced Treatment	**$3.0**	**45%**
GAC	$3.0	45%
Ancillary Facilities	**$1.5**	**23%**
Chemicals	$0.2	3%
Residuals	$0.1	2%
Electrical and I&C	$1.2	18%
Site/Civil	**$0.3**	**5%**
Site Work	$0.3	5%

Figure 8-2: Water Treatment Infrastructure Project Cards

ACSA-FF US250* **Total Project Cost: $1.2M**

Piping Replacement to Improve Fire Flow Along Route 250

COMPONENTS	COST($M)	%
Pipeline	**$1.2**	**100%**
Distribution Piping	$0.8	67%
Site/Civil	$0.4	33%

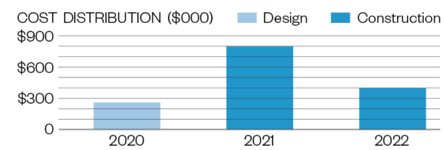


* The fire flow improvements for Route 250 may not be necessary depending on additional fire flow analysis, building investigations and further requirements review by ACSA, Albemarle County Schools, and ACFR.

ACSA-FF WAHS* **Total Project Cost: $1.4M**

Piping Replacement to Improve Fire Flow Near West Albemarle High School

COMPONENTS	COST($M)	%
Pipeline	**$1.4**	**100%**
Distribution Piping	$0.9	64%
Site/Civil	$0.5	36%



* The fire flow improvements for WAHS may not be necessary depending on additional fire flow analysis, building investigations and further requirements review by ACSA, Albemarle County Schools, and ACFR.

FWT **Total Project Cost: $1.9M**

Piping Replacement to Improve Operating Pressures With Expanded Crozet FWPS

COMPONENTS	COST($M)	%
Pipeline	**$1.9**	**100%**
Distribution Piping	$1.3	68%
Site/Civil	$0.6	32%

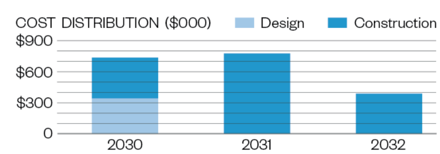


Figure 8-3: Water Distribution Infrastructure Project Cards

9. Recommended Crozet Service Area Program

The Drinking Water Infrastructure Plan (DWIP), Figure 9-1, provides the answer to "when" the discrete projects identified in Section 8 should be implemented to meet the demands for finished drinking water in the Crozet Service Area. The project timing is tied to the 5-year incremental demand increases presented in Section 3 for the "Growth Outside DA MDD" projection curve. Essentially, each specific project is programmed to start in the DWIP such that construction is complete just before the projected maximum day demand would exceed the available capacity in the particular component. For example, the current firm capacity of the Crozet WTP finished water pump station is 1.5 mgd, and the projected maximum day demand will exceed that capacity in the year 2033. The DWIP therefore assumes that the future third finished water pump would be installed and on-line in 2033 such that it is available to supply the higher demand that will exist after that time.

The DWIP includes the color-coded project cards presented in Section 8 to allow the timing for implementation of water supply, water treatment and water distribution projects to be more easily differentiated. The proposed project timing included in the DWIP should be considered flexible and can be adjusted to respond to changes in the projected demand patterns. As the proposed project timing is directly related to incremental increases in MDD, if the rate of MDD growth is slower than anticipated, the implementation of the projects can be deferred until a particular capacity threshold is reached.

In addition to the project implementation timeline, the DWIP also provides the overall program costs for the Crozet Service Area, by year, and on a cumulative basis. The overall cost for the recommended Crozet Service Area DWIP is $20.6 million (in 2018 dollars).

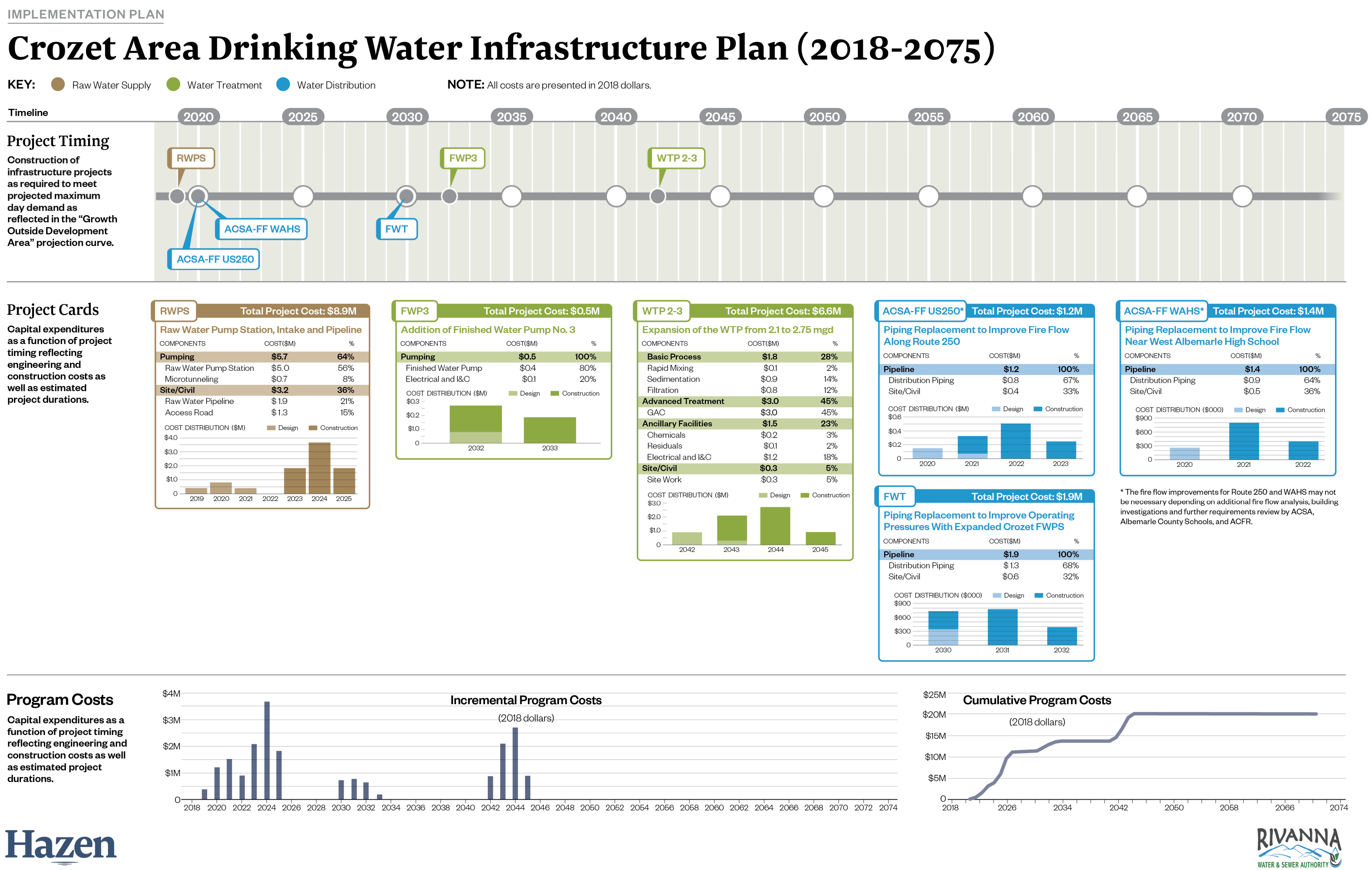


Figure 9-1: Drinking Water Infrastructure Plan - Crozet Area

Appendix A: Modeling the Crozet Water Supply System with OASIS

Modeling the Crozet Water Supply System with OASIS

April 2018

Prepared by

HYDROLOGICS
Advancing the management of water resources

and

Hazen

for

RIVANNA
WATER & SEWER AUTHORITY

Contents

Section 1. Introduction

This report describes the development, calibration and verification of the OASIS hydrologic model of the Crozet water supply system. It also describes a series of safe yield (SY) and alternative minimum instream flow scenarios that were modeled for the Crozet system, which is operated by the Rivanna Water and Sewer Authority (RWSA).

HydroLogics augmented its existing OASIS hydrologic model of the RWSA system to model:

1. The existing Crozet system, which consists of Beaver Creek Reservoir, its multi-purpose spillway and intake structure, and the Crozet water treatment plant (WTP), and

2. The proposed Crozet configuration, in which the Beaver Creek Dam will be upgraded and a dedicated water supply intake constructed upstream of the dam to serve the expanded WTP.

The model adds flows downstream of the dam and to the Mechums USGS gage and uses a new inflow data set that was developed for this project (see Section 2.2), which extends from 1925 to 2017.

This report gives an overview of the model inputs and operations (Section 2), and the results of the various scenarios modeled (Section 3).

Section 2. Model Components

2.1 Schematic

The model uses a map-based schematic that includes nodes for reservoirs, water withdrawals (water treatment facilities), and arcs that convey water between nodes. The model schematic is shown on the following page and is sized to show the full RWSA system.

