HYSR

An Assessment of the Potential Impacts on the Firm Yield of Surface Water Supply Systems in Southeast Massachusetts Due to Climate Change

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1.1 Introduction

The first purpose of this report is to describe in depth the mathematical techniques used to estimate the Firm Yield of the surface water supply systems of the City of Fall River, the Town of Somerset and the Stone Bridge Fire District of Tiverton, RI. The second purpose is to present estimates of the Firm Yield for each of the systems as determined using estimated daily streamflows and the climate observed during the study Base Period of 1950-80. Using each system's present-day system operating rules, HYSR examined reservoir behavior and reliability for this period which bounds the 1960s Drought-of-Record. The third purpose of this study is to report on the results of an analysis of how the Base Period Firm Yield for each of these three communities might be significantly enhanced by modifying present reservoir operating rules and new/replacement water supply system infrastructure. The final purpose is to describe the results of estimating the Firm Yield of each system under the influence of Global Warming/Climate Change, as projected by five separate CMPI6 (Coupled Model Intercomparison Project Phase 6) General Circulation Models (GCMs). The 1950-80 Base Period climate and streamflows are rescaled using the GCM predicted climate for the "mid-century" period of 2025-2055 and for the "late century" climate period of 2070-2100.

1.2 A Brief Overview of the Study Water Supply Systems

Referring to Figure 2.1, the Fall River surface water supply system consists of three sources: North Watuppa Pond, South Watuppa Pond and the Copicut Reservoir. The Fall River water treatment filtration plant is located on the west side of North Watuppa Pond. North and South Watuppa Ponds are naturally occurring glacial kettle ponds, and comprise the second and third largest naturally occurring water bodies in the Commonwealth. They are physically separated by an earth fill dam which was built on a naturally occurring isthmus known at the Narrows. South Watuppa Pond drains to the Quequechan River which originally provided water and power to various Fall River textile and industrial mills. The Quequechan River discharges to Mount Hope Bay, in the area of the Taunton River estuary; all three considered being within the Mount Hope Bay watershed.

The Copicut Reservoir is sited on the Copicut River in Dartmouth. The Copicut Reservoir and Copicut River are located in the Buzzards Bay watershed. Planning for the Copicut Reservoir began during the mid-1950s and its construction was completed in 1972. Depending on water levels in North Watuppa Pond, and the time of the year, water is pumped from the Copicut Reservoir to a brook located east of North Watuppa Pond which drains into North Watuppa.

The Somerset surface water supply system consists of two parts: the Somerset Reservoir, which was constructed in 1966 and the Segreganset River pumping station, which was also constructed in 1966. This system is located in the Taunton River watershed. The US Geological Survey sited a daily streamgage on the Segreganset River about a mile upstream of the pumping station. The Town of Somerset uses the gage to govern its withdrawals from the river. The pumping station is only employed during November through May and only if the measured flows at the gaging station exceed a threshold. A water treatment plant is located next to the Somerset Reservoir.

The Stone Bridge Fire District is located in Tiverton, RI. Its sole source is Stafford Pond, which is owned by Fall River. A naturally occurring glacial kettle pond, like North and South Watuppa Ponds, it was developed as a local water supply in the 1940s. Stafford Pond and its watershed are located within the South Watuppa Pond watershed and therefore are also considered to be within the Mount Hope Bay watershed. The Stone Bridge water treatment plant is located beside Stafford Pond.

1.3 Prior Study

Quite recently, the City of Fall River was funded for a very timely study by the Municipal Vulnerability Program (MVP) grant program, which is administered by the Massachusetts Executive Office of Energy and Environmental Affairs (EOEEA). The "*Regional Emergency Water System Interconnectivity Analysis-Report of Findings*" was conducted by Woodard & Curren, Inc. (2021) with the support of Fall River and the local communities. The focus of the study was on the Dighton, Fall River, Somerset and Swansea water supply systems. A major goal of the study was to *"….examine the limitations of existing system connections to transfer water among the four communities to create redundancy*." During the course of this study, existing inter-municipal connection pipelines were surveyed and field-tested for leaks. Separate system, computer-based hydraulic models were linked to examine the adequacy of water pressure for both potable use supply and fire-fighting. A second goal of the study was to "… *assess the volume of water available within each system under various drought and demand conditions, and the hydraulic requirements for access*." Although the study reported on the "hydraulic requirements for access", the "volume of water within each system under various drought and demand conditions" was not. The present study by HYSR accomplishes this goal.

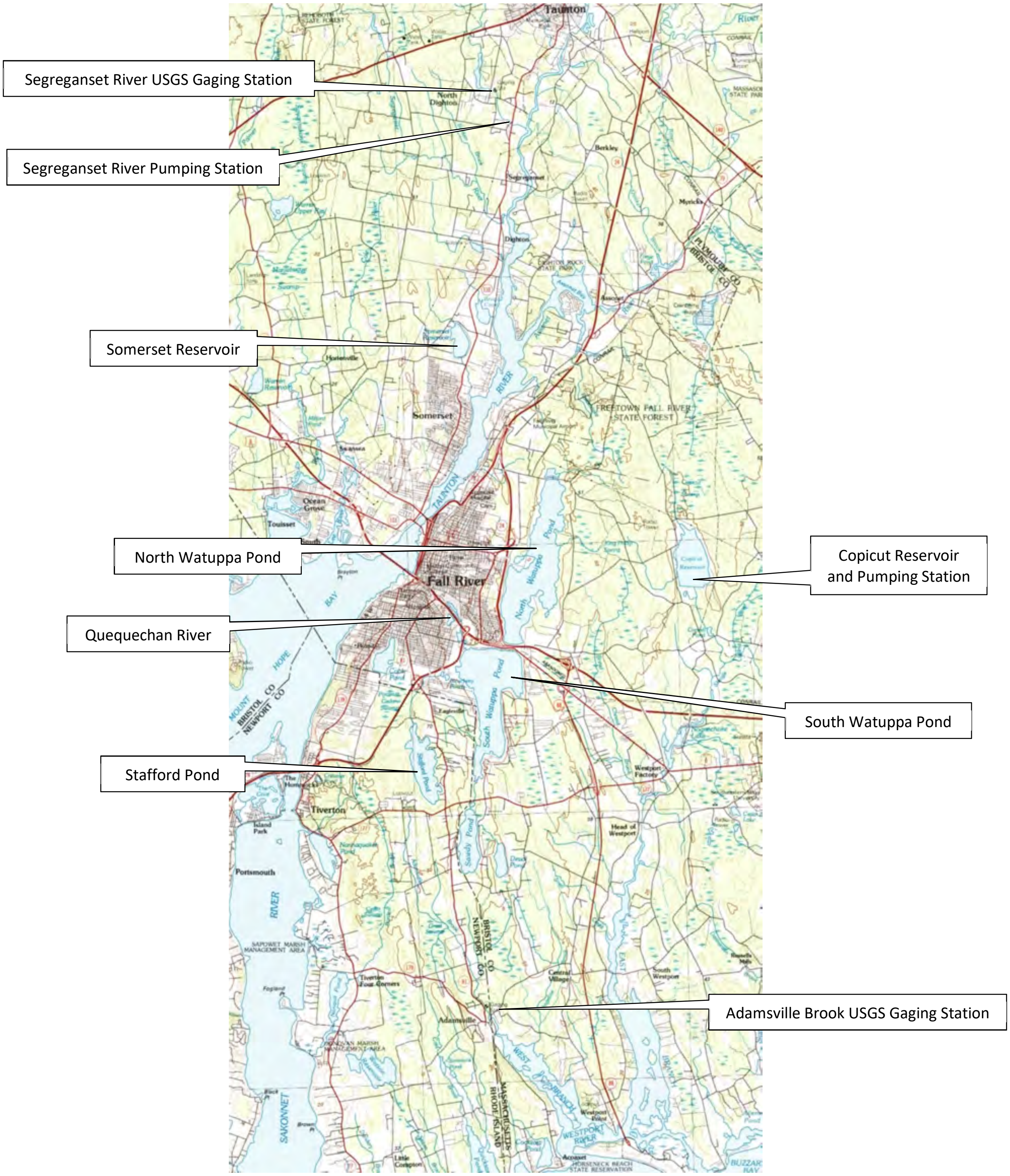

Figure 1.3.1 Surface Water Supply Sources, Pumping Stations and USGS Streamgages

1.4 Potential Impacts Due to Climate Change

What's interesting to note is that clear evidence of change has been recorded by the Blue Hills Weather Observatory, which is located in Milton, MA. The Blue Hills Observatory is the oldest continuous weather observatory in the North America and it was established in 1884 by a Harvard University professor of meteorology. At that time, meteorology was a very new science, having its roots in Norway. Boston temperature records go back to the 1830s. Precipitation and snowfall among other weather variables and indices records from the Blue Hill Observatory have been collected and maintained since 1885.

Various Figures provided below illustrate how various aspects of the regional climate have changed over time. All these graphs shown here were produced by M. Iacono (2024), Chief Scientist, Blue Hill Observatory. Each graph shows a blue curve and a red curve. The blue "10-year moving average) is estimated by averaging all the daily values observed during a continuous 10-year period. For example, the 1900 10-year moving average is calculated from the daily data for 1900, 1901, 1902, … , 1908, 1909. The red 30-year moving average is calculated by averaging all the daily values observed during a continuous 30-year period. Similar to the 10-year moving average, the 30-year moving average for 1900 is calculated from the daily data for 1900, 1901, 1902, … , 1928, 1929. These moving averages are simply a form of curve smoothing so one's eye isn't distracted by single year observations/anomalies. The National Oceanic and Atmospheric Administration reports 30-year averages every 10 years as a way to track climate changes and trends.

Figure 1.4.1 shows the Boston air temperature since 1830. M. Iacono (2024) reports that since 1884, the annual average daily air temperature has risen by 4.2 ^{o}F which is a rate of 0.32 ^{o}F/decade. The upward trend is statistically significant at the 99% confidence level, which means there's only a 1% chance that there is no upward trend. Just above the bottom horizontal axis are asterisks, *, which indicate the dates of major volcanic eruptions. The lowest annual average temperature observed occurred in 1875 which is when major eruptions occurred in both Iceland and Alaska. The second lowest year, 1835 is when a volcano in Nicaragua erupted. A volcanic eruption pours vast quantities of smoke and dust into the atmosphere, effectively reflecting incoming solar radiation, consequently cooling the earth a period of one or more years.

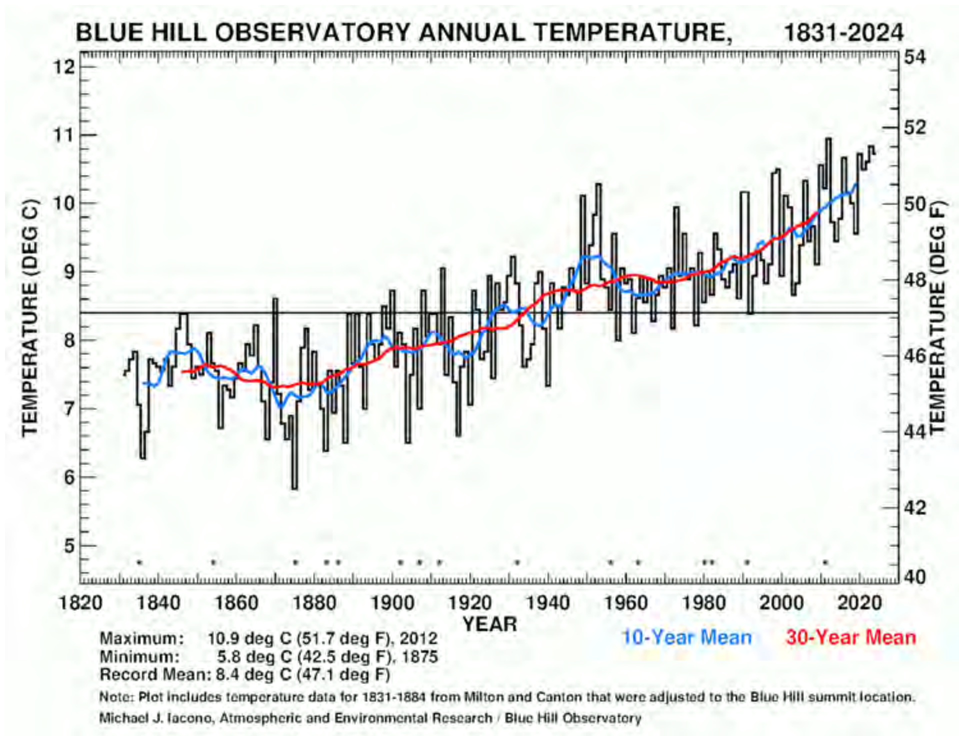

Figure 1.4.1 Annual Air Temperature at Blue Hill Observatory

As shown in Figure 1.4.2, globally, the annual average daily air temperature has risen by 2.0 ^{o}F since 1880. Nationally, the annual average daily air temperature has risen by 2.3 ^{o}F since 1895. Since 1885, the air temperature in southern New England has risen by about 4.2 ^{o}F. Figure 1.4.3 shows the seasonal temperature trends at the Blue Hill Observatory. M. Iacono (2024) reports a 4.5 ^{o}F rise in winter temperatures since 1885. The summer air temperatures have risen 4.3 ^{o}F since 1885. While not shown, the spring and fall air temperatures have respectively risen by 3.9 ^{o}F and 3.7 ^{o}F since 1885. Figure 1.4.4 shows the total annual precipitation observed at the Blue Hill Observatory. The upward trend is 0.67 in/decade with a total change in precipitation of 8.7 inches since 1885. While highly variable from year-to-year, M. Iacono (2024) reports that the trend is statistically significant. Figure 1.4.5 shows total annual snowfall at Blue Hill Observatory. A slight positive trend is seen. Figure 1.4.6 shows the rise in annual vapor pressure at Blue Hill. Vapor pressure is a measure of the moisture content of the atmosphere and contributes to about 1% of the total surface air pressure. As the atmosphere's air temperature rises, the moisture holding power of the atmosphere rises too. M. Iacono (2024) reports that it has increased by about 5% per ^{o}C of air temperature rise since the mid-1950s.