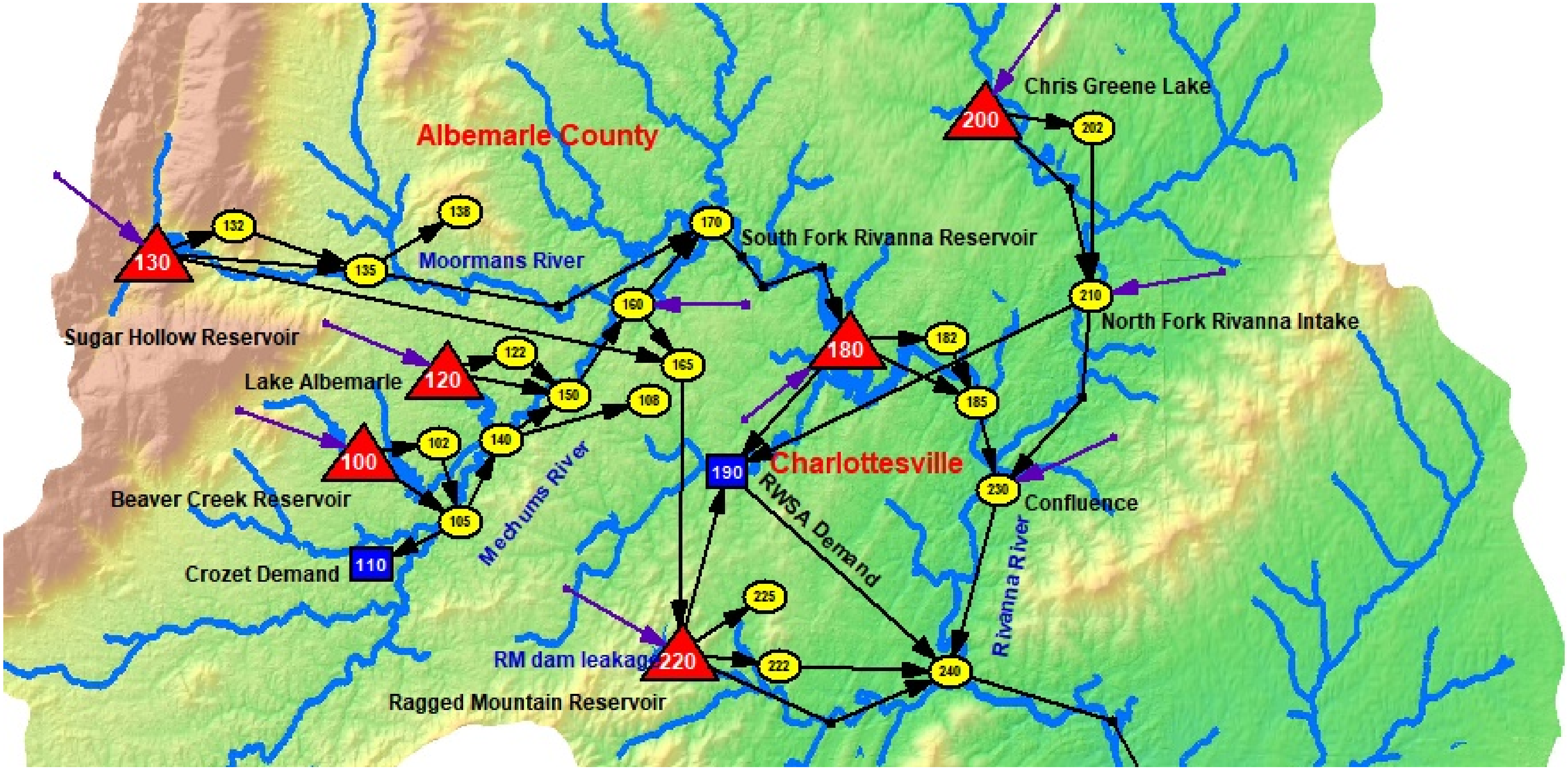


Figure 1. RWSA OASIS Model Schematic

2.2 Model Input

Model input consists of "static" data, which represent the physical characteristics of the system, and daily timeseries data of the natural inflows to and precipitation/evaporation on the reservoir.

Static Data

The model uses a storage-area-elevation (SAE) curve to convert storage volumes to surface areas and elevations that are used for computing net evaporation and elevation-based operating rules. The SAE curve was updated for this project using recent bathymetry data provided by RWSA. Figure 2 shows the relationship between storage and elevation and includes the elevations of the intake gates.

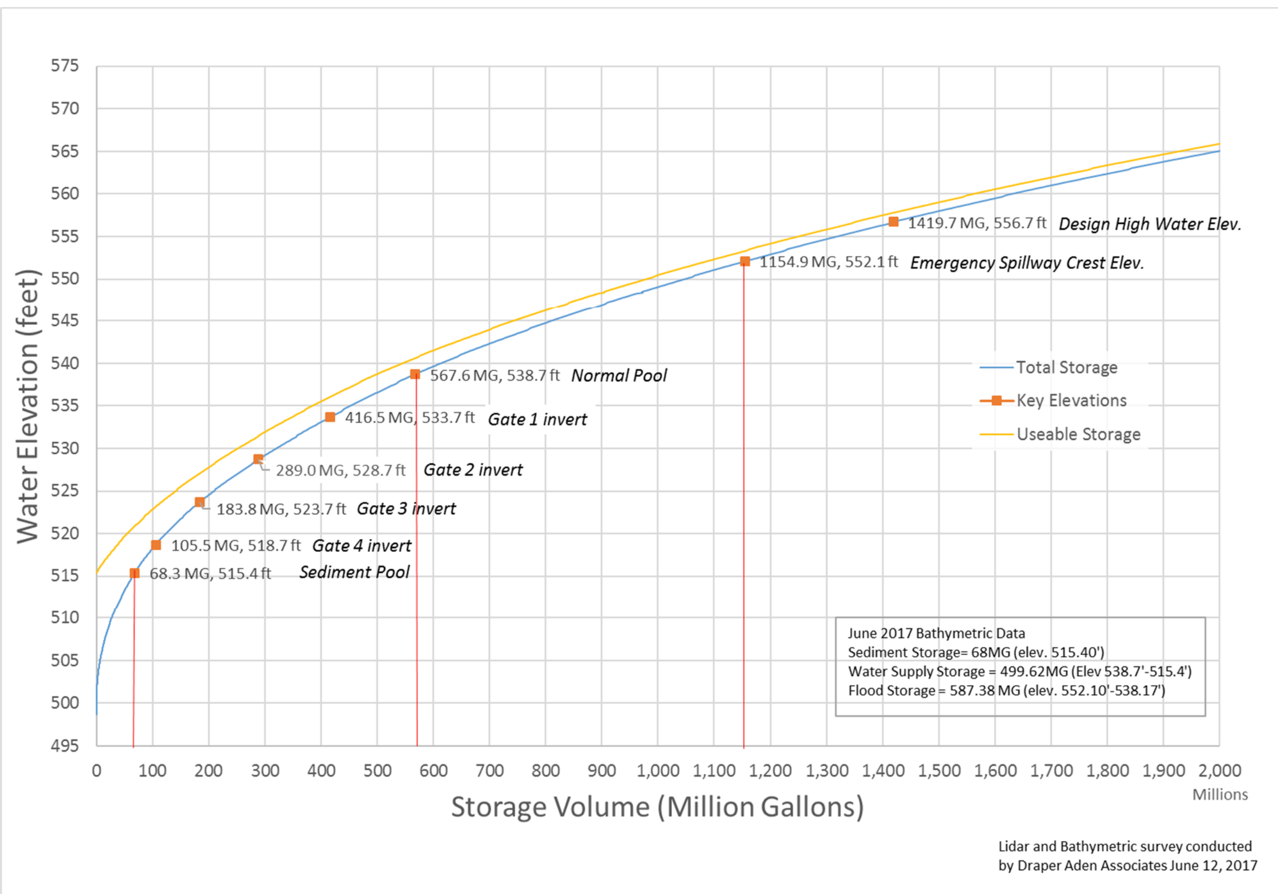


Figure 2. Beaver Creek Reservoir Storage-Elevation Curve

The model uses an annual average daily demand with a monthly multiplier to represent the Crozet water treatment plant withdrawals. The annual average demand for the basecase scenario used is 0.5 MGD. That demand is varied when computing SY and doing sensitivity analysis on future demands. The monthly peaking factors shown in Table 1 were computed using recent years of data (2015 – 2017) and remains the same for all scenarios.

Table 1. Monthly Peaking Factor for Crozet WTP Demand

January	0.93
February	0.95
March	0.95
April	0.97
May	1.04
June	1.04
July	1.06
August	1.07
September	1.10
October	1.01
November	0.97
December	0.91

Other static input data relevant to the Crozet portion of the RWSA model includes lookup tables for the dam spillway and gate outlet structures, which are shown below.

Table 2. Beaver Creek Spillway Discharge Curve

Reservoir Elevation	Discharge (cfs)
538.7	0.0
538.8	1.9
538.9	5.5
539.0	10.1
539.1	15.5
539.2	21.6
539.4	35.8
539.6	52.3
539.8	70.6
540.0	90.7
540.3	118.1
540.5	147.8
540.8	179.6
541.0	213.5
541.3	240.3
541.5	241.0
541.8	241.7
542.0	242.4
543.0	245.2
550.0	263.7

Table 3. Beaver Creek Gate Outlet-Discharge Curves

Reservoir Elevation	Gate 1 (2.6" open) Discharge (cfs)	Gate 2 (2.6" open) Discharge (cfs)	Gate 3 (2.6" open) Discharge (cfs)	Gate 4 (2.6" open) Discharge (cfs)	Gate 5 (1" open) Discharge (cfs)
500	0.0	0.0	0.0	0.0	1.2
501	0.0	0.0	0.0	0.0	1.8
502	0.0	0.0	0.0	0.0	2.3
503	0.0	0.0	0.0	0.0	2.6
504	0.0	0.0	0.0	0.0	3.0
505	0.0	0.0	0.0	0.0	3.3
506	0.0	0.0	0.0	0.0	3.6
507	0.0	0.0	0.0	0.0	3.8
508	0.0	0.0	0.0	0.0	4.0
509	0.0	0.0	0.0	0.0	4.3
510	0.0	0.0	0.0	0.0	4.5
511	0.0	0.0	0.0	0.0	4.7
512	0.0	0.0	0.0	0.0	4.9
513	0.0	0.0	0.0	0.0	5.1
514	0.0	0.0	0.0	0.0	5.2
515	0.0	0.0	0.0	0.0	5.4
516	0.0	0.0	0.0	0.0	5.6
517	0.0	0.0	0.0	0.0	5.8
518	0.0	0.0	0.0	0.0	5.9
519	0.0	0.0	0.0	0.4	6.1
520	0.0	0.0	0.0	1.1	6.2
521	0.0	0.0	0.0	1.5	6.4
522	0.0	0.0	0.0	1.8	6.5
523	0.0	0.0	0.0	2.0	6.7
524	0.0	0.0	0.4	2.2	6.8
525	0.0	0.0	1.1	2.5	6.9
526	0.0	0.0	1.5	2.6	7.1
527	0.0	0.0	1.8	2.8	7.2
528	0.0	0.0	2.0	3.0	7.3
529	0.0	0.4	2.2	3.2	7.4
530	0.0	1.1	2.5	3.3	7.6
531	0.0	1.5	2.6	3.4	7.7
532	0.0	1.8	2.8	3.6	7.8
533	0.0	2.0	3.0	3.7	7.9
534	0.4	2.2	3.2	3.8	8.0
535	1.1	2.5	3.3	4.0	8.2
536	1.5	2.6	3.4	4.1	8.3
537	1.8	2.8	3.6	4.2	8.4
538	2.0	3.0	3.7	4.3	8.5
539	2.2	3.2	3.8	4.4	8.6
540	2.5	3.3	4.0	4.5	8.7

Timeseries Data

Historic Data

To verify the accuracy of model inflows and operations, historical elevation, withdrawal, and outflow data were used as inputs to the model for verification runs. In these runs, the model is forced to match known inflows/outflows and then the resulting elevation/storage is compared to historic values. Historic elevation and withdrawal data was available going back to 2001. During periods of high reservoir outflows, historic outflow data was not reliable due to issues with the spillway conduit flow meter. Therefore, the historic high outflows had to be estimated based on the known elevations, using a methodology developed by Hazen.

Data on the historical outflow from the reservoir including both flow over the spillway and withdrawals for water supply were essential for verifying the OASIS model of the Beaver Creek Reservoir and making calibration adjustments when the model verification based on the standard area-adjusted gage inflow indicated improvements were needed. Based on records maintained by RWSA, two potential methods were evaluated for developing an outflow estimate from the Beaver Creek Reservoir. The first method used data directly from RWSA's area-velocity ultrasonic flow meter placed in the 42" discharge line from the dam. However, this equipment has only been in service since 2013, providing a rather short period to calibrate the model, and staff indicated the data were not accurate during high flows. These shortcomings led to the second method, which involved the development of a hydraulic model of the spillway and sluice gates as operated using historical reservoir elevation data. The reservoir elevation data has been recorded hourly in the SCADA system since August 2008. This frequency of data logging allowed for sufficient temporal disaggregation to estimate outflow using steady-state assumptions to model a dynamic system. As shown in Figures 3A and 3B, the resulting hydraulic model of the spillway and sluice gates driven with the reservoir elevation data proved more reliable and accurate than the data from the outflow meter in the discharge line, which were therefore not used.

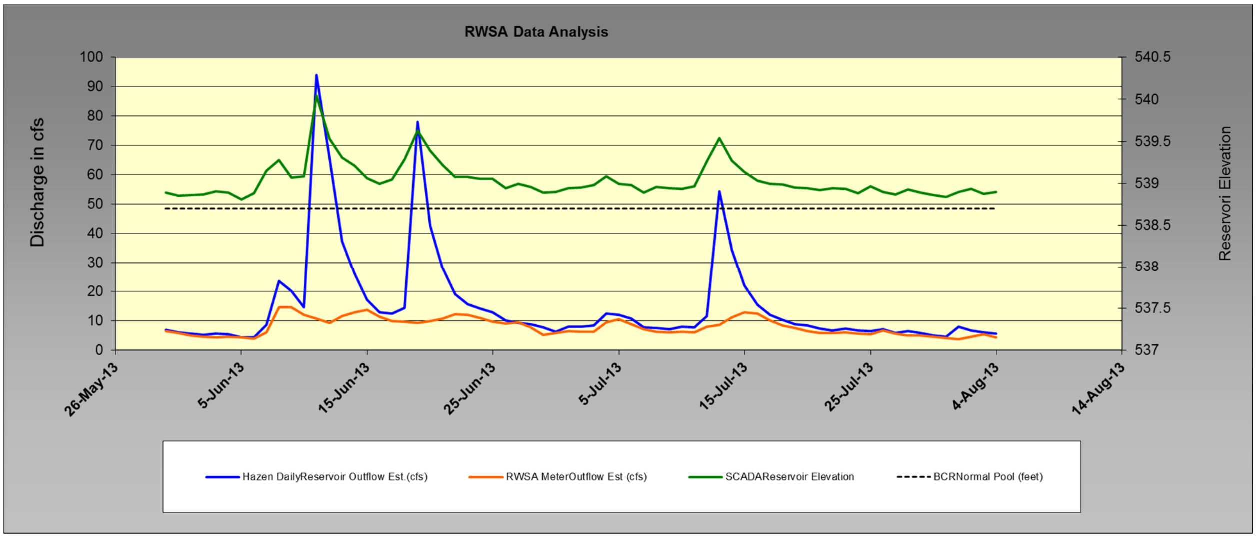


Figure 3A: Comparison of Hydraulic Model Outflow Estimate, Ultrasonic Outflow Meter Data, and Reservoir Elevation Data

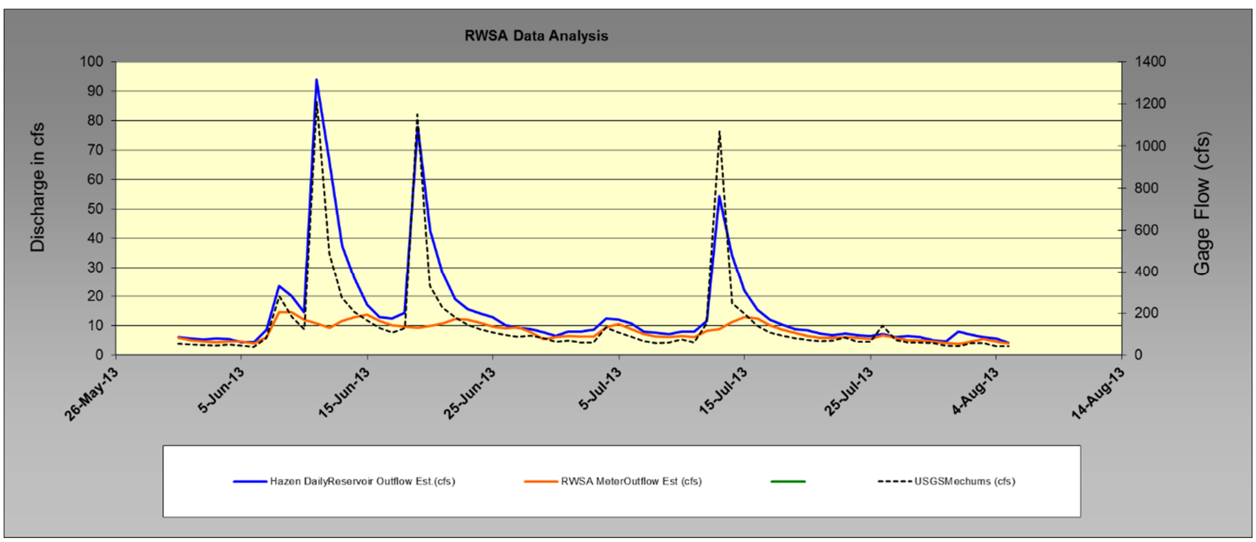


Figure 3B: Comparison of Hydraulic Model Outflow Estimate, Ultrasonic Outflow Meter Data, and Mechums River Gage Data

Discharge estimates from the reservoir when below normal pool were developed using flow equations and assumptions for the existing intake ports and standard operating protocol provided by RWSA. Staff provided a description of the typical sluice gate opening and this was further refined during a field visit to the reservoir in November 2017. The sluice gate portion of the hydraulic model assumes that staff open either gate 1 or gate 2 (depending on when during the 2008 – 2017 period as gate 1 was in disrepair until June 2015) 27 turns, producing about 5 ¾" stem rise and leaving an effective port opening of just under 3 inches after accounting for the necessary gate travel to clear the seat facings and shutter lip. This produced a reasonable match with the outflow meter, which staff considered reasonably accurate during low flow releases. Figure 4 shows an example match during a period when the reservoir was below normal pool in August and September 2015.

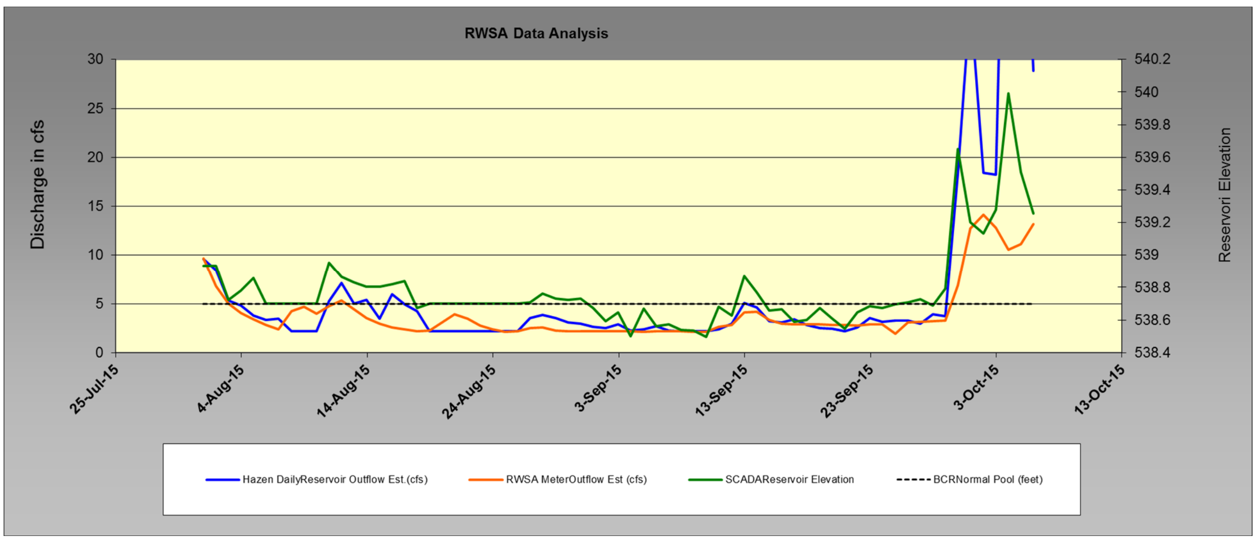


Figure 4: Comparison of Hydraulic Model Outflow Estimate, Ultrasonic Outflow Meter Data, and Reservoir Elevation Data

Inflows

The existing model used the Mechums gage with a drainage area adjustment for the inflow to Beaver Creek Reservoir. An extended inflow record back to 1925 was developed by a previous consultant using other gages to fill in the gaps in the Mechums gage record by employing statistical relationships and drainage area adjustment[1]. The other reference gages included: Rivanna R. near Charlottesville, Rivanna R. at Palmyra, S.F. Rivanna R. near Earlysville, and N.F. Rivanna R. near Proffit.