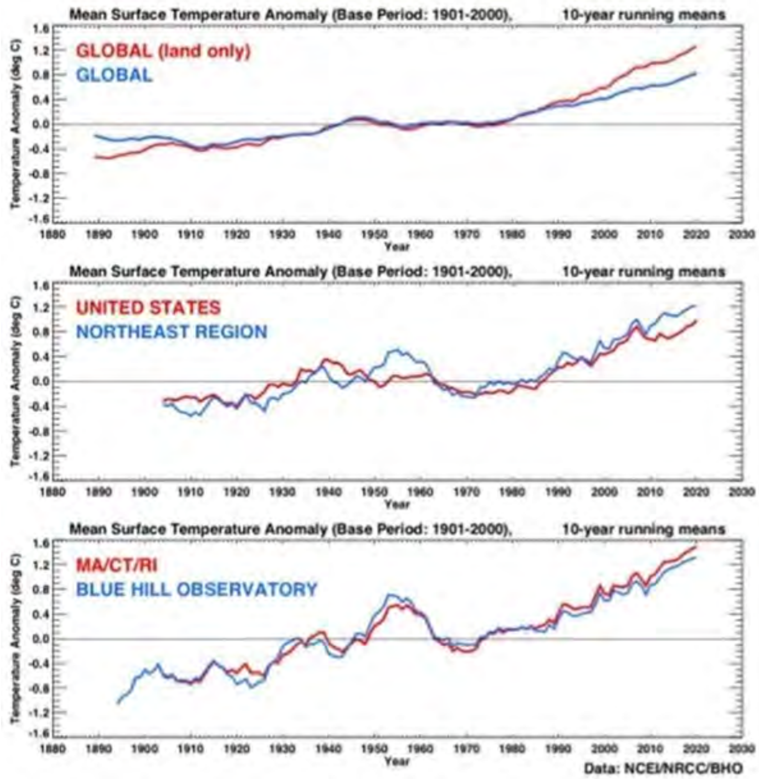

Figure 1.4.2 Annual Air Temperature Global, US, Northeast US, MA/CT/RI and Blue Hill Observatory

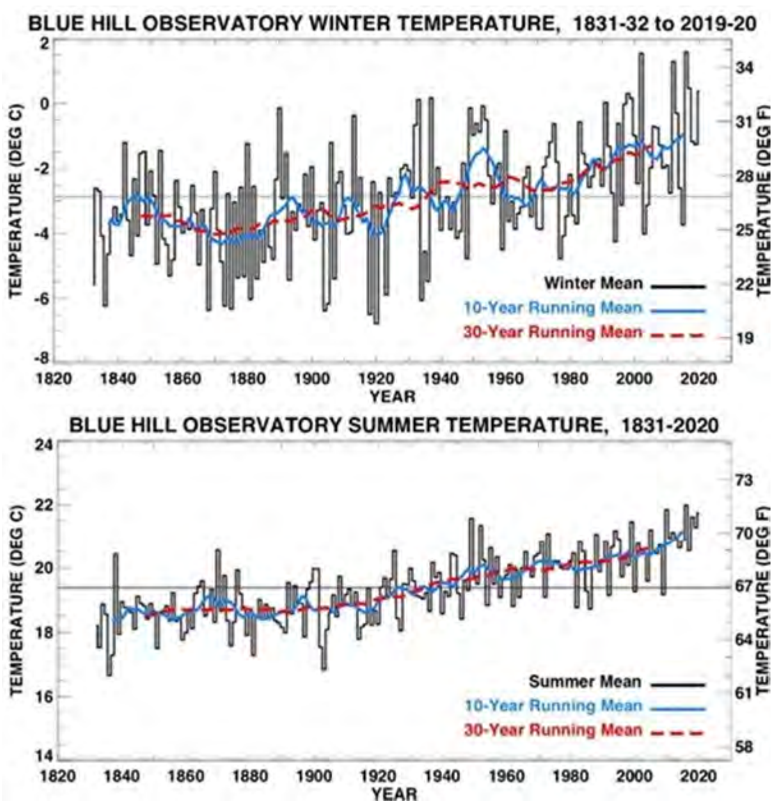

Figure 1.4.3 Summer and Winter Air Temperature at Blue Hill Observatory

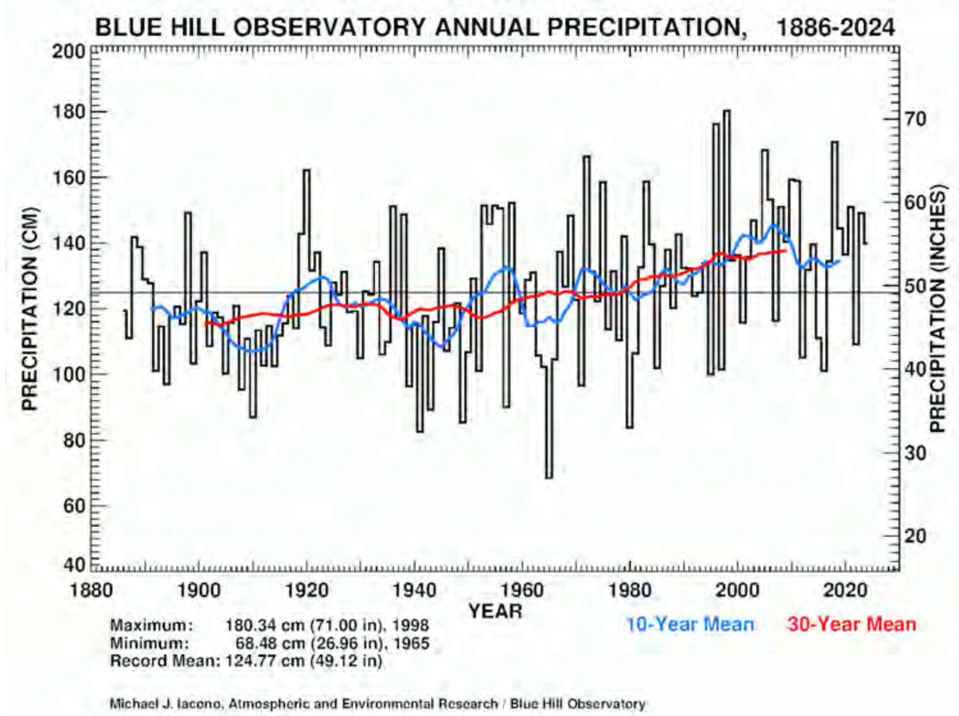

Figure 1.4.4 Annual Precipitation at Blue Hill Observatory

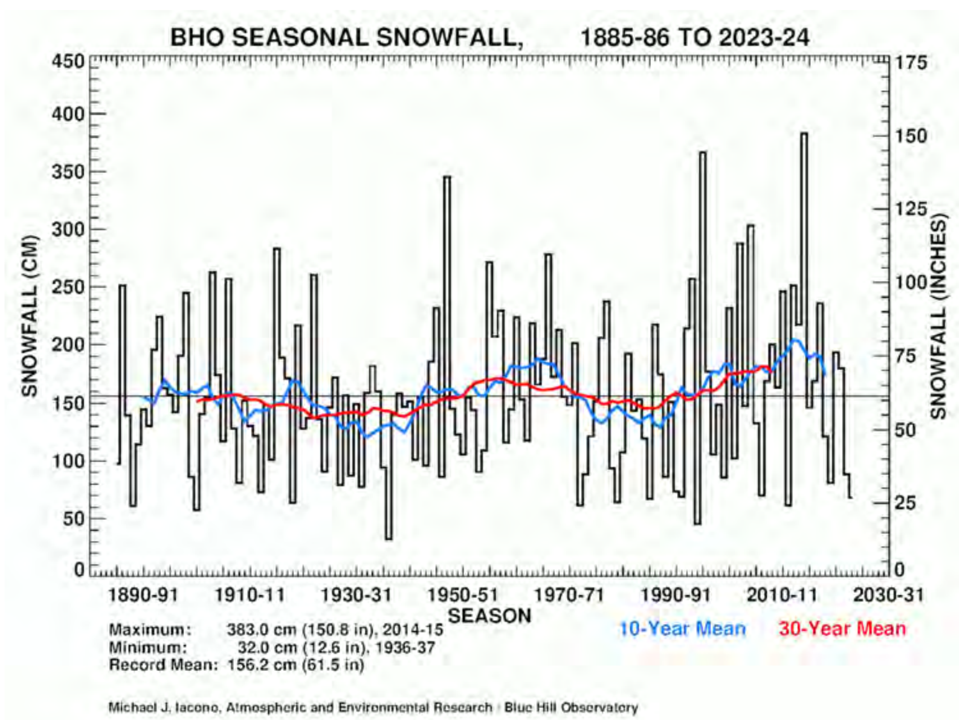

Figure 1.4.5 Annual Snowfall at Blue Hill Observatory

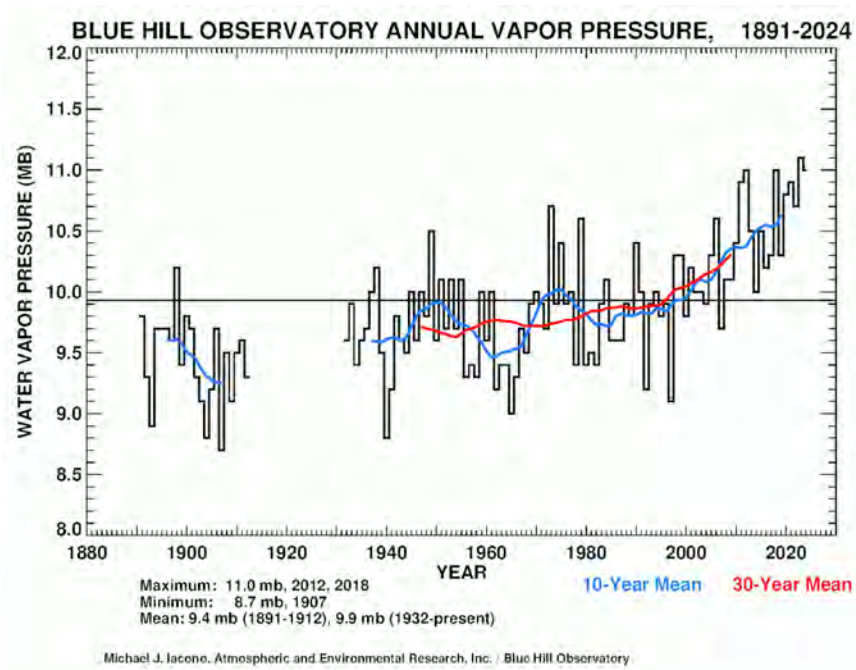

Figure 1.4.6 Annual Vapor pressure at Blue Hill Observatory

Figure 1.4.7 shows the Houghton Pond Freeze and Thaw Julian Calendar Day of the year near the Blue Hill Observatory. The length of time the pond stayed frozen has decreased by two weeks since 1886, an indicator of a positive warming trend. A similar natural indicator is when the first ripe blueberries are found at Blue Hill, as shown

by Figure 1.4.8. Blueberries have ripened about one week earlier since the 1880s.