The aforementioned drainage area adjustment is often an appropriate approach for determining the runoff characteristics of a smaller drainage area contained within a larger gaged area. However, analysis indicates that this is not so in the Mechums gage drainage area and the much smaller Beaver Creek Reservoir area, where the relative small size and steeper slopes evidently result in somewhat different runoff characteristics. Specifically, rainfall will run off the steeper target watershed more quickly than for the reference gage, resulting in relative higher overall runoff volumes and corresponding lower rates of infiltration into the soil[2]. The precipitation intensity and temporal pattern over the target drainage area may also not be well represented by that over the larger gaged area. In this case, during the period from September 2008 through August 2017, the target watershed exhibited an estimated average runoff rate of 1.16 cfs/mi^2 compared with a rate of 1.02 cfs/mi^2 at the reference gage. However, it was found that applying a uniform adjustment factor of 1.14 (1.16 ÷ 1.02) in addition to the area adjustment factor did not improve model verification particularly during reservoir drawdown periods as compared to no adjustment at all. Similar testing of additional USGS gages from outside the basin were not successful in producing improved inflow results. See section 3.1 for a summary of verification results.

As a result of these findings, the runoff relationship between the reservoir watershed and the total watershed down to the gage was studied in further detail.

The model verification mismatch observed when a constant flow calibration factor (1.14) was applied to gage inflows was determined to be due to the temporal difference in runoff between the target and reference watershed. While the target watershed generally produces greater than proportional runoff during storm events and wet periods, the opposite is true during dry periods. A method was needed to account for this fact. It was therefore decided to adjust the reference gage (Mechums River gage) unit discharge with a factor based on the relative wetness or dryness of antecedent streamflow conditions. The back-calculated unit inflow for the 9 years with available reservoir outflow data was compared to unit flow at the gage over the same period. Antecedent conditions were determined using a running 30-day average of discharge at both locations. Results were ordered by percentile – from least discharge to greatest discharge. The results in Figure 5 show that there is a greater unit runoff in the target

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[1] "Beaver Creek Reservoir Safe Yield Study," Gannett Fleming, June 8, 2007.

[2] Langbein, W.B. and others, 1947, Topographic Characteristics of Drainage Basins: U.S. Geological Survey Water-Supply Paper 968-C, p. 125-144.

watershed under most conditions, but during the driest of conditions this characteristic was reversed. Using this ordering, the unit runoff at the reference gage was adjusted by the ratio of the unit runoff derived for the 2008-2017 period to extend the inflow record for the Beaver Creek Reservoir over the full period in which discharge data at the Mechums River gage has been recorded, allowing the OASIS model to incorporate a much larger hydrologic history than would be possible otherwise.

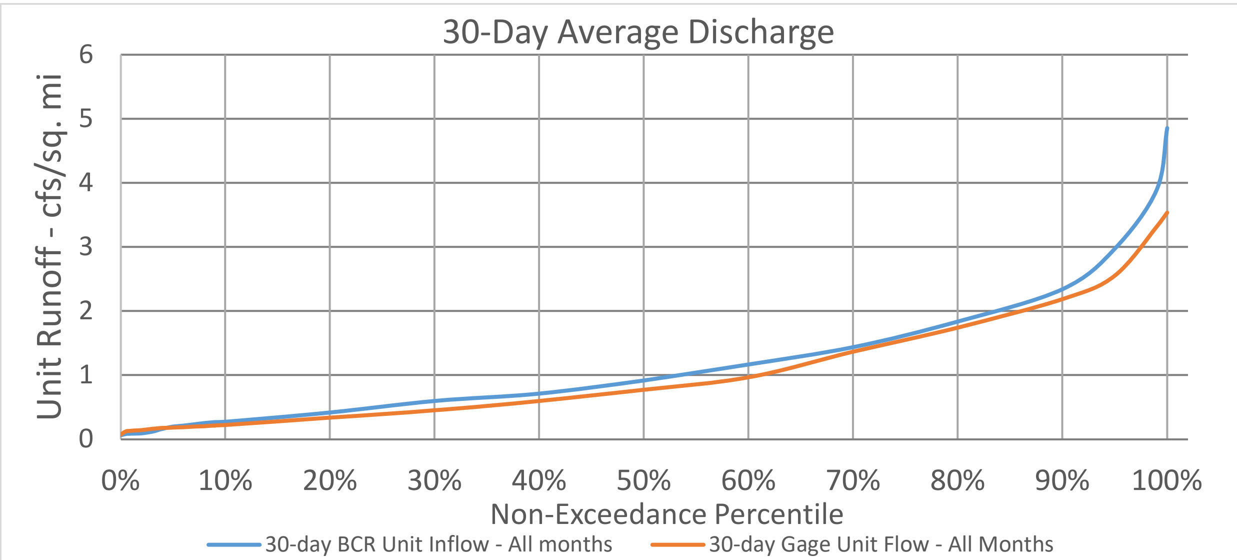


Figure 5: Unit Runoff by Percentile Rank for Beaver Creek Reservoir and the Mechums River Gage for September 2008 – August 2017

This method was applied with discharge at both locations separated by season, with the wet season running from November through May and the dry season running June through October. The application of the method does not produce a perfect validation curve, but it accounts for the additional inflow evident in the target watershed through multiple lines of analysis while retaining as good a fit as possible during reservoir drawdown periods. Figure 6 contains an example of how the Beaver Creek inflow is calculated using this method.

For the local inflow to the Mechums gage downstream, the Beaver Creek inflow as computed above is subtracted out from the gage flow.

Date	30-day average unit runoff at gage (cfs/sm)	Lookup Adjustment Factor	Gage Flow (Cfs)	Gage Unit Runoff (Cfs/sm)	Estimated BCR Unit Runoff (Cfs/sm)	Estimated BCR Inflow (CFS)
12/16/2014	0.81	1.09	59.3	0.62	0.68	6.35
12/17/2014	0.81	1.09	58.9	0.62	0.67	6.31
12/18/2014	0.81	1.09	52.8	0.55	0.60	5.66
12/19/2014	0.82	1.09	49.7	0.52	0.57	5.33
12/20/2014	0.82	1.09	48.7	0.51	0.56	5.22

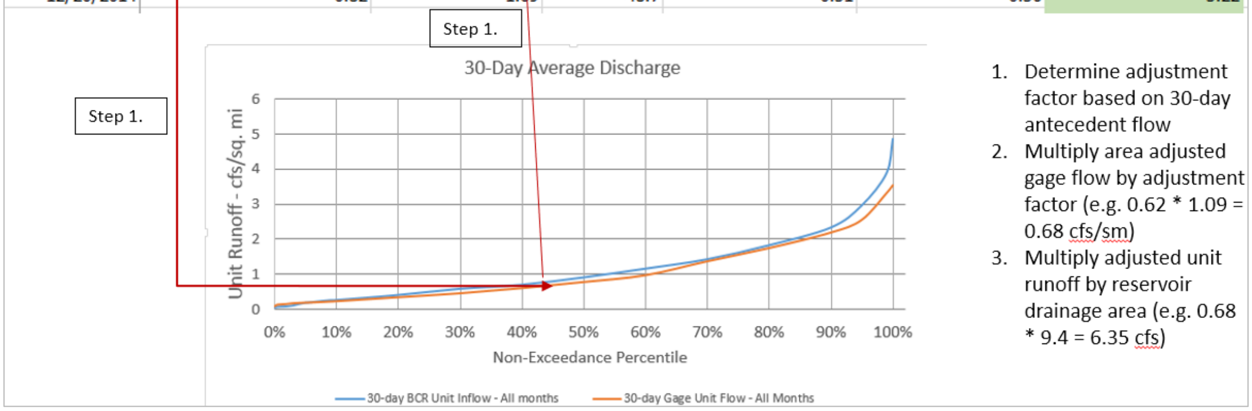


Figure 6: Sample Beaver Creek Reservoir Inflow Calculation

Net Evaporation

Net evaporation in the model is computed as evaporation minus precipitation, in inches. Evaporation was originally derived from a monthly pattern provided by RWSA, disaggregated to daily values, and later switched to use the same data set as for the rest of the system (based lake surface evaporation studies by USGS in North Carolina[3]). Daily precipitation data for the reservoirs was provided by RWSA from 2008 to 2017. Prior to 2008, National Climatic Data Center weather stations (primarily the Charlottesville 2W station) were used by a previous consultant[4] to develop the precipitation record back to 1925. In the model, net evaporation is multiplied by the surface are of the reservoirs (as computed by the elevation-storage-area tables) and subtracted from storage for each time step.

[3] Turner, J.F., "Evaporation Study in a Humid Region, Lake Michie North Carolina,", USGS Pub. No. 272-G, 1966.

[4] "Beaver Creek Reservoir Safe Yield Study," Gannett Fleming, June 8, 2007.

2.2 Operating Rules

Existing Operating Rules

For the basecase scenario, the model was set up to emulate the operation of the outlet structure gates, as specified by RWSA staff. The first gate is opened when the reservoir drops to near the spillway elevation, and subsequent gates are opened when the flow in the open gate cannot provide enough outflow. Only one gate is ever open at a time.

Gate operating protocol:

>= 538.8, all gates closed

< 538.8, open gate 1 2.6"

Gate 1 flow < 1.0 mgd, open gate 2 2.6"

Gate 2 flow < 1.0 mgd, open gate 3 2.6"

Gate 3 flow < 1.0 mgd, open gate 4 2.6"

Spillway outflows were modeled using the curve shown in Table 2, and by using the total reservoir inflow for the day to estimate the hourly change in storage and resulting hourly spillway outflow, and then summing the hourly outflows into a daily total. This method provides an accurate estimate of the daily spill since hour-to-hour changes in the reservoir elevation can produce large changes in spill, and simply using the beginning of day reservoir elevation would not provide an accurate outflow during spill events.

Proposed Minimum Instream Flow Protocol

Virginia Water Protection (VPW) regulations[5] require a permit modification for changes to permitted project or surface water withdrawal. Permit requirements include the identification of impacts to instream flow requirements; for the proposed changes to Beaver Creek reservoir, an alternative minimum instream flow (MIF) protocol was developed to satisfy VA DEQ regulations.

For the proposed MIF for Beaver Creek reservoir, the future dam and intake upgrades are assumed to be in place, so the downstream storage loss that presently occurs due to current gate operations are no longer applied in these model evaluations. Instead, the withdrawal would be made from an intake within the reservoir, and the minimum release from the dam would be based on natural inflow to the reservoir, to mimic natural variation, which is how MIF is managed at other RWSA reservoirs. The specific minimum release protocol is as follows:

- Requirement decreases as reservoir draws down:
  - Usable storage >= 90%, minimum release = 50% of inflow
  - Usable storage < 90%, minimum release = 40% of inflow
  - Usable storage < 80%, minimum release = 20% of inflow
  - Usable storage < 70%, minimum release = 10% of inflow
  - Impose 1 mgd upper limit on minimum release
- Flow over spillway at water surface elevations > 538.7 ft counts towards minimum release

Virginia DEQ guidelines require that SY be calculated with a 60-days of demand reserve on top of the lowest physical limit of accessible storage in the reservoir. In the model this 60-day reserve was calculated on a rolling-average basis, to capture the seasonality of demands.

Finally, demand reductions were considered for certain SY scenarios. The follow demand reductions, triggered by reservoir storage levels, were used when tested:

- Usable storage < 80%, 5% total reduction (Stage 3)
- Usable storage < 70%, 20% total reduction (Stage 4)

[5] 9VA25-210 – Virginia Water Protection Permit Program Regulation, https://law.lis.virginia.gov/admincode/title9/agency25/chapter210/

Section 3. Results

3.1 Model Verification

As mentioned previously, the model was configured to match historic withdrawals, and outflows were computed using the gate operations which are a close match to historic operations, and the resulting reservoir elevation was compared to historic data to determine the accuracy of inflows. Figure 7 shows the final verification result after an iterative series of inflow adjustments summarized in Section 2 were made. Note that the poor fit in 2002 is expected, because Beaver Creek operations were altered in that year[6], though RWSA staff indicate that specific details of those operations are unknown. Figure 8 shows the original model verification attempt which relied on the spillway conduit flow meter data as the source of both demand and outflow from the reservoir which came prior to the development of the hydraulic model of the spillway described in Section 2.2. Figure 9 shows a subsequent verification run using a single inflow adjustment factor tested after the development of the hydraulic model of the spillway and sluice gates. Figure 10 shows results where various gages from outside the basin were tested as reference gages for the inflow.

[6] Changes to operations are alluded to in an email from Gene Potter to Bill Brent on January 31, 2002 as RWSA was managing the worst drought in the modern record, but details regarding what these operations were and when they began and ended are not available.

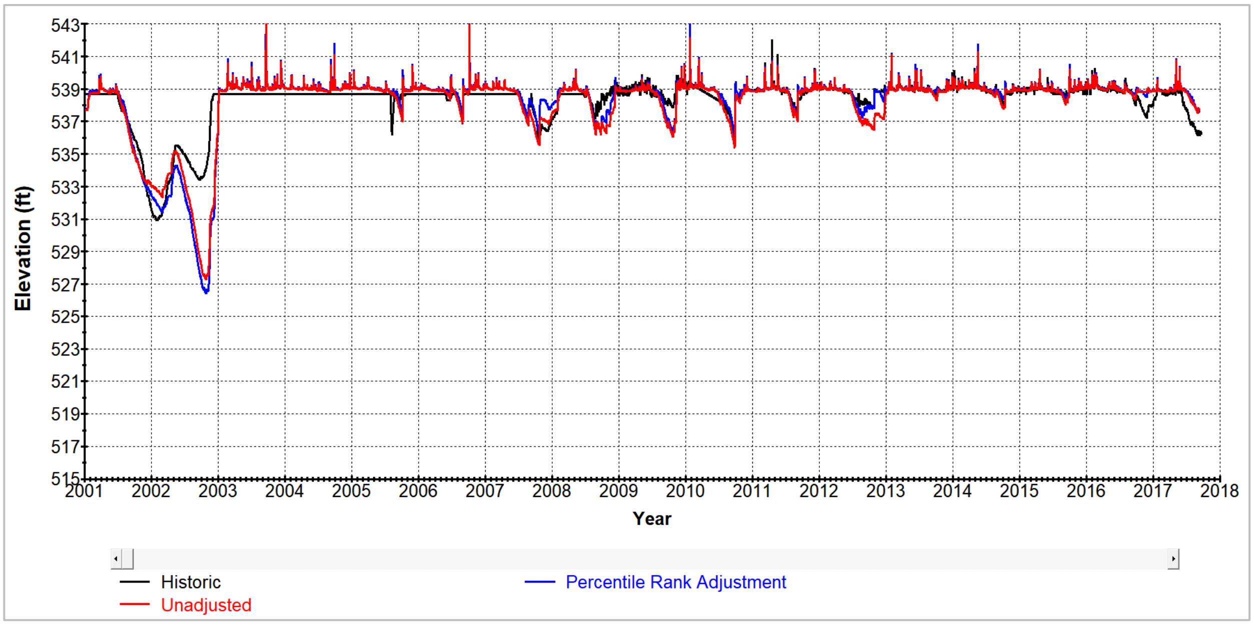


Figure 7. Beaver Creek Reservoir Final Verification Results

Figure 8: Original Model Verification Using Spillway Outflow Meter

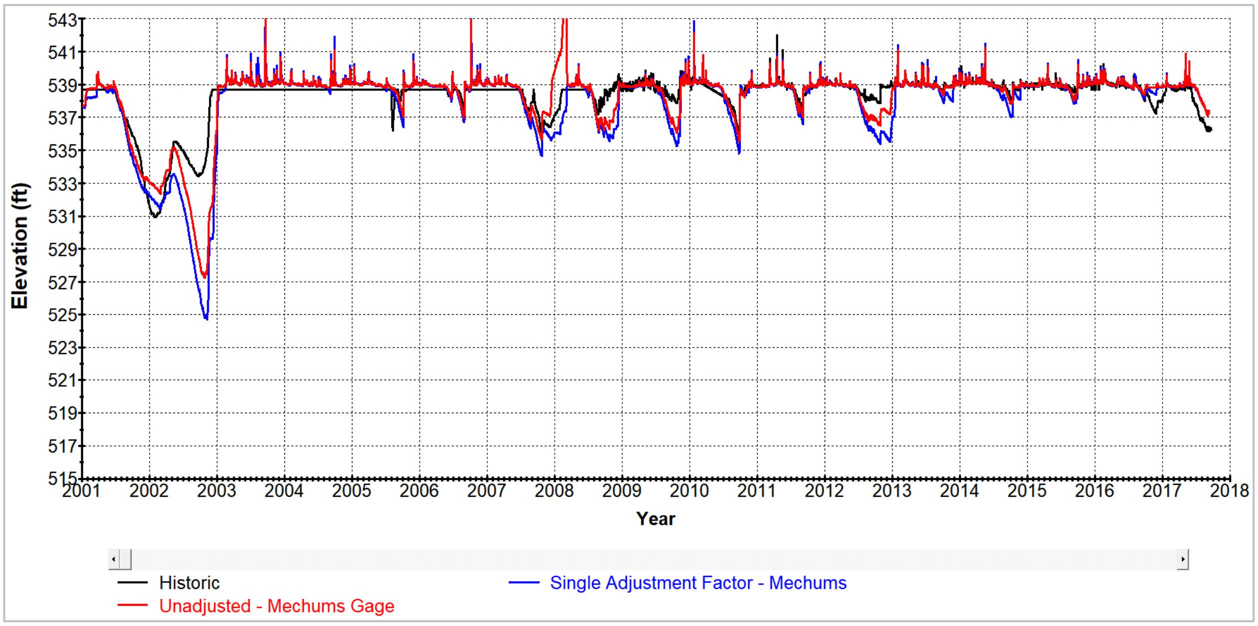


Figure 9. Beaver Creek Reservoir Verification with a Single Inflow Adjustment Factor

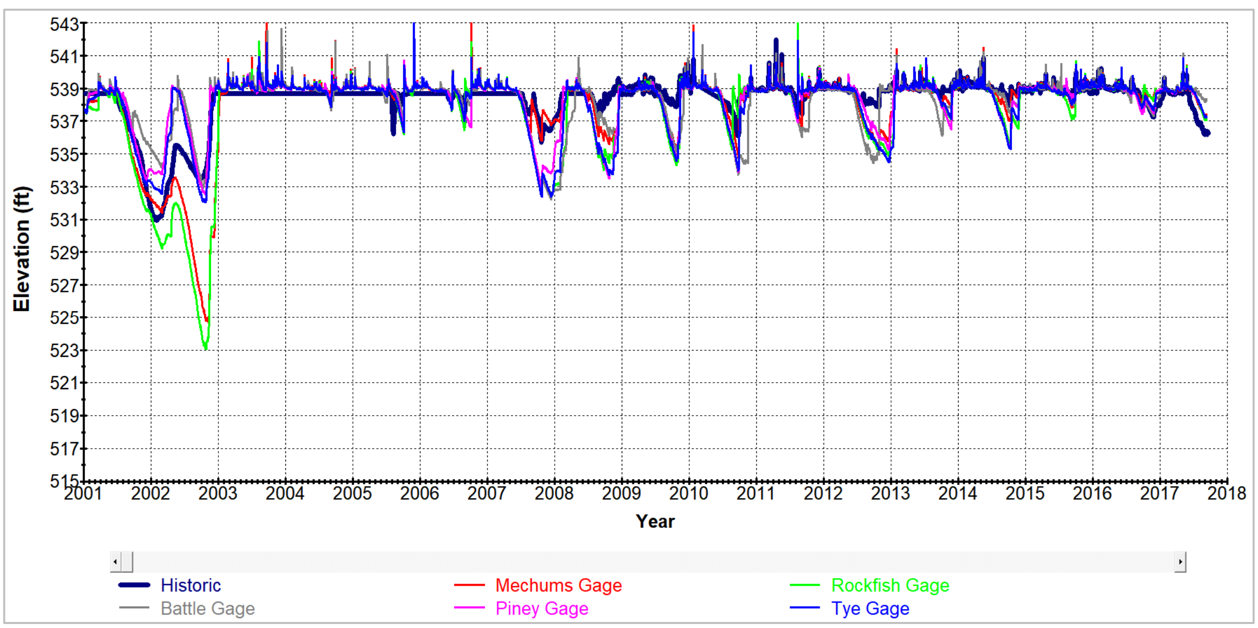


Figure 10. Beaver Creek Reservoir Verification with Various Reference Gages

3.2 Safe Yield Results

The safe yield of the system was computed with a variety of operating and physical assumptions. The details of the variations are covered in Section 2, and Table 4 summarizes the results of each combination of assumptions. In addition to different operational assumptions, two versions of lowest usable storage (or, dead storage) were tested – one representing the invert of the lowest usable gate on the intake structure (518.7 ft), and one for the top of the sediment pool (515.4 ft). While the current intake structures would realistically limit withdrawals down to the Gate 4 intake, a new intake in the future could allow access to water down to the top of the sediment pool.