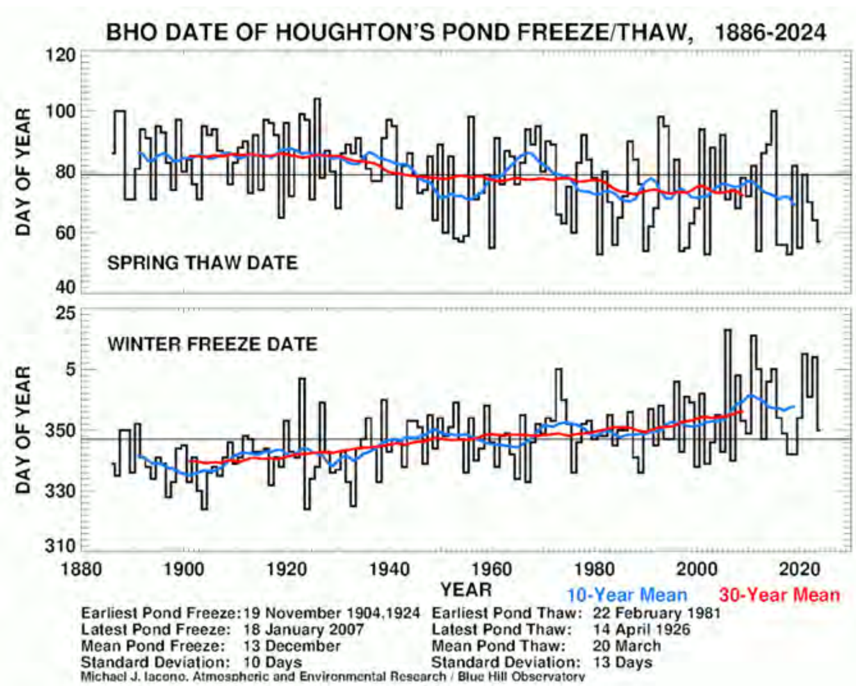

Figure 1.4.7 Blue Hills: Houghton Pond Freeze and Thaw Julian Calendar Day of the Year

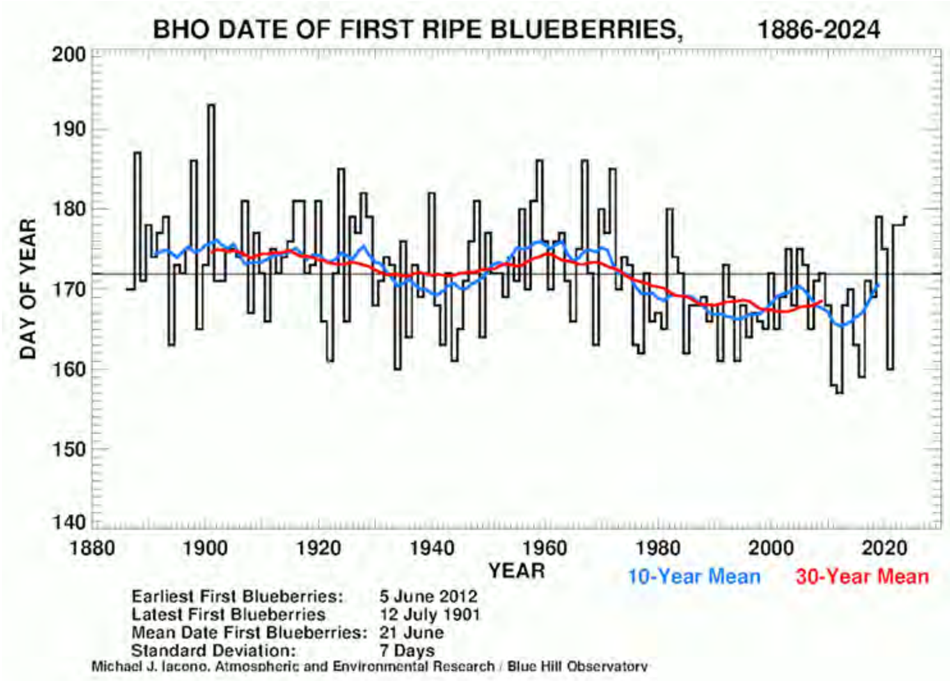

Figure 1.4.7 Blue Hills Julian Calendar Day First Day of Blueberries

1.5 Acknowledgements

HYSR would like to thank the extensive assistance given to this study by Mr. Paul Ferland, EIT, Administrator of Community Utilities for the City of Fall River. He was willing to support the study proposal, present it to the MVP grant administrator, and locate long buried reports, blueprints, and data and to assist with the system-level survey. Many thanks to Ms. Astrid Guerrero, Mr. Zach Henderson and Ms. Lindsey Wilcox, P.E., of Woodard & Curran for conducting GIS analysis as needed, creating the study bathymetry maps and for digitizing written historic water system data for the City of Fall River.

Thanks to Mr. David Piela, Director of Treatment at the Fall River water treatment facility and his colleagues for helping to find old drawing and blue prints and assisting with the system level survey and to Mr. Trevor Coelho, Project Manager. My thanks to Mr. Chris Wickman, Superintendent of Somerset Water and Sewer and Mr. Paul Sylvia, P.E. former Superintendent of Somerset Water and Sewer. Thanks also to Mr. Carl Destremps, Superintendent of the Stone Bridge Fire District and treatment plant operator Mike Normandin.

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MVP Program Coordinator Southeast Region, Executive Office of Energy and Environmental Affairs for helping the City of Fall River gain approval and ultimately funding for the study.

1.6 References

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2.1. An Introduction to Reservoir Firm Yield Analysis.

Historically, surface water supply system reliability was referred to as Safe Yield. It was determined using a Mass Curve analysis also known as the Rippl mass curve method, developed by W. Rippl in 1883. A graphical analysis technique, the Mass Curve method was used for many years. By the mid-1960s, time-share mainframe computers became available and a Mass Balance method was developed and used for very general safe yield assessments in southern New England, as described by Collins et al. (1969). A series of four storage-yield curves were developed and published by Collins et al. (1969). The method was developed to update storage yield curves prepared during dry periods of 1914 and 1940 using data collected during the Period-of-Record Drought of the mid-1960s. Figure 2.1.1 is the storage-yield curve that would have been used to estimate the 1960s drought safe yield for the water supply systems discussed in this report.

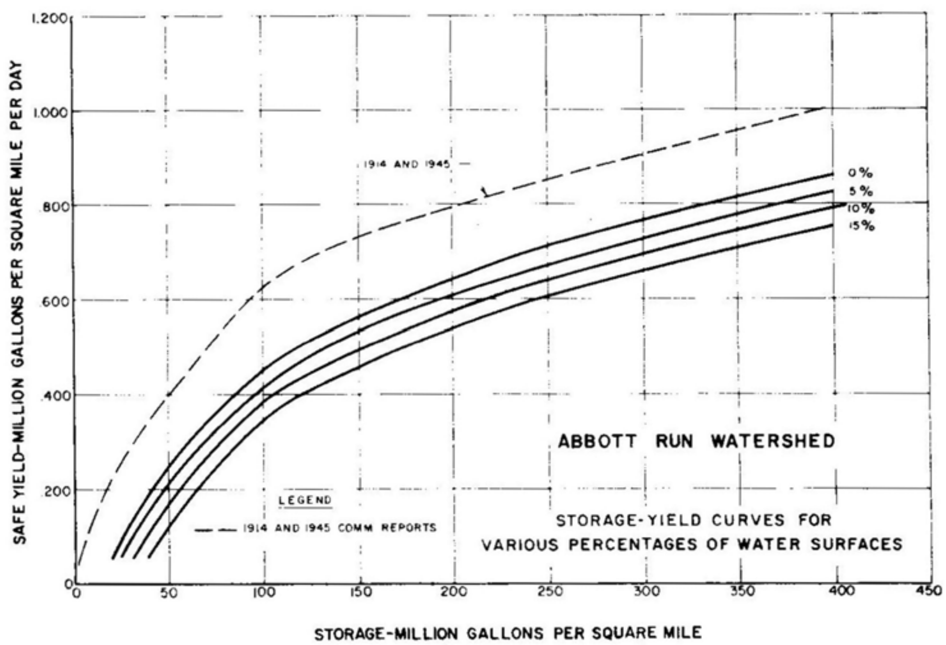

Figure 2.1.1 Storage Yield Curve for the Abbott Run Watershed, Valley Falls, RI

The clear advantage to using a storage-yield curve is its ease of use. The analyst only needed to know the size of the reservoir's watershed area, the total storage capacity and the surface area of the reservoir when full. The safe yield per square mile of watershed was read off the vertical axis of the curve. For example, a reservoir has a 3.0 mi^2 watershed and a total storage capacity of 1,050 million gallons. When full, its surface area is 288 acres = 0.45 square miles. The storage per square mile is then 1,050/3.0 mi^2 = 350 MG per square mile. The fraction of the watershed covered by the reservoir is 0.45 mi^2/3.0 mi^2 = 0.15. Starting at 350 MB/mi^2 on the horizontal axis of Figure 2.2.1, rise to intersect the 0.15 curve. From that curve, move horizontally to the Safe Yield vertical axis and read 0.700 million gallons per square mile. Given the watershed area of 3.0 mil^2, the safe yield is 0.700 MG/mi^2 x 3.0 mi^2 = 2.1 million gallons per day.

One of the down sides of the method was that the general practice was to use the entire volume of the reservoir whether or not all the water was accessible. Second, the surface evaporation was estimated using water level measurements in a copper pan floating in a water tank on Beacon Hill in Boston. D. Fitzgerald, chief engineer of the Boston Metropolitan District Commission, conducted these evaporation experiments. Ultimately, he was suspicious of the validity of his results due to the water in the copper pan being warmer than the water in the tank the pan floated in. His work is described in FitzGerald (1886, 1892). Lacking an alternative, water suppliers continued to use the FitzGerald evaporation pan estimates for safe yield analysis. It was for this reason that Fennessey (2000, 1996a, 1996b)

With the enactment of the Massachusetts Water Management Act in 1985, regulations were developed by the Massachusetts Department of Environmental Protection, henceforth MA DEP, or the Department. Those regulations require that a water supply system must apply for a water withdrawal permit if usage increased by 0.1 mgd over the amount that user was registered for. If the supply source was a surface-water system and a permit was required, the applicant was required to perform a Firm Yield analysis of their system. Fennessey (1996) devised the methodology used by MA DEP. Firm Yield is used now instead of Safe Yield because Safe Yield connotes a guarantee of sorts. Water systems specialists understand that there will always be a worse drought, and that even with systems operating at their system's Firm Yield rate, there will be supply failures.

2.2. Theoretical Motivation.

It is important to briefly discuss the theoretical underpinnings of a reservoir analysis. Because there is a physical basis for a mathematical model of a reservoir, the model presented herein is universally applicable. While this statement implies that the model is applicable everywhere, it is not necessarily solvable. The governing equation is the Conservation of Water Mass for an incompressible fluid. Mathematically, this model has no explicit solution because many of the independent variables in the equation have no analytical form and must stem from empirical time series data. Rather, the model is solved by approximation.

A mass flow diagram of a single reservoir is shown as Figure 2.2.1. The model reservoir equation for a single reservoir is derived using Figure 2-2-1 and is shown as Equation 2-2-1.

$$\frac{dS(t)}{dt} = A_r(t)\left[p(t) - e_p(t)\right] + A_w(t)\left[q_{si}(t)\right] + q_{gi}(t) - \left[q_{so}(t) + q_{go}(t) + q_w(t) + q_r(t)\right] \qquad (2.2.1)$$

where: S is the instantaneous volume of water in active storage; $A_r(t)$ is the area of the lake surface; $A_w(t)$ is the area of the contributing watershed; p(t) is the instantaneous rate of precipitation falling on $A_r(t)$; $q_{si}(t)$ is the instantaneous surface water inflow rate per unit area from $A_w(t)$; $q_{gi}(t)$ is the instantaneous groundwater inflow rate; $q_{so}(t)$ is the uncontrolled instantaneous spillway discharge rate; $q_{go}(t)$ is the instantaneous groundwater outflow rate (including seepage beneath the dam); $q_w(t)$ is the instantaneous withdrawal rate and $q_r(t)$ is the instantaneous controlled lake release, should one be required. None are required in the present study.

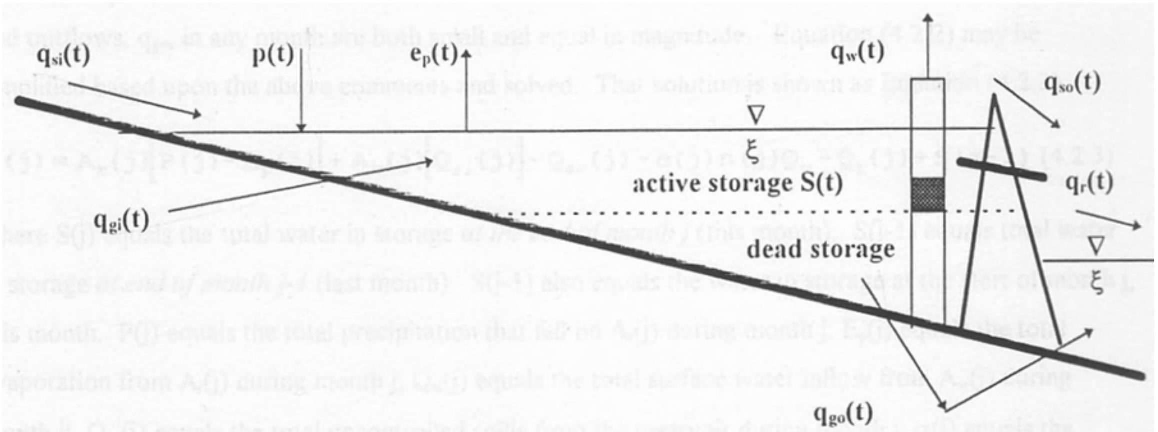

Figure 2.2.1 Water Supply Reservoir Mass Balance Diagram

The analyst is interested not in the time rate of change in the reservoir storage, but in the amount of water in the reservoir (the storage state) at any given point in time. To estimate the water in storage at some given time, call it t_2, Equation 2.2.1 must be integrated from some point in time, say t_1 when the storage $S(t_1)$ was known, to determine $S(t_2)$, the volume of water in storage at time t_2.

The total change in reservoir storage, $S(t_2) - S(t_1)$ between time $t=t_1$ and $t=t_2$ is determined by separating variables (multiply both sides of Eq. 2.2.1 by dt) and integrate over time interval t_2-t_1, as shown by Eq. 2.2.2:

$$\int_{S_1}^{S_2} dS(t) = \int_{T_1}^{T_2} A_r(t)[p(t) - e_p(t)] + A_w(t)[q_{si}(t)] + q_{gi}(t) - [q_{so}(t) + q_{go}(t) + q_w(t) + q_r(t)]dt \quad (2.2.2)$$

The solution to Eq. 2.2.2 is shown below as Eq. 2.2.3:

$$S(t_2) - S(t_1) = A_r(\Delta t)[P(\Delta t) - E_p(\Delta t)] + A_w(\Delta t)[Q_{si}(\Delta t)] + Q_{gi}(\Delta t) - [Q_{so}(\Delta t) + Q_{go}(\Delta t) + Q_w(\Delta t) + Q_r(\Delta t)] \quad (2.2.3)$$

where during time period $\Delta t= t_2-t_1$, $P(\Delta t)$ equals the total precipitation that fell on the reservoir surface; $E_p(\Delta t)$ equals the total evaporation from the reservoir surface; $Q_{si}(\Delta t)$ = equals the total streamflow per unit area that flowed into the reservoir; $Q_{gi}(\Delta t)$ equals the total groundwater inflow to the reservoir; $Q_{so}(\Delta t)$ equals the total amount of water unintentionally spilled from the reservoir into the stream channel below the impoundment; $Q_{go}(\Delta t)$ equals the total reservoir outflow as groundwater; $Q_w(\Delta t)$ equals the total withdrawal one reservoir user and $Q_r(\Delta t)$ represents the total amount of water intentionally released from the reservoir. For this study, a daily model time step ($\Delta T=t_2-t_1=1$ day) was used to route flows through the systems system to not only estimate that system's firm yield but to also generate time series of the Mass Balance Equation components if desired. The addition of a monthly use factor, α(mon) can be incorporated into Equation 2.2.3. By assuming that groundwater inflows are equal to groundwater outflows, those two terms are eliminated. In fact, there is no practical way to estimate either term on a daily basis.

2.3. The Practical Application of the Mass Balance Equation.

We can simplify Eq. 2-2-1 to Equation 2-3-1, knowing that Δt = 1 day, to solve for the volume of water in storage at the end of the day:

$$S(j+1) = S(j) + A_r(j)[P(j) - E_p(j)] + A_w(j)[Q_{si}(j)] + Q_T(j) - [Q_{so}(j) + \alpha(mon) + Q_w(j) + Q_{ow}(j) + Q_r(j)] \tag{2.3.1}$$

where S(j+1) is the volume of water in storage at the end of today; S(j) is the water that was in storage at the start of today and all the other terms are additions to or subtractions from S(j) during day j. The volume of water at the start of the next day, is S(j+1) and at the end of day j+1, the solution to Equation 2.3.1 will be S(j+1). Please note that in Equation 2.3.1, two new terms have been added. $Q_T(j)$ equals the total volume of water transferred to the reservoir during day j and $Q_{ow}(j)$ is the total volume of water some other user withdraws from the reservoir that day.

The firm yield model requires the analyst to solve Equation 2.3.1 for tens of thousands of days, which comprise the "period-of-record" used. Climate data, such as daily P(t) and $E_p(t)$ records need to be located or developed from some other location. Daily inflows, $Q_{si}(t)$ need to be estimated as well because there are no streamgages located near any of the reservoirs. The relationship between $A_r(t)$ and $A_w(t)$ is shown in Equation 2.3.2 and Figure 2.3.1 which are shown below.

$$A_{TOT} = A_r(j) + A_w(j) \tag{2.3.2}$$

where A_{TOT} is the total watershed area; $A_r(j)$ is the reservoir surface area at during day j and $A_w(j)$ is the area of the contributing watershed during day j. When the reservoir is completely empty, its surface area, $A_r(j)$ equals 0 and the contributing watershed area, $A_w(j)$ equals A_{TOT}

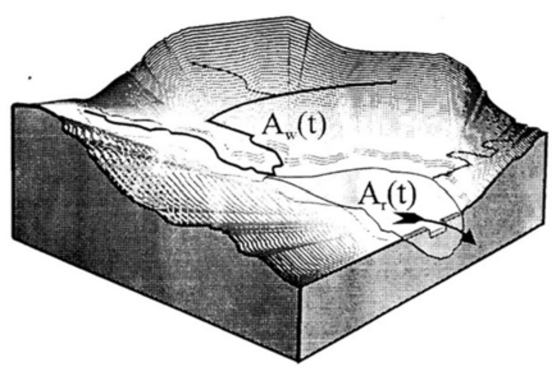

Figure 2.3.1. A Reservoir Sited in a Watershed

Because A_{TOT} is defined by the outlet/spillway of the reservoir, it remains constant but A_r and A_w dynamically change over time as the reservoir fills and empties according to the Mass Balance Equation. In Figure 2.3.1 above, A_{TOT} is the peripheral rim, including the spillway of the dam.

By using what is called the stage-storage-area curve relationship, to be discussed shortly, the reservoir volume, S and its surface area, A_r is known on day 1 because S(j=1) is specified as the reservoir initial volume condition, which is necessary to solve Equation 2.3.1. Because the elevation of the reservoir surface is the third part of the stage-storage-relationship, that can also be used as an initial condition. In other words, by specifying Z(j=1) on the very first day of the simulation, both S(1) and $A_r(1)$ are known. Equation 2.3.2 is rearranged to solve for $A_w(1)$, as shown below by Equation 2.3.3 and that will be known

$$A_w(j) = A_{TOT} - A_r(j) \tag{2.3.3}$$

The final term of Equation 2.3.1 is $Q_{so}(j)$, which is the volume of water spilled during day j. For an analysis of the firm yield sort, the reservoir is considered "full" when the surface reaches the invert elevation of the spillway. The assumption is that if the spillway, or some alternative form of outlet, has a discharge capacity that's less than or equal to the volume of water in S(j), then the volume above the spillway elevation is all spilled. If S_{max} is defined as the volume of water in storage when the reservoir is full, then the following condition, shown below as Equation 2.3.4, is assumed:

$$Q_{so}(j) = S(j+1)) - S_{MAX} \tag{2.3.4}$$

and S(j+1) is reset to S_{MAX}, otherwise there was no spill and $S_{so}(j) = 0$.

2.4. The Stage-Storage-Area Relationship

In the last section, mention was made of a relationship between the reservoir stage (elevation of the water surface), storage (volume of all the water, both active and dead storage) in the reservoir and area, which is the surface area of the reservoir when it is at that elevation. The relationship is established using a bathymetric map, which shows contours of equal depth estimated from bathymetric field measurements. Typically, the topmost contour is assumed to represent the surface of the reservoir at Normal Pool, i.e. when it's full. Typically, that elevation is tied to a survey benchmark elevation. Working from the top down, the elevation of the bottom of the reservoir is estimated.

Figure 2.4.1. is an enhanced bathymetry map of Big Alum Pond, located in Sturbridge, MA, which was developed using these techniques. On this particular map, the 0 contour denotes the Normal Pool. A spatial analysis of the processed bathymetry data results in a quantitative relationship between the depth of water in each lake and the surface area within each depth contour. The depth-area relationship for Big Alum Pond is shown in Table 2.4.1.

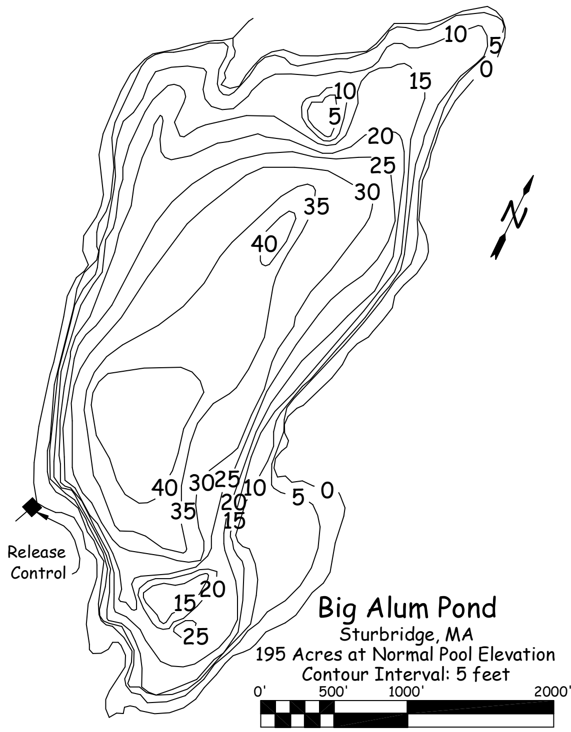

Figure 2.4.1. Big Alum Pond Bathymetric Map

Table 2.4.1.
Big Alum Pond Bathymetry Data

Depth (feet)	Area (acres)
0	195
5	177.3
10	158.6
15	134.5
20	115.0
25	85.0
30	64.9
35	34.9
40	12.4
45	0

The volume of water in storage, S, and the area of the lake surface, A_r, is part of the mass balance relationship but the lake depth is not. The bathymetry map depth-area relationship is used to develop the necessary lake volume-area characteristics. The cumulative volume of water in the lake, V, equals the sum of the active storage, S, and the uncontrollable dead storage, V_{dead}. V is described as a function of elevation from the bottom of the lake, as is shown below by Equation (2.4.1).

$$V(z) = \int_{Z_{min}}^{Z} A(z)dz \qquad (2.4.1)$$

where V(z) is the total volume of water in the lake when the lake surface is at elevation z relative to the bottom of the lake (zero feet) and A(z) is the surface area of the lake when the lake surface is at reference elevation z. Since the bathymetry data is not continuous, V(z) must be estimated numerically. One approach is to use the trapezoidal rule from integral calculus. V(z) is then estimated using Equation (2.4.2).

$$\begin{aligned} V(z) &= \int_{Z_{min}}^{Z} A(z)dz \\ &\cong \sum_{i=1}^{N-1} \left[\frac{A(z_i) + A(z_{i+1})}{2} \right] [z_{i+1} - z_i] \end{aligned} \qquad (2.4.2)$$

where z_i and z_{i+1+} are respectively the local elevation to the bottom and the top of the ith of “N” “slabs” of water. The thickness of each slab is the absolute difference between z_{i+1} and z_i.

Since the true maximum depth of each lake is not known, in this study, depending upon the area of the bathymetry map's shallowest contour, this contour may be assumed to equal the maximum depth of the lake, or the bottom is assumed to equal the depth of what would be the next contour. This assessment may be made on a case-by-case basis. For Big Alum Pond example, as indicated on Figure 2.4.1, the shallowest contour lies at a depth of 40 feet. In this case, because the 40-foot contour area is large, the deepest point of the lake is assumed to lie 45 feet below the elevation of the normal pool (the elevation of the invert/crest of the impoundment's spillway).

Using the data in Table 2.4.1., the volume Big Alum Pond's nine 5-foot-thick slabs of water is estimated. This relationship is demonstrated by modifying Table 2.4.1. to create Table 2.4.2. in conformance with Equation (2.4.2).

Table 2.4.2.
Big Alum Pond Depth-Area-Volume Relationship

Depth (feet)	Area (acres)	Slab Index i^{th} of N	Local Z_i and Z_{i+1} Elevation (feet)	Slab Volume (acre-feet)	V(z) (acre-feet)
0	195	9	40 to 45	930.8	4,400.5
5	177.3	8	35 to 40	839.8	3,469.7
10	158.6	7	30 to 35	732.9	2629.9
15	134.5	6	25-30	623.9	1,897.0
20	115.0	5	20-25	500.0	1,273.1
25	85.0	4	15-20	374.6	773.1
30	64.9	3	10-15	249.5	398.5
35	34.9	2	5-10	118.1	149.0
40	12.4	1	0-5	30.9	30.9
45	0				0

At this stage of the analysis, a unique three-dimensional estimate of the relationship between the lake's depth, z, its surface area, A(z), and its total storage volume, V(z) has been established. As previously described, the mass balance model requires the active storage volume, S, rather than the total volume, V. The final form of the stage-storage-area table is shown in Table 2.4.3. To convert the storage from acre-ft to million gallons, multiple the acre-ft value by 0.30585. For the present study, HYSR used stage elevation with units of feet (msl), surface area with units of acres and storage with units of acre-ft.