For all scenarios the SY drought was based on 2001-2002 hydrology (which was identified as the critical drought when running the model for the entire record of hydrology 1925 – 2017). The yield of 1.65 mgd is sufficient to meet the projected annual average daily demand for Crozet in year 2075.

Table 4. Beaver Creek Reservoir SY Results

60 Day Reserve?	Minimum Release?	Demand Reductions?	SY with Specified Dead Storage Level (mgd)	
			518.7 ft (Gate 4 invert)	515.4 ft (Top of sed. pool)
No	No	No	1.64	1.71
	Yes	No	1.49	1.56
Yes	No	No	1.47	1.54
	Yes	No	1.34	1.40
		Yes	1.56	1.65

Appendix B: Pump Draw-Down Test Calculation Summary

Drawdown Test Results

Pump 3 | 1118 gpm Average Flow
280.7 ft Average TDH

	Tank Level	Time	18-Apr	
1	22.7 ft	11:02 AM	suction	637.25
	21.8 ft	11:16 AM	discharge	916.77
	0.9 ft	13.78 min		
	15000.00 gal	TDH =	280 ft	
	1089 gpm			

	Tank Level	Time		
2	21.8 ft	11:16 AM		
	21 ft	11:30 AM		
	0.8 ft	14.47 min		
	13333.34 gal			
	921 gpm			

	Tank Level	Time	18-Apr	
3	21 ft	11:30 AM	suction	635.75
	20.5 ft	11:38 AM	discharge	917.69
	0.5 ft	7.22 min		
	8333.34 gal	TDH =	281.9 ft	
	1153 gpm			

	Tank Level	Time	18-Apr	
4	23.3 ft	10:56 AM	suction	636.90
	20.5 ft	11:38 AM	discharge	917.43
	2.8 ft	42.00 min		
	46666.68 gal	TDH =	280.5 ft	
	1111 gpm			

Pump 1 | 497 gpm Average Flow
259.0 ft Average TDH

	Tank Level	Time	19-Apr	
1	26.6 ft	11:54 AM	suction	641.2
	25.8 ft	12:20 PM	discharge	900.23
	0.8 ft	26.98 min		
	13333.34 gal	TDH =	259.0	
	494 gpm			

	Tank Level	Time		
2	25.9 ft	12:20 PM		
	25.4 ft	12:37 PM		
	0.5 ft	16.21 min		
	8333.34 gal			
	514 gpm			

	Tank Level	Time		
3	26.6 ft	11:54 AM		
	25.3 ft	12:38 PM		
	1.3 ft	44.84 min		
	21666.67 gal			
	483 gpm			

Pumps 1 & 2 | 990 gpm Average Flow
277.5 ft Average TDH

	Tank Level	Time	18-Apr	
1	20.5 ft	11:42 AM	suction	634.35
	18.2 ft	12:21 PM	discharge	911.44
	2.3 ft	38.99 min		
	38333.34 gal	TDH =	277.1 ft	
	983 gpm			

	Tank Level	Time	18-Apr	
2	20.5 ft	11:42 AM	suction	635.4
	20.2 ft	11:48 AM	discharge	908.9
	0.3 ft	6.50 min		
	5000.00 gal			
	769 gpm			

	Tank Level	Time	18-Apr	
3	20.2 ft	11:48 AM	suction	634.75
	19.3 ft	12:03 PM	discharge	911.97
	0.9 ft	14.50 min		
	15000.00 gal	TDH =	277.2	
	1035 gpm			

	Tank Level	Time	18-Apr	
4	19.3 ft	12:03 PM	suction	633.85
	18.4 ft	12:18 PM	discharge	911.92
	0.9 ft	15.74 min		
	15000.00 gal	TDH =	278.1	
	953 gpm			

Appendix C: New Crozet WTP Finished Water Pump Performance Curves

SEH

Building a Better World for All of Us®

SHOP DRAWING TRANSMITTAL

RIVAN 141766 — SEH File Number

78.00 — File Location

October 30, 2017 — Date

To: John Anderson
Anderson Construction, Inc.
PO Box 10053
Lynchburg, VA 24506

To:

Project: RFB No. 331 – CZWTP FWPS

Owner: Rivanna Water & Sewer Authority

We are:

☒ Enclosing ☐ Sending under separate cover

The following submittals have been reviewed by this office and are being returned for action as noted.

Specification Section No.	Item	Supplier/Manufacturer	No. of Copies Received	Action Taken: Reviewed	Action Taken: Furnish as Corrected	Action Taken: Revise and Resubmit	Action Taken: Rejected
33 28 31	Vertical Turbine Pump – Factory Curves	Flowserve	ecopy	X			

Short Elliott Hendrickson Inc., 3535 Vadnais Center Drive, St. Paul, MN 55110-5196
SEH is 100% employee-owned | **sehinc.com** | 651.490.2000 | 800.325.2055 | 888.908.8166 fax

Remarks:

☒ **Reviewed** ☐ **Furnish as Corrected**	Review does not extend to quantities, dimensions, fabrication processes, construction means or methods, coordination of the work or safety features. Comments or corrections made do not relieve Contractor from compliance with the drawings and specifications.
☐ **Revise & Resubmit** ☐ **Rejected/Resubmit**	In resubmitting, identify REVISIONS made.

Review of this submittal is expressly limited as provided in the Contract Documents and is only to determine compliance with information given in the Contract Documents and conformance with the design concept of the completed project. Review does not affect contract price or time.

SHORT ELLIOTT HENDRICKSON, INC.

By: BJW

Date: 10/30/17

SEH

☐ No action taken. Submittals must first be approved and stamped by the Contractor.

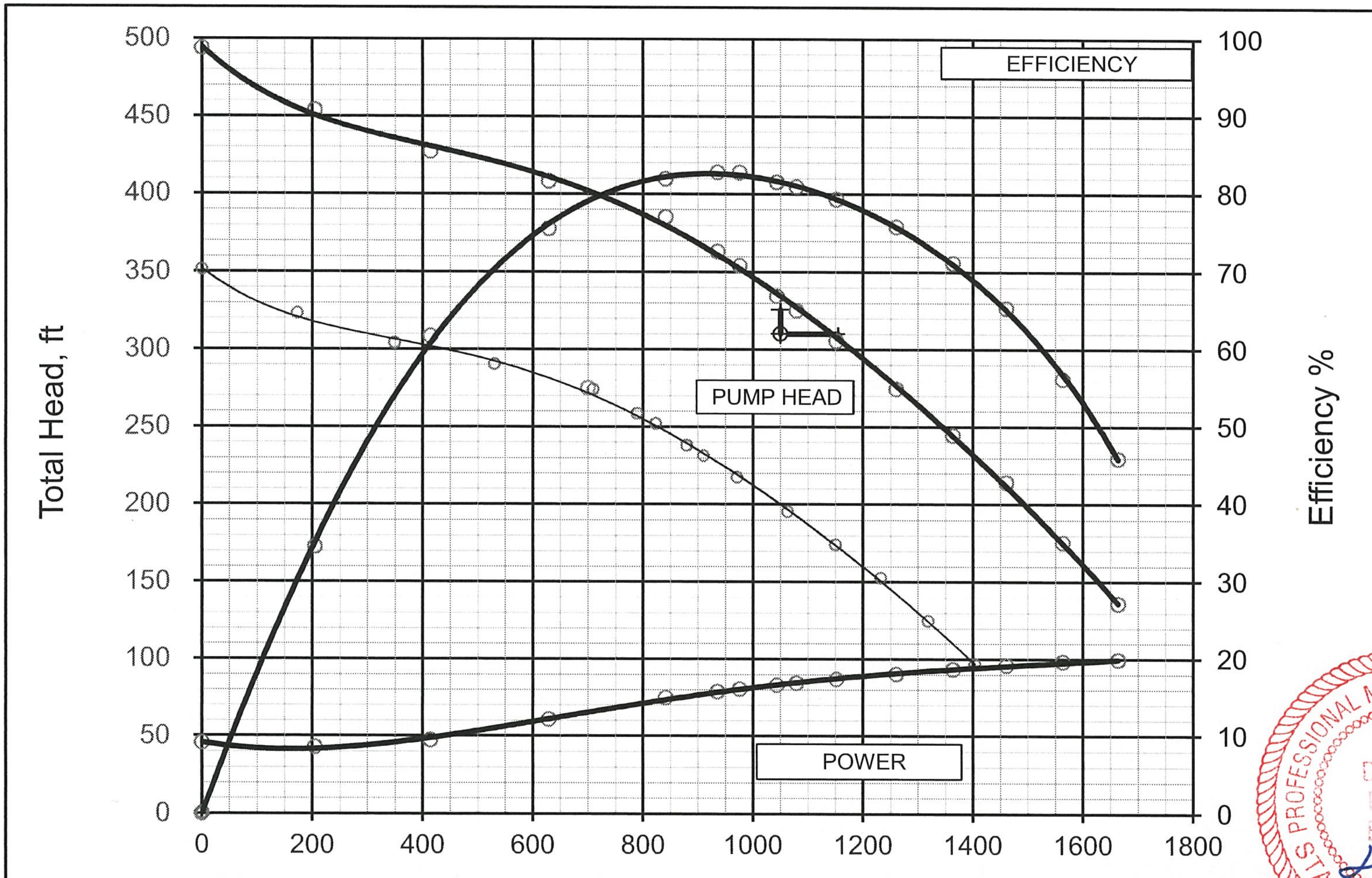
By: Brad Weiss

Distribution:

Owner ______ Contractor ______ SEH File ______ SEH Proj. Rep. ______ Other ____________________

Pump # 1

Tested trim 9.33



Reduced Speeds
1493
0
0
0
0
0
0

BHP
1.00
SpGr

Power
125
75
25

Customer specified margins

Capacity USGPM

PROFESSIONAL MECHANICAL ENGINEER
STATE OF NEBRASKA

Design Conditions		
1050 USGPM	310 Ft.	