The stage-storage-area curve for Big Alum Pond is shown as Figure 2.4.2. This illustrates the relationship between the active storage and dead storage, the Normal Pool elevation, the invert elevation of a vertical slide gate and the spillway elevation. Using these three relationships in the Firm Yield analysis requires estimating each variable at elevations other than every five feet. The analyst may choose to fit a smooth curve to each or use simple linear interpolation. None of relationships are truly exact, in particular the storage volume, since the approach assumes that the bottom of the reservoir is impermeable and that bank storage is negligible.

Therefore, any error during to using linear interpolation compared with a smooth function is not only small but acceptable. Fortunately, underestimating the actual storage means that the estimated firm yield is actually somewhat greater. The consequence is a conservative estimate with an extra margin of safety.

Two final conditions are imposed that relate to the above V_{DEAD} and S_{MAX}. For a reservoir that can be completely emptied, V_{DEAD} will equal zero and this will occur at elevation Z=0. In the case of a raw water intake, for example, for water quality consideration, V_{DEAD} will be greater than zero and Z_{DEAD}, preferably referred to herein as Z_{MIN}, will also be greater than 0 ft at the local elevation. When modeling Firm Yield in this study, HYSR specified Z_{MIN} and Z_{MAX} as boundary conditions, where Z_{MAX} is the elevation of the reservoir outlet structure's invert or spillway elevation. By specifying a lower Z_{MIN} representing, for example, a deeper intake, the active storage is increased and the Firm Yield is increased.

With regard to maximizing the Firm Yield, estimating the greatest sustainable average daily withdrawal rate during a sustained drought represents the optimal solution for a single purpose water supply reservoir. To approach an optimal solution, all the active storage is entirely depleted for one day, as mentioned above. In the case of a multi-reservoir system, the optimal solution is to simultaneously fully deplete all the active storage in each the reservoir on the same day. In the present study, HYSR estimated the system Firm Yield by ensuring that the model used up all the active storage in each reservoir but not on a single day. To do so would require a model that was beyond the scope of the present study.

Finally, it should be mentioned that each of the reservoirs in the present study are "single purpose" in that none of them are used for hydropower generation, flood control, irrigation, etc. However, South Watuppa Pond is used for recreation and HYSR assumes that it will likely continue to be used that way even after Fall River begins to further develop it as a water supply source. Before that happens, the City will likely enter into negotiations with the South Watuppa Pond community to determine the limit to active-draw-down during an extended dry period and rules to exceed those limits. The HYSR model has been developed to assist with those negotiations or to examine alternatives in a future study.

Table 2.4.3
Big Alum Pond Stage-Storage-Area

Stage (feet)	Stage (feet msl)	Storage (acre-feet)	Storage ($x10^6$ gallons)	Surface Area (acre)
0	677.3	0	0	0
5	682.3	31.0	9.5	12.4
10	687.3	149.3	45.7	34.9
15	692.3	398.8	122.0	64.9
20	697.3	773.6	236.6	85.0
25	702.3	1273.6	389.5	115.0
30	707.3	1897.3	580.3	134.5
35	712.3	2630.1	804.4	158.6
40	717.3	3469.8	1061.2	177.3
45	722.3	4400.6	1345.9	195.0

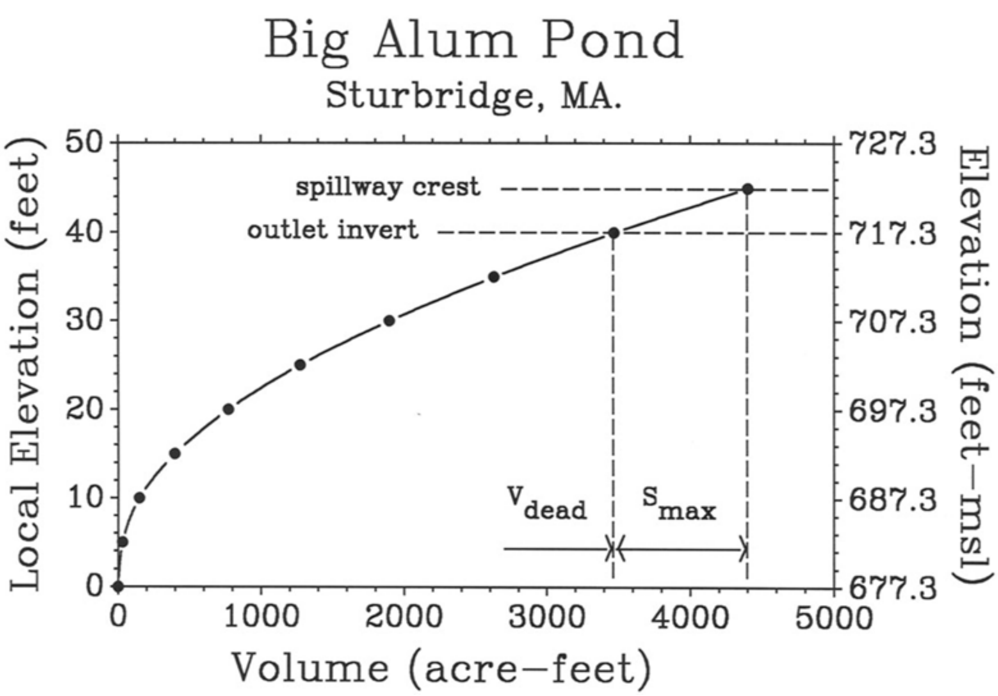

Figure 2.4.2 Big Alum Pond Stage-Storage Curve

2.5. The Firm Yield Analysis.

In the previous section, the application of the Mass Balance is explained but what was not explained was how to use it to determine the Firm Yield of a reservoir system. First, it's necessary to define the Firm Yield. It is the largest continuous withdrawal, Q_w that can be sustained during an extended drought during which the "active storage" is fully depleted only once. In other words, the reservoir during a single day of the simulation the only water left at the end of the day, S(j+1) is S_{DEAD}, the "dead storage", as was shown in Figure 2-2-1. The surface elevation is high enough to flow into an intake tower or without cavitating raw water pumps, etc. The process involves either repeating or "looping" the decades of daily Mass Balance analysis while slowly increasing Q_w until $S(j+1) = S_{DEAD}$. Alternatively, the modeler can specify a value of Q_w to see how close $S(j+1) = S_{DEAD}$ and repeat the process one iteration at a time, slightly varying Q_w until that condition is met.

The optimum solution to the Firm Yield problem is to fully deplete the active storage for that single day. In the case of a multi-source system, such as Fall River's, the optimum solution would be to empty the active storage of all three sources simultaneously on the same day. In practice, this would require constant monitoring and withdrawing water from each source at variable or nearly variable rates. The HYSR Firm Yield estimates are made by maximizing the daily water withdrawal from each source, however, the three reservoirs fully deplete their active storage on different days and in some cases, even different years. Constructing a model to achieve the optimum Firm Yield, is beyond the present study's Scope of Work. However, in the case of the Fall River reservoirs, for example, if they fully deplete their individual active storages on nearly the same day, the optimum solution has been nearly achieved for that particular combination of withdrawal rates and each reservoir's S_{MIN} reservoir boundary condition.

2.6 References

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3.1 Introduction to Climate Data Needs

As discussed in the last chapter, the Firm Yield Mass Balance Model requires daily precipitation data and daily evaporation data. Ideally, precipitation and evaporation records for each reservoir would be available, but unfortunately, that is rarely the case. When it comes to actual lake evaporation, this data is virtually never available. However, daily (24 hour) precipitation, maximum and minimum observed air temperatures and snowfall are typically recorded by a Town or City's public works department. To overcome the lack of daily evaporation data, HYSR uses a model developed by Fennessey and Vogel (1996) that approximates the monthly average daily free surface evaporation determined by an energy budget approach. Fennessey (1995) determined that there was no significant difference between Firm Yield estimates made by repeating 12-monthly means values of E_p year after year versus the difficult-to-construct monthly E_p time series. Those twelve-monthly values are then converted to estimates of the average 365 or 366-day daily evaporation rate. The 365 or 366 daily evaporation cycle is repeated year after year, as will be discussed in a subsequent section.

3.2 Developing Climate Data Time Series

Compiling a long-term climate record for Fall River proved to be a significant challenge. While the city began keeping records in 1894, over the years, that record keeping became sporadic. Figure 3.2.1 shows the first entries of the NOAA Fall River Summary of the Day dataset, which was collected and maintained by the City. Unfortunately, entire months and occasionally entire years of data were never recorded or those records were lost before providing them to NOAA.

	A	B	C	D	E	F	G	H	I	J
1	STATION	NAME	LATITUDE	LONGITUD	ELEVATION	DATE	PRCP	SNOW	TMAX	TMIN
2	USC001926	FALL RIVER	41.71667	-71.1333	57.9	1894-01-01	0	0	32	20
3	USC001926	FALL RIVER	41.71667	-71.1333	57.9	1894-01-02	0	0	38	18
4	USC001926	FALL RIVER	41.71667	-71.1333	57.9	1894-01-03	0	0	42	32
5	USC001926	FALL RIVER	41.71667	-71.1333	57.9	1894-01-04	0	0	46	37
6	USC001926	FALL RIVER	41.71667	-71.1333	57.9	1894-01-05	0.02	0	48	37
7	USC001926	FALL RIVER	41.71667	-71.1333	57.9	1894-01-06	0.02	0	40	34
8	USC001926	FALL RIVER	41.71667	-71.1333	57.9	1894-01-07	0	0	40	33
9	USC001926	FALL RIVER	41.71667	-71.1333	57.9	1894-01-08	0	0	29	23
10	USC001926	FALL RIVER	41.71667	-71.1333	57.9	1894-01-09	0	0	28	24
11	USC001926	FALL RIVER	41.71667	-71.1333	57.9	1894-01-10	0.03	0.5	32	20
12	USC001926	FALL RIVER	41.71667	-71.1333	57.9	1894-01-11	0.11	0	37	20
13	USC001926	FALL RIVER	41.71667	-71.1333	57.9	1894-01-12	0	0	25	10
14	USC001926	FALL RIVER	41.71667	-71.1333	57.9	1894-01-13	0	0	25	5
15	USC001926	FALL RIVER	41.71667	-71.1333	57.9	1894-01-14	0	0	35	25
16	USC001926	FALL RIVER	41.71667	-71.1333	57.9	1894-01-15	0	0	43	30
17	USC001926	FALL RIVER	41.71667	-71.1333	57.9	1894-01-16	0.3	0	44	35

Figure 3.2.1 NOAA Fall River Weather Data

Fortunately, the City of Fall River located a dot-matrix computer print-out of occasional daily records from 1947-1983. The print-out was scanned page-by page, and each page was converted to a single EXCEL file, all of which were provided to HYSR. HYSR spent several months preparing daily precipitation, maximum air temp, minimum air temperature and snowfall to create the continuous time series used for the present study. To that end, HYSR was able to piece together a continuous record of daily precipitation, daily temperature and daily snowfall for the period of record of 1940-1990.

In prior studies, HYSR has employed an inverse-distance squared surface interpolation approach to provide site-specific estimates. This was considered; however, the nearest NOAA Summary-of-the-Day observatories in New Bedford and Taunton are nearly as discontinuous as Fall River. NOAA's last record for Fall River was in 1978. Because of the T.F. Green airport, that NOAA station is a First Order Observatory with an excellent and continuous record which began in 1947 and continues to this day, HYSR considered using the Providence records directly but felt the effort to create a Fall River record was worthwhile. For these reasons and given that the five surface water supply sources are rather close together, the same climate is used for all sites. The difference in elevation among them is not sufficient to employ an adiabatic temperature correction.

As mentioned, the HYSR Firm Yield model doesn't use air temperature or snowfall, only precipitation which falls on the reservoir surface. The daily average temperature is never actually recorded, rather the maximum daily and the minimum daily temperatures are recorded and have been since the late 19th century, as shown in Table 3.2.1. It is accepted practice to add these two together and divide by 2 to provide an estimate of the average daily temperature. That average daily temperature data is used to create monthly average daily air temperatures, which in turn are used to create the estimates of monthly average day free-surface evaporation.

Preliminary study of each of the five reservoirs was conducted using a monthly time-step Mass Balance simulation model. Through historic records from the Blue Hills Observatory, significant dry periods occurred during the latter 1910s and the 1940s, the early 1950s as well as the drought of the 1960s. Because North Watuppa and South Watuppa Ponds and Stafford Pond have a significant volume of water per square mile of watershed, starting a simulation with them at S(day 1)= S_{MAX}, i.e. full during an historic dry spell, would be hard to justify. With this in mind, the climate record was constructed for the period-of1940-1990. The daily time-step HYSR Firm Yield model begins in 1940 and ends in 1980 because all the test simulations showed the reservoirs having recovered from the drought of the 1960s, having refilled by the early 1970s.

Although the HYSR daily Firm Yield simulation model runs for 1940-1980, the period of 1950-1980 was chosen as the base climate. With regard to "climate change" analysis, which will be discussed in a later chapter, the "mid-century climate" period of 2025-2055 and the "late-century climate" period of 2070-2100 are used to assess the potential impact to systems' firm yield under these two sets of climate data. The Firm Yield simulation runs through the same period of record of 1940-1980 but the climate change firm yield uses the 2025-55 and 2070-00 climate to rescale the daily precipitation on and evaporation from the reservoir surface and estimated daily inflows to each reservoir. More on this later.

Figures 3.2.2 and 3.2.3 respectively show the Fall River mean monthly average day air temperature and the mean (average) monthly average total precipitation. Although it's not used by the Firm Yield Model, Figure 3.2.4 shows the mean monthly total snowfall.

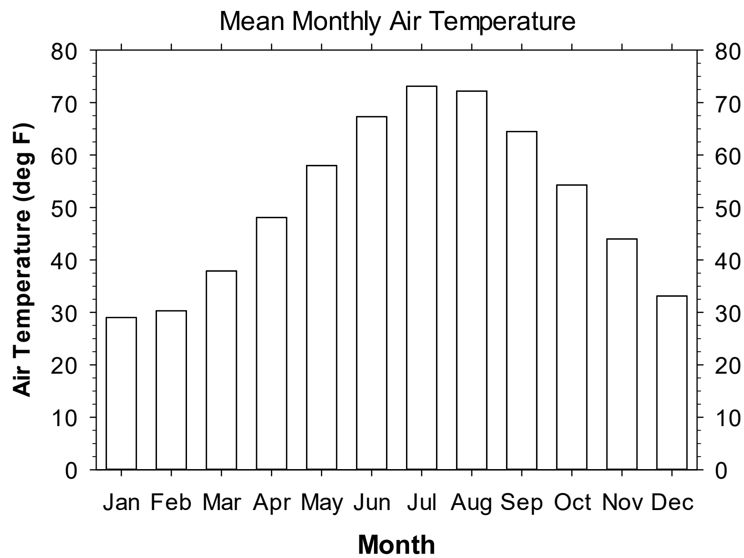

Figure 3.2.2 Monthly Mean Daily Air Temperature

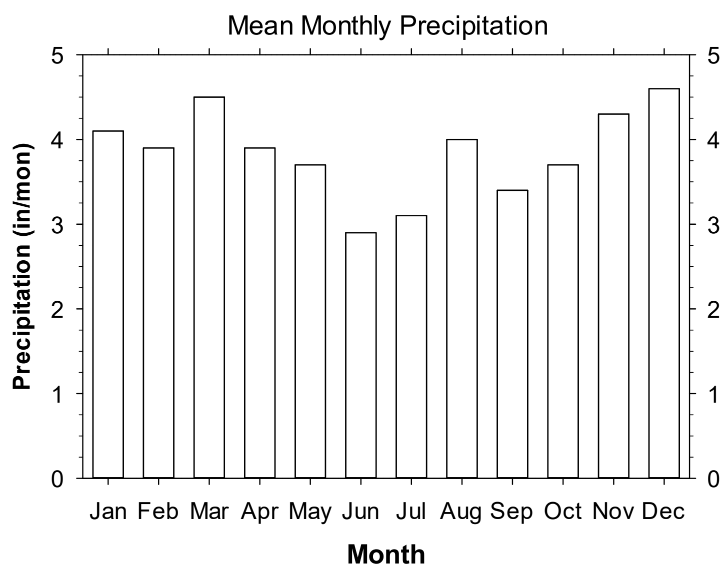

Figure 3.2.3 Mean Monthly Total Precipitation

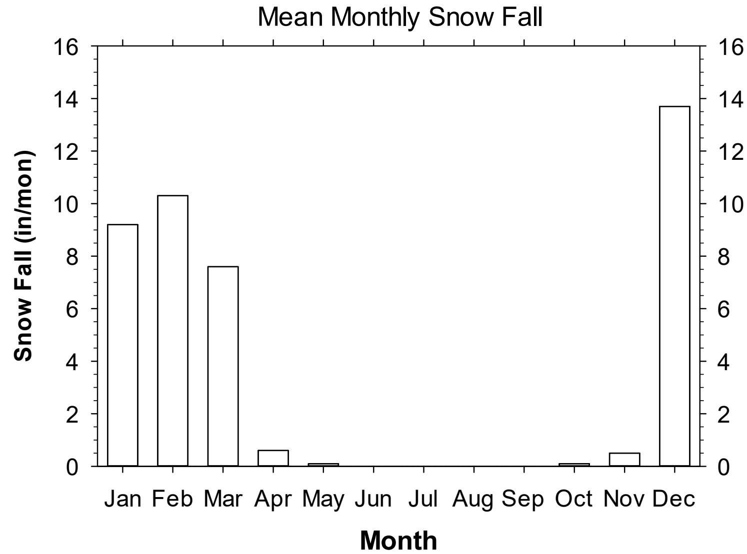

Figure 3.2.4 Mean Monthly Total Snowfall

3.3 Developing the Daily Reservoir Evaporation Time Series

As mentioned above, the first step in developing the daily evaporation time series is to develop monthly average daily values of evaporation. HYSR uses the method described by Fennessey and Vogel (1996) in a Journal of Hydrology article. Figure 3.3.1 shows the monthly mean daily reservoir evaporation rate.

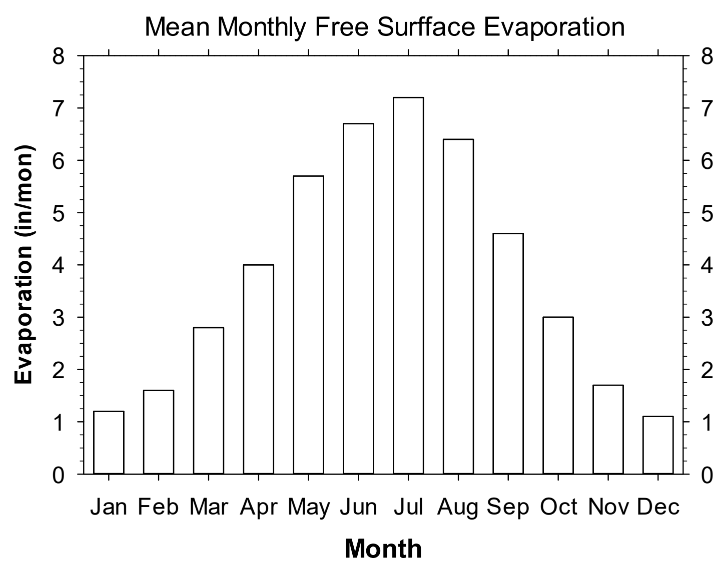

Figure 3.3.1 Monthly Mean Daily Reservoir Evaporation

The Firm Yield model requires estimates of daily pond and reservoir evaporation rates. HYSR developed these by using statistical OLS multivariate regression analysis somewhat similar to the approach used to estimate the mean monthly evaporation rates as described by Fennessey (2000) and Fennessey and Vogel

(1996). A 2-harmonic Fourier function's five coefficients are statistically estimated by assuming that the average monthly E_p rate occurs on the 15th day of each month.

The 2-harmonic Fourier function, E_p(*jday*), is fitted to *jday*=1,365 days to the 12 monthly estimates of free surface evaporation from Table 3.3.1 below, where *jday* is the Julian calendar day of the year. For example, Jan. 1 = *jday* 1 and Dec. 31 = *jday* 365. Specifically, the function is fitted to the twelve monthly E_p values by assuming that the mean monthly value occurs on the 15th day of each month. For example, January 15 is *jday* 15 and December 15 is *jday* 350.

$$\begin{aligned} E_p(\text{jday}) = E_{pa} &+ a_1\cos\left[\frac{2\pi\ \text{jday}}{365}\right] + b_1\sin\left[\frac{2\pi\ \text{jday}}{365}\right] \\ &+ a_2\cos\left[\frac{4\pi\ \text{jday}}{365}\right] + b_2\sin\left[\frac{4\pi\ \text{jday}}{365}\right] \end{aligned} \tag{3.3.1}$$

With E_{pa} (annual mean daily E_p) known, equations for the other four Fourier coefficients, a_1, a_2, b_1 and b_2 are developed using OLS statistical regression. The annual daily E_p (inches/day) cycle is repeated each year during the entire Period-of-Record (POR). One year's cycle is shown below as Figure 3.3.2. The hash marks on the horizontal axis are on or about the 15th day of each month.

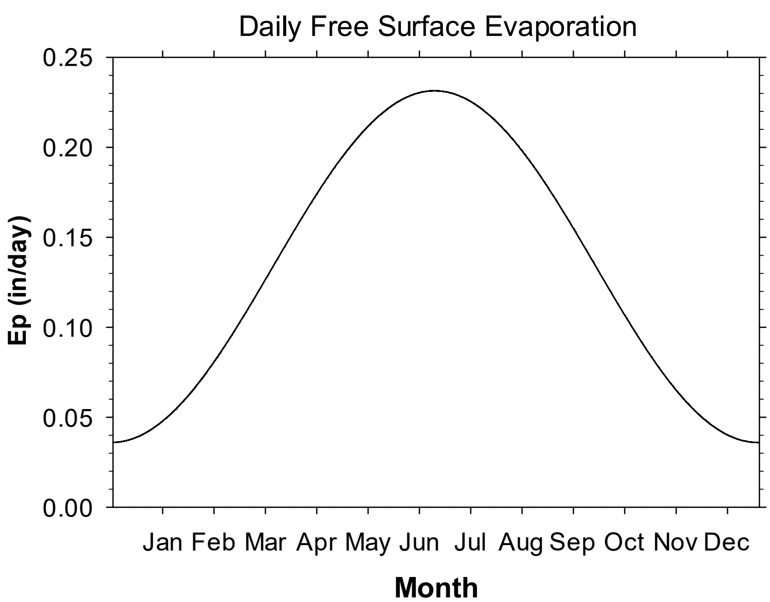

Figure 3.3.2 One Year Cycle of Mean Daily Reservoir Evaporation

The 1950-80 Base Period climate monthly values are shown below in Table 3.3.1 for reference. The annual totals/mean daily values are shown in Table 3.3.2.

Table 3.3.1
1950-80 Mean Monthly Observed Base Climate

Month	Temperature Mean (^oF)	Precipitation Mean (in/mo)	Evaporation Mean (in/mo)
January	29.0	4.1	1.0
February	30.3	3.9	1.4
March	37.9	4.5	2.3
April	48.1	3.9	3.4
May	58.0	3.7	4.7
June	67.3	2.9	5.6
July	73.1	3.1	5.9
August	72.2	4.0	5.2
September	64.5	3.4	3.9
October	54.3	3.7	2.5
November	44.0	4.3	1.4
December	33.1	4.6	0.9

Table 3.3.2
1950-80 Mean Annual Observed Base Period Climate

Temperature Mean (^{o} F)	Precipitation Mean (in/yr)	Evaporation Mean (in/yr)	Snowfall Mean (in/yr)
51.1	46.1	45.7	42.3

3.4 References

Fennessey, N.M., *Estimating Average Monthly Lake Evaporation in the Northeast United States,* J. of American Water Resources Association, Vol. 36, pp. 759-769, 2000.

Fennessey, N.M. and R.M. Vogel, *Regional Models of Potential Evaporation and Reference Evapotranspiration in the Northeast U.S.A.*, J. of Hydrology, Vol. 184, pp. 337-354, 1996.

Fennessey, N.M., *The Sensitivity of Reservoir Yield in Massachusetts to Model Time Step and Surface Moisture Fluxes*, ASCE J. of Water Resources Planning and Management, Vol. 121, No. 4, pp. 310-317, 1995.

4.1 Introduction to Estimating Daily Streamflow

As described in Chapter 2, decades of estimates of daily streamflows are required by the Firm Yield Mass Balance model. Daily reservoir inflows were denoted as $Q_{si}(t)$. Because no streamgages are located on the tributaries of the reservoirs, it is necessary to generate estimates of daily flows. For the present study, HYSR used the QPPQ Transform method which was developed by Fennessey (1994). Over the past 15+ years, the US Geological Survey (USGS) has extensively tested the QPPQ Transform. Farmer et al. (2014) determined that the QPPQ Transform method outperformed 21 alternative methods for estimating daily flows at ungaged sites for 185 watersheds located in the southeast US. The most recent QOPOQ Transform testing has been done by Russell et al. (2010) using a nationwide network of watersheds gaged by the USGS. The USGS and various agencies in at least eleven states around the US and South Africa have adopted the method, for example, Lorenz and Ziegeweid (2016). Recently, the QPPQ Transform has undergone extensive testing with the New Hampshire Department of Environmental Service (NHDES) river basin planning process (see Fennessey, 2018, 2023). The method has been used to generate daily streamflow estimates at ungaged sites and to extend the record of daily USGS streamgage data for over 30 years. Both applications of the QPPQ Transform are used in the present study.