Certified Performance Test

By: Karl Launchbaugh Date: 20-Oct-17

FLOWSERVE

MODEL	12EMM
Speed	1770
Test and Curve Number	
12EMMAE23	

FLOWSERVE

PERFORMANCE TEST RESULTS

		Pump #	1
TEST NUMBER:	12EMMAE23	BOWL SIZE/TYPE:	12EM
DATE:	10/20/2017	BOWL MATERIAL:	CL 30 Cast Iron ID2013
TESTED BY:	GARY LESPREANCE	IMPELLER TYPE / DIA.:	M 9.33
TESTED FOR:	Anderson Construction	IMPELLER MATERIAL:	316L SS CF3M ID2102
WITNESSED BY:	DEREK PFEIFER	IMPELLER SETTING:	0.0833
SERIAL NUMBER:	1710NSH02418-1	NUMBER OF STAGES:	5
S.O. NUMBER:	SH02418	DESIGN FLOW, GPM:	1050
CUSTOMER MOTOR HP:	125	DESIGN HEAD, FEET:	310
TEST MOTOR:	100HP @ 1200	THRUST CONSTANT:	5.43

DATA CORRECTED TO 1770 RPM AND 1.00 S.G.

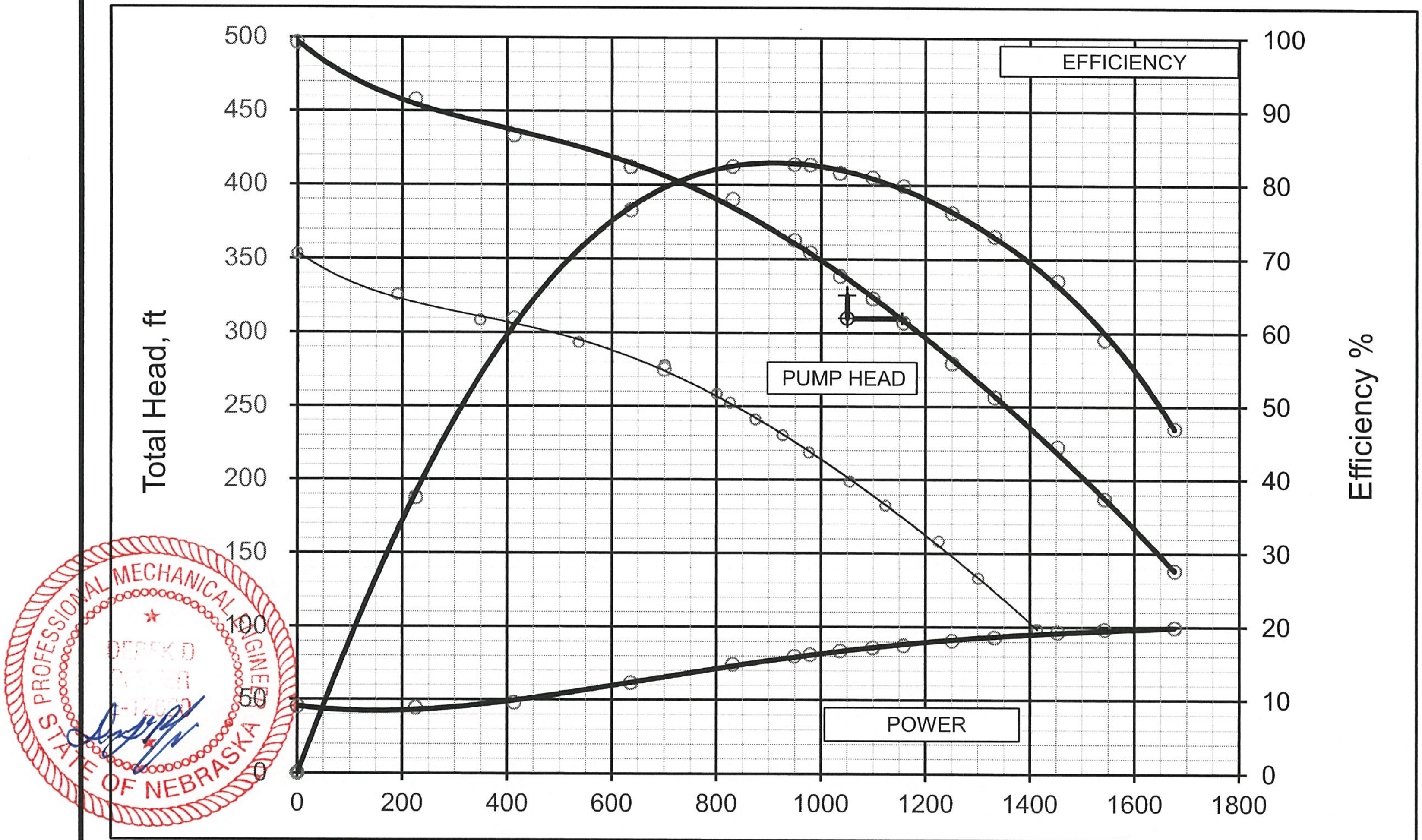
FLOW (GPM)	PUMP HEAD (FT.)	EFF. (%)	POWER (HP)	HEAD/STAGE (FT.)	POWER/STAGE (HP)
0	493.8	0.0	70.55	98.8	14.11
204	453.9	34.5	67.92	90.8	13.58
414	427.2	61.7	72.36	85.4	14.47
629	408.1	75.5	85.78	81.6	17.16
841	385.2	81.9	99.85	77.0	19.97
936	363.3	82.8	103.68	72.7	20.74
976	354.1	82.7	105.44	70.8	21.09
1043	334.4	81.5	108.05	66.9	21.61
1079	325.1	80.9	109.46	65.0	21.89
1152	305.5	79.3	112.04	61.1	22.41
1260	274.5	75.8	115.24	54.9	23.05
1363	244.6	71.2	118.35	48.9	23.67
1461	214.2	65.3	120.94	42.8	24.19
1563	175.4	56.2	123.30	35.1	24.66
1664	136.0	45.9	124.51	27.2	24.90

PROFESSIONAL MECHANICAL ENGINEER
DEREK D
PFEIFER
STATE OF NEBRASKA

Pump # 2

Tested trim 9.33

Reduced Speeds
1493
0
0
0
0
0
0



BHP
1.00
SpGr
Power
125
75
25

Customer specified margins

Capacity USGPM

Design Conditions		
1050 USGPM	310 Ft.	

Certified Performance Test

By: Karl Launchbaugh Date: 20-Oct-17

FLOWSERVE

MODEL	12EMM
Speed	1770
Test and Curve Number	
12EMMAE24	

FLOWSERVE

PERFORMANCE TEST RESULTS

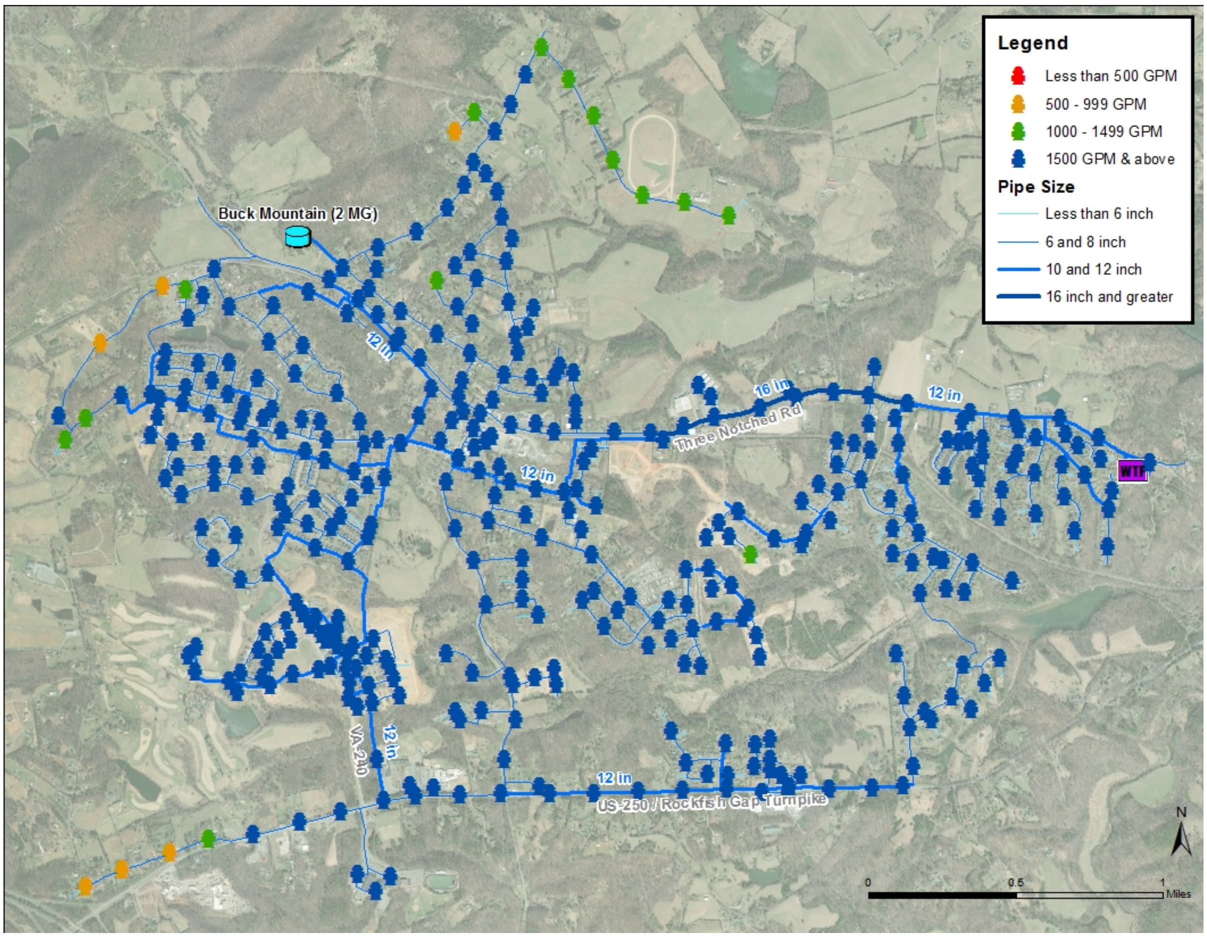
		Pump #	2
TEST NUMBER:	12EMMAE24	BOWL SIZE/TYPE:	12EM
DATE:	10/20/2017	BOWL MATERIAL:	CL 30 Cast Iron ID2013
TESTED BY:	GARY LESPREANCE	IMPELLER TYPE / DIA.:	M 9.33
TESTED FOR:	Anderson Construction	IMPELLER MATERIAL:	316L SS CF3M ID2102
WITNESSED BY:	________________	IMPELLER SETTING:	0.0833
SERIAL NUMBER:	1710NSH02418-2	NUMBER OF STAGES:	5
S.O. NUMBER:	SH02418	DESIGN FLOW, GPM:	1050
CUSTOMER MOTOR HP:	125	DESIGN HEAD, FEET:	310
TEST MOTOR:	100HP @ 1200	THRUST CONSTANT:	5.43

DATA CORRECTED TO 1770 RPM AND 1.00 S.G.