4.2 The QPPQ Transform Method

HYSR's QPPQ Transform was specifically developed to provide an analysis with a long record of estimated daily streamflow at the ungaged site. While the implementation of the method is complicated, conceptually it is less so. The QPPQ Transform process is summarized by the following four steps and illustrated by Figure 4.2.1:

1. The upper left quadrant: *Q*. The analyst picks a suitable USGS "index" stream gage site with a long period-of-record (POR) of observed daily flows, $Q_I(t)$.
2. The upper right quadrant: *P*. The analyst estimates the probability of occurrence for each observed daily flow and uses $Q_I(t)$ to construct an "observed" period-of-record (POR) Flow Duration Curve (FDC), $Q_I(p)$.
3. The lower right quadrant: *P*. Using soil, climate, and topographic characteristics of the ungaged watershed, the analyst uses a regional FDC model to construct a "model" FDC, $Q_O(p)$, at the ungaged site.
4. The lower left quadrant: *Q*. Knowing the probability of each daily flow during the long sequence at the gaged site, and assuming those flows occur with equal probability at the ungaged site, the analyst generates an equally long sequence of daily flows at the ungaged site, $Q_O(t)$.

While Figure 4.2.1 suggests that the 2nd and 3rd quadrant FDCs are parallel, in the actual application they are not. Because the shape of each FDC depends on the independent variables mentioned in paragraph 3. above, they will always be different. What they have in common is that their respective daily flows occurred with equal probability on the same day.

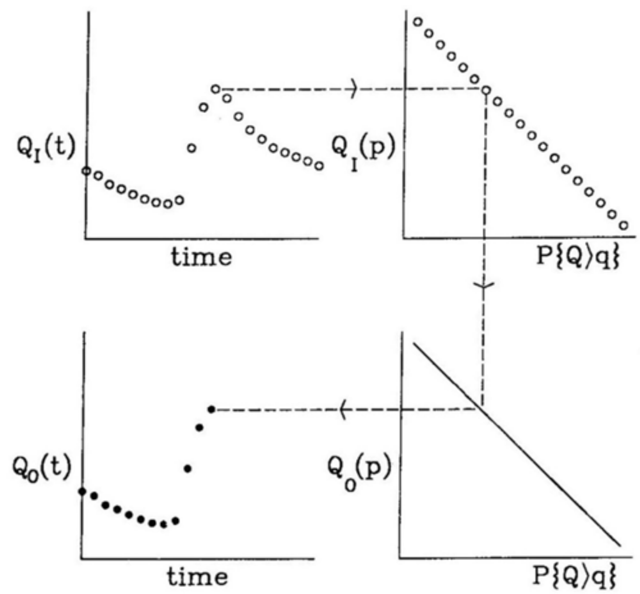

Figure 4.2.1. The QPPQ Transform Method

As discussed above, the third step of the QPPQ Transform method requires the analyst to construct a streamflow duration curve for the ungaged site using a regional FDC model. An FDC represents the relationship between the magnitude and the likelihood or probability of the occurrence of daily streamflow at a particular location in a river basin. It provides an estimate of the percentage of time a given streamflow was equaled or exceeded over a historical period of record. As a result, an FDC provides a simple yet comprehensive graphical view of the historical variability associated with streamflow at a site.

Figure 4.2.2 shows the hydrograph of a river, with flows that rise and fall with the seasons, overlain with the FDC constructed from those daily observations. The top horizontal axis is time and the bottom horizontal axis is exceedance probability, p. Each value of flow, Q, has a corresponding p. An FDC is simply a plot of Q_p, the p^{th} quantile or percentile of daily streamflow, versus p. As described by Fennessey and Vogel (1990), Vogel and Fennessey (1994, 1995), At very high flows, p nearly equals 0, and at very low flows, p nearly equals 1. For example, Q_{10}, a high flow quantile shown as the blue diamond, occurs with p=0.1, and Q_{90}, a low flow quantile, shown as the red diamond, occurs with p=0.9. Q_{50} is the median daily flow quantile, shown as the magenta diamond, and occurs with p=0.5.

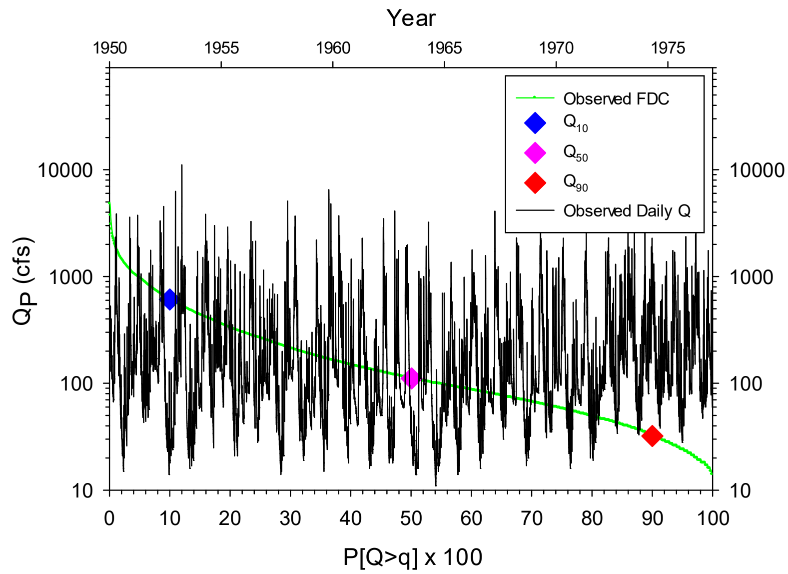

Fig. 4.2.1. Comparison between a River Hydrograph and its Flow Duration Curve

4.3 Estimating Quantiles from Stream Gage Data

To construct an FDC from a historic record of stream gage data, let *n* equal the number of daily observations from that gage. Let the ith member of the record be described as q_i where i=1,*n*. If the streamflow data are rank-ordered, i.e. sorted in descending order, then the result is the set of order statistics, $q_{(i)}$, where i = 1,*n* . Here $q_{(1)}$ equals the largest observed value among all q_i, and $q_{(n)}$ equals the smallest observed value among all q_i. In other words, the daily data belonging to a specific population is arranged in descending order from largest to smallest. Each quantile is estimated using a weighted estimator, shown below as Equation 4.3.1.

$$Q_p = (1-\theta)q_{(i)} + \theta q_{(i+1)} \tag{4.3.1}$$

where i=[(n+1)*p*] and θ = [(n+1)*p*-i] and *p* equals the exceedance probability, *p*=P[Q≥q]. The quantile estimator Q_p is undefined for values of *p* that lead to i=[(n+1)*p*]= 0. This approach was used to construct the Step 2 “observed” FDC.

4.4 The Regional FDC Model

As discussed earlier, a streamflow duration curve (FDC) is typically constructed from observed daily USGS streamflow data. To recap, the FDC provides the analyst with an estimate of how often (the probability) a specific streamflow rate is equaled or exceeded. Conversely, given a specific probability, 0<p<1, the analyst can estimate the corresponding streamflow rate. Because of the need to estimate these probabilities at ungaged sites HYSR developed what is now known as a regional FDC model.

The method is referred to as a *regional model* because the streamflow data and the variables used to construct the probability function parameter equations come from a specific region; therefore, the model may only be used in that region. As discussed by Fennessey (1994, 2018), the HYSR QPPQ Transform model is applicable to estimating all flows at ungaged sites located in the entire northeast U.S., including the New England states, New York, New Jersey and Pennsylvania.

The basis for the regional FDC model is the three-parameter Generalized Pareto (GPA). The GPA was determined to be the best probability function among many that were tested. The GPA quantile function, Q_p, is shown below as Eq. 4.4.1.

$$Q_p = \xi + \frac{\alpha}{\kappa}\left[1 - p^k\right] \tag{4.4.1}$$

where ξ (lower bound), α (scale), and κ (shape) are the three probability function parameters and p is the exceedance probability. Using multivariate statistical regression, Fennessey (1994) developed a regional equation for each of these three parameters that allows one to construct a POR daily FDC at an ungaged site using Eq. 3-2.

The advantage of the continuous regional FDC model approach used by HYSR is two-fold. First, HYSR's GPA probability function is monotonic. Second, interpolation between quantile estimates and extrapolation of the extremes is not necessary when using HYSR's continuous GPA model because the method estimates flows between the model's lower bound, ξ, which may or may not equal 0, and infinity.

As mentioned, the HYSR model is a regional model. This is so because HYSR developed separate multivariate equations for each of the three model parameters: ξ, the lower bound parameter; α, the scale parameter and κ, the shape parameter. The independent variables employed to describe each parameter as the dependent variable include the following, as listed in Table 4.3.1. All variables may be estimated using the USGS StreamStats GIS system, which was also used to delineate the watersheds analyzed in the present study.

Table 4.3.1.

QPPQ Transform GPA Model Variables

Watershed Area, AREA (mi^2)
Area of Lakes, Ponds & Reservoirs (%)
Area of Impervious Surface (%)
Area of USDA NRC HSG A (%)
Area of USDA NRC HSG C (%)
Area of USDA NRC HSG D (%)
Mean Annual Precipitation, PREC (inches)
Mean Annual Temperature, TEMP (^{o}F)
Mean Watershed Elevation, ELEV (feet MSL)
Main Stream Channel Slope, C-SLOPE (ft/mile)
Watershed Aspect relative to true north (degrees)

4.5 The USGS Index Streamgages

Step 1 of the QPPQ Transform requires the choosing and analysis of daily streamflow data from a USGS streamgage. For many years, HYSR relied solely on a special network of streamgages assembled by the USGS referred to as the HydroClimatological Data Network of HCDN prepared and released by Slack and Landwehr (1992). The period-of-record of any gage in the HCDN had to be at least 25 years long and located on watersheds that were free of significant anthropogenic impacts. The network is special because it was assembled expressly to investigate for evidence of climate change in “natural flow” streamflow records. Each gage in the network had associated with it an extensive list of climate, topographic and soil variables. More recently, Falcone (2011) released the large nationwide GAGES II database of gaged watersheds with many variables estimated for each one. Falcone et al (2010) describe its development. The number of GAGES II variables was far more extensive and comprehensive than the HCDN variable list thanks in large part to the advancement of Geographic Information System technology and development of many very diverse data layers during the period between the 2010 and 1988 development dates. By comparison, the HCDN variable list was developed from historic analysis of USGS topographic maps, USDA Soils maps, National Weather Service maps, all by hand. Any GIS work done at prior to its 1988 publication date was still computer main-frame based.

One of the HCDN streamgages, Adamsville Brook (USGS 011060000), was located in Adamsville, RI. Its 8.01 mi^2 watershed was gaged from 10-1-1940 – 9-30-1978. One of the systems in the present study, Somerset Reservoir, diverts water from the Segreganset River near Dighton, MA and pumps it to the Somerset Reservoir.

At the time the Somerset Reservoir pumping station was constructed on the Segreganset River in 1966, the USGS sited a streamgage station (USGS 01109070) about a mile upstream in Dighton. The watershed area at the gage is 10.6 square miles and its period-of-record is 10-1-1966 – the present day. The Town of Somerset monitors this streamgage to govern its transfer withdrawals depending on the season and the flow rate in the river. Both streamgage sites are shown below in Figure 4.5.1