FLOW (GPM)	PUMP HEAD (FT.)	EFF. (%)	POWER (HP)	HEAD/STAGE (FT.)	POWER/STAGE (HP)
0	496.6	0.0	70.23	99.3	14.05
225	457.7	37.5	69.42	91.5	13.88
413	433.3	61.9	73.00	86.7	14.60
636	412.0	76.6	86.36	82.4	17.27
831	390.3	82.5	99.25	78.1	19.85
949	363.0	82.8	105.04	72.6	21.01
979	354.5	82.7	105.99	70.9	21.20
1037	338.5	81.7	108.51	67.7	21.70
1099	323.5	81.0	110.78	64.7	22.16
1158	307.0	79.9	112.40	61.4	22.48
1251	279.3	76.3	115.62	55.9	23.12
1332	256.4	73.1	117.92	51.3	23.58
1453	222.2	67.1	121.52	44.4	24.30
1543	187.0	59.1	123.31	37.4	24.66
1676	138.1	46.9	124.53	27.6	24.91

PROFESSIONAL MECHANICAL ENGINEER
STATE OF NEBRASKA

Appendix D: Available Fire Flows at Distribution System Hydrants



Appendix E: Cost Estimates for Recommended Infrastructure Improvements

Hazen

Rivanna Water and Sewer Authority
Drinking Water Infrastructure Plan - Crozet Area
Project Cost Estimates

Item Description	Quantity	Unit	Unit Cost	Total Cost	Notes
Raw Water Pump Station and Intake	1	LS	$ 2,715,000	$ 2,715,000	Cost is average cost based on Figure 29-5, Pumping Station Cost Curve, Sanks
Microtunneling	600	LF	$ 621	$ 372,600	Assumes 24" diameter and includes microtunnel wet retrieval; Oregon DOT
Access Road	6083	SY	$ 120	$ 730,000	Assumes 3,650 linear feet of road, 15 ft wide
Raw Water Transmission Main - Pipe	6900	LF	$ 49	$ 338,100	Assumes 16" diameter; based on ACIPCO pipe cost matrix
Raw Water Transmission Main - Joints	115	EA	$ 340	$ 39,100	Assumes 30% restrained joints; 16" fast-grip gasket
Raw Water Transmission Main - Valves	2	EA	$ 12,000	$ 24,000	
Erosion Control	6900	LF	$ 10	$ 69,000	
Installation	43	crew day	$ 6,500	$ 280,313	Assumes 160 lf installed per day
Project Management	7	months	$ 40,000	$ 280,000	Based on installation days (160 lf per day), 5 days per week plus 90d mob and 60d closeout
Permanent Easement	1	LS	$ 36,000	$ 36,000	
Subtotal				*$ 4,884,113*	
General Contractor OH&P (20%)				$ 976,823	
General Conditions (5%)				$ 244,206	
Contingency (30%)				$ 1,465,234	
			Estimated Construction Cost	*$ 7,570,374*	
Eng./Program/Constr./Misc. (18%)				$ 1,362,667	
			Estimated Total Program Cost	**$ 8,933,042**	

Hazen

Rivanna Water and Sewer Authority
Drinking Water Infrastructure Plan - Crozet Area
Project Cost Estimates

Item Description	**Quantity**	**Unit**	**Unit Cost**	**Total Cost**	**Notes**
Finished Water Pump	1	EA	$ 200,000	$ 200,000	
Electrical and I&C	1	LS	$ 50,000	$ 50,000	
Subtotal				*$ 250,000*	
General Contractor OH&P (20%)				$ 50,000	
General Conditions (5%)				$ 12,500	
Contingency (30%)				$ 75,000	
			Estimated Construction Cost	*$ 387,500*	
Eng./Program/Constr./Misc. (18%)				$ 69,750	
			Estimated Total Program Cost	**$ 457,250**	

Item Description	Quantity	Unit	Unit Cost	Total Cost	Notes
Rapid Mix	1	LS	$ 50,000	$ 50,000	
New Sed Basin w/ Plates	1	LS	$ 500,000	$ 500,000	
New Filter	1	LS	$ 425,000	$ 425,000	
Chemical Feed Upgrades	1	LS	$ 100,000	$ 100,000	
GAC	1	LS	$ 1,650,000	$ 1,650,000	
Residuals Handling	1	LS	$ 50,000	$ 50,000	
Electrical and I&C	1	LS	$ 681,250	$ 681,250	
Misc. (Yard Piping, Paving, SWM, etc.)	1	LS	$ 150,000	$ 150,000	
Subtotal				*$ 3,606,250*	
General Contractor OH&P (20%)				$ 721,250	
General Conditions (5%)				$ 180,313	
Contingency (30%)				$ 1,081,875	
			Estimated Construction Cost	*$ 5,589,688*	
Eng./Program/Constr./Misc. (18%)				$ 1,006,144	
			Estimated Total Program Cost	**$ 6,595,831**	

Rivanna Water and Sewer Authority
Drinking Water Infrastructure Plan - Crozet Area
Project Cost Estimates

Item Description	Quantity	Unit	Unit Cost	Total Cost	Notes
FW Transmission Main - Pipe	4200	LF	$ 60	$ 252,000	Assumes 18" diameter; based on ACIPCO pipe cost matrix
FW Transmission Main - Joints	70	EA	$ 405	$ 28,350	Assumes 30% restrained joints; 18" fast-grip gasket
FW Transmission Main - Valves	3	EA	$ 12,000	$ 36,000	Assumes valve every 2000 feet (start, middle, end) to isolate for disinfection per VDH
Erosion Control	4200	LF	$ 10	$ 42,000	
Installation	26	crew day	$ 6,500	$ 170,625	Assumes 160 lf installed per day
Project Management	6.2	months	$ 40,000	$ 249,000	Based on installation days (160 lf per day), 5 days per week plus 90d mob and 60d closeout
Roadway	1867	SY	$ 140	$ 261,333	
Subtotal				*$ 1,039,308*	
General Contractor OH&P (20%)				$ 207,862	
General Conditions (5%)				$ 51,965	
Contingency (30%)				$ 311,793	
			Estimated Construction Cost	*$ 1,610,928*	
Eng./Program/Constr./Misc. (18%)				$ 289,967	
			Estimated Total Program Cost	**$ 1,900,895**	

Hazen

Rivanna Water and Sewer Authority
Drinking Water Infrastructure Plan - Crozet Area
Project Cost Estimates

Item Description	**Quantity**	**Unit**	**Unit Cost**	**Total Cost**	**Notes**
FW Transmission Main - Pipe	3030	LF	$ 28	$ 84,840	Assumes 12" diameter; based on ACIPCO pipe cost matrix
FW Transmission Main - Joints	51	EA	$ 150	$ 7,575	Assumes 30% restrained joints; 12" fast-grip gasket
FW Transmission Main - Valves	3	EA	$ 2,500	$ 7,500	Assumes valve every 2000 feet (start, middle, end) to isolate for disinfection per VDH
Erosion Control	3030	LF	$ 10	$ 30,300	
Installation	19	crew day	$ 6,500	$ 123,094	Assumes 160 lf installed per day
Project Management	5.9	months	$ 40,000	$ 235,350	Based on installation days (160 lf per day), 5 days per week plus 90d mob and 60d closeout
Roadway	1347	SY	$ 140	$ 188,533	Assumes 4ft trench width and full depth paving replacement; $120/sy plus $20/sy traffic ctrl
Subtotal				*$ 677,192*	
General Contractor OH&P (20%)				$ 135,438	
General Conditions (5%)				$ 33,860	
Contingency (30%)				$ 203,158	
			Estimated Construction Cost	*$ 1,049,648*	
Eng./Program/Constr./Misc. (18%)				$ 188,937	
			Estimated Total Program Cost	**$ 1,238,584**	

Hazen

Rivanna Water and Sewer Authority
Drinking Water Infrastructure Plan - Crozet Area
Project Cost Estimates

Item Description	**Quantity**	**Unit**	**Unit Cost**	**Total Cost**	**Notes**
FW Transmission Main - Pipe	3750	LF	$ 28	$ 105,000	Assumes 12" diameter; based on ACIPCO pipe cost matrix
FW Transmission Main - Joints	63	EA	$ 150	$ 9,375	Assumes 30% restrained joints; 12" fast-grip gasket
FW Transmission Main - Valves	3	EA	$ 2,500	$ 7,500	Assumes valve every 2000 feet (start, middle, end) to isolate for disinfection per VDH
Erosion Control	3750	LF	$ 10	$ 37,500	
Installation	23	crew day	$ 6,500	$ 152,344	Assumes 160 lf installed per day
Project Management	6.1	months	$ 40,000	$ 243,750	Based on installation days (160 lf per day), 5 days per week plus 90d mob and 60d closeout
Roadway	1667	SY	$ 140	$ 233,333	Assumes 4ft trench width and full depth paving replacement; $120/sy plus $20/sy traffic ctrl
Subtotal				*$ 788,802*	
General Contractor OH&P (20%)				$ 157,760	
General Conditions (5%)				$ 39,440	
Contingency (30%)				$ 236,641	
			Estimated Construction Cost	*$ 1,222,643*	
Eng./Program/Constr./Misc. (18%)				$ 220,076	
			Estimated Total Program Cost	**$ 1,442,719**	