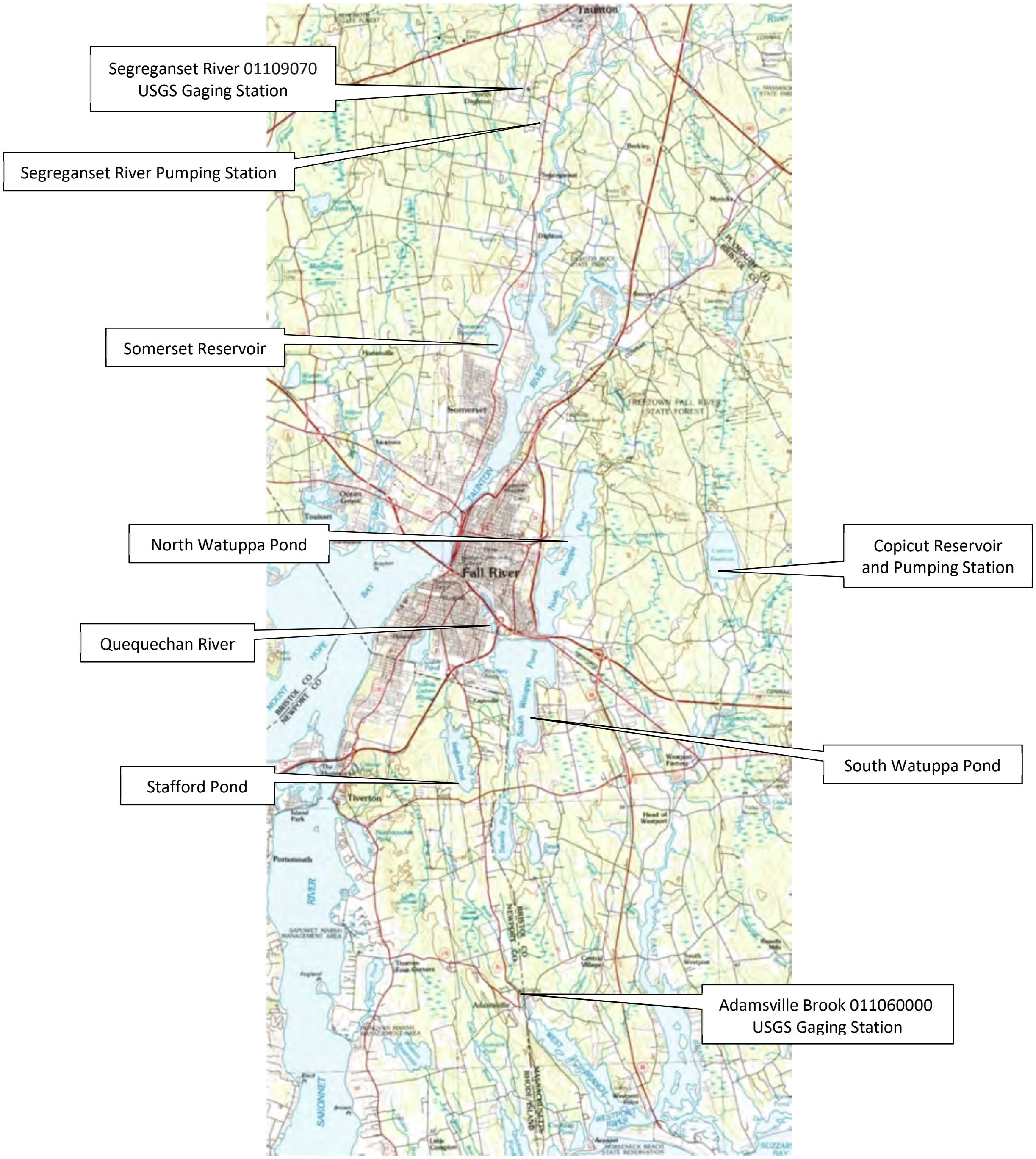

Fig. 4.5.1. Surface Water Supply Sources, Pumping Stations and USGS Streamgages

According to the GAGES II network key (Falcone, 2011), the Human Impact Index provides the analyst a quantitative way to assess anthropogenic impact to the watershed. The Adamsville Brook watershed has an Impact Index of 9 and the Segreganset River watershed had an Impact Index of 10. With both indices so close to one another, HYSR determined that each gage record could be extended by the other. This allowed the choice of two Index Gages to generate estimates of daily inflows to all the study water supply reservoirs and to extend the Segreganset River back to 1940 to determine the Somerset Reservoir Firm Yield during a period before the USGS streamgage was sited on the river.

4.6 Extending the USGS Index Streamgage Records

When used to estimate daily flows at an ungaged site, a FDC is constructed using GIS-based, watershed-specific climate, soil and topography factors, as discussed earlier. The FDC describes the range and frequency of daily streamflows. Development of the FDC is a key step toward estimating daily streamflows in the QPPQ Transform method. Because observed historic streamflow records exist at both the Adamsville Brook and Segreganset River USGS gage sites, it is not necessary to use the GIS-based watershed factors to construct a FDC at either site. Instead, a FDC was developed directly from the historic daily flows available from the historical records from each stream gage site and the historic record of each site is sampled to provide estimates of the 3-parameter GPA probability function.

By using one gage as the Index Gage and its time series used to drive Steps 1 and 2 of the QPPQ Transform for the other "ungaged" site, which had its step 3 FDC constructed by that site's estimated GPA parameters, determined from that gage's data, the period-of-record of the latter site is extended. The process is repeated where the other second site plays the role of the Index Gage and the first site is the "ungaged" site. The result was that the Adamsville Brook period-of-record was extended forward in time from 1978 to 2023 and the Segreganset River period-of-record was extended backward in time from 1966 to 1940.

With two Index Gages to choose from, each having a POR from 1940-2023, the question was, which one was preferred to drive the QPPQ Transform at which study site? A table of GPA model variables was compiled for each study watershed as needed for the GPA parameter models. Using StreamStats and the coordinates of each streamgage, HYSR also estimated the distance between the geographic mid-point of each study watershed and each streamgage. Comparing a particular reservoir's watershed variables to those of each gaged watershed, the gage with the most closer matches was chosen. The distance between the watershed center-point and the gage was given the same weight of consideration as each GPA parameter variable. Because the PREC and TEMP are the same for each of the study watersheds, those variables weren't considered. In the end, Adamsville Brook was used to drive the QPPQ Transform to generate inflows only to the Copicut Reservoir. The Segreganset River was used to drive the QPPQ Transform to generate inflows for North and South Watuppa Ponds, the Somerset Reservoir, the pumping station withdrawal point on the Segreganset River and Stafford Pond in Tiverton, RI.

4.7 References

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5.1 Introduction to the Fall River Water Supply System

The present-day Fall River, MA water supply system is comprised of three bodies of water: North and South Watuppa Ponds and the Copicut Reservoir. North and South Watuppa Pond are located in East Fall Fiver and border on Westport, MA. Together they comprise the second largest natural water body in the Commonwealth; the Assawompset Pond Complex (APC) in Lakeville and Freetown is the largest. All three are glacial kettle ponds, formed during the retreat of the last glacial period. The Copicut Reservoir, which was constructed in 1972, is a compacted earth dam which is located on the border between the City of Fall River and the Town of Dartmouth, MA. Fall River owns the North Watuppa Pond and Copicut Reservoir watersheds and water rights.

The Fall River Water Filtration Plant is located on the west side of North Watuppa Pond. Approximately 11 mgd of raw water flows by gravity into the raw water sump and then lifted by four raw water pumps into the treatment process train. Raw water is pumped from the Copicut Reservoir depending on both the season and the water level in North Watuppa Pond by way of a pipeline that discharges to a site on Blossom Brook which is located a few thousand feet from the eastern short of North Watuppa Pond.

A compacted earth dam is located between North and South Watuppa Pond on a thin spit of land that would occasionally flood and was formerly known at the Narrows. The hydraulic connection between North and South Watuppa Ponds is by way of the Narrows Gate House, which is located on the southeast side of North Watuppa Pond. Due to water quality considerations, historically, South Watuppa Pond has been used as primarily as an industrial water supply source and as an emergency municipal supply source but when pumping water from the Copicut Reservoir was either insufficient or not possible and North Watuppa Pond was drawn down to dangerous levels. Otherwise, the two are not hydraulically connected.

5.2 A Brief History of the Fall River Water Supply system

The Fall River municipal water supply system was formally established in 1871. Its purpose was to provide healthier potable water for the City by purchasing and hydraulically isolating North Watuppa Pond from South Watuppa Pond and the Quequechan River with the construction of the North Watuppa Dam and Narrows Gate House. The dam and the gate house are located on the spit of land that historically separated North and South Watuppa called The Narrows. Historically, South Watuppa Pond and the Quequechan River were the primary source of first hydro-mechanical power and later industrial process and steam boiler make-up water for the textile industry. The City constructed “Interceptor Channels” on both the east side and west side of North Watuppa Pond in the 1870s to ensure that polluted runoff from the east side of Fall River and the west side of Westport was diverted from North Watuppa Pond to South Watuppa Pond.

According to a report prepared by Hayden et al (1954), the first dam on the Quequechan River was built about 1700 on the west side of Main St which formed only a small mill pond. Around 1830, this dam was removed and replaced with a new dam which raised the level of the mill pond by three feet. Mills were locating along the river for hydro-mechanical power at which point the Watuppa Dam was built in 1827 impounding the Quequechan River, raising the river five feet above its original elevation. Constructed and owned by the Watuppa Reservoir Company, the Watuppa Dam was located near the intersection of Pleasant St. and Fourth St.

As discussed by Conforti (1996), North and South Watuppa Ponds were a single body of water, partially separated by an isthmus of sand extending approximately equidistant from the western and eastern shores. In the early 1800s, the first action taken to divide the lake into separate ponds occurred when stepping stones were laid across the isthmus. The first road across "The Narrows" in Fall River was the Watuppa Turnpike, built in 1827. It was a 49.5-foot toll road that started at Plymouth Avenue, ran beside Pleasant Street, crossed The Narrows, and ended at Blossom Road in Westport. In 1875, the Old Colony Railroad constructed the Watuppa Branch, alongside the road to connect Fall River with New Bedford. Vintage postcards from the early 1900s illustrate what the Narrows looked like.

Figure 5.2.1a The Narrows

Figure 5.2.1b The Narrows

Figure 5.2.1c The Narrows

Figure 5.2.1d The Narrows

The Watuppa Water Board was established in 1871 as the City of Fall River Board of Water Commissioners. The Board was responsible for the construction of the Fall River Waterworks, including a coal-fueled, steam-fired pumping station, a raw water intake house, and a 121 foot tall standpipe (water tower) between 1872 and 1875. A legal battle between the Watuppa Water Board and the Watuppa Reservoir Company which began in 1871 was finally ended in 1897. Now the City, gained full control of the water rights to North Watuppa Pond and began to acquire all privately held property in the North Watuppa Pond watershed. The system began delivering potable water to the public in 1874. The intake-house, pumping station and standpipe are located on Bedford St. on the southwest side of North Watuppa Pond. By 1876, over 45 miles of water pipe, ranging from 6 to 24 inches in diameter, had been installed.

Figure 5.2.2 Fall River Water Tower

Figure 5.2.2 shows the original water tower/stand pipe which was used to pressurize the network of water supply pipes. Figure 5.2.3 shows the original coal-fired steam powered pumping plant and raw water intake structure. The intake structure is off-shore to the right. Figure 5.2.4 is a section of a US Geological Survey topographic maps published in 1888. It slows the location of the original Watuppa Dam, built on the Quequechan River, The Narrows and Lake Noquochoke, which is located on the town line between Fall River and the Town of Dartmouth. Figure 5.2.5 is a map of the Quequechan River and downtown Fall River by Hayden et al (1954).

Figure 5.2.3a The Original Fall River Pumping Station

Figure 5.2.3b The Pumping Station and Water Intake

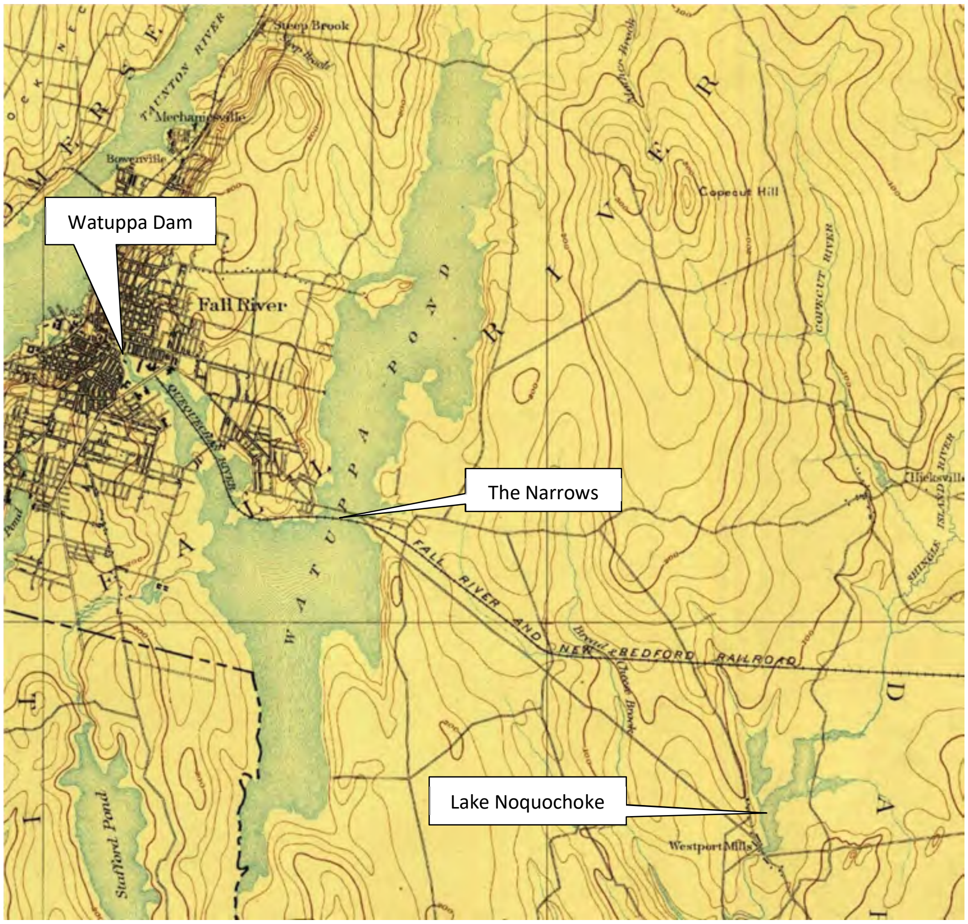


Figure 5.2.4 Fall River Quad Sheet 1888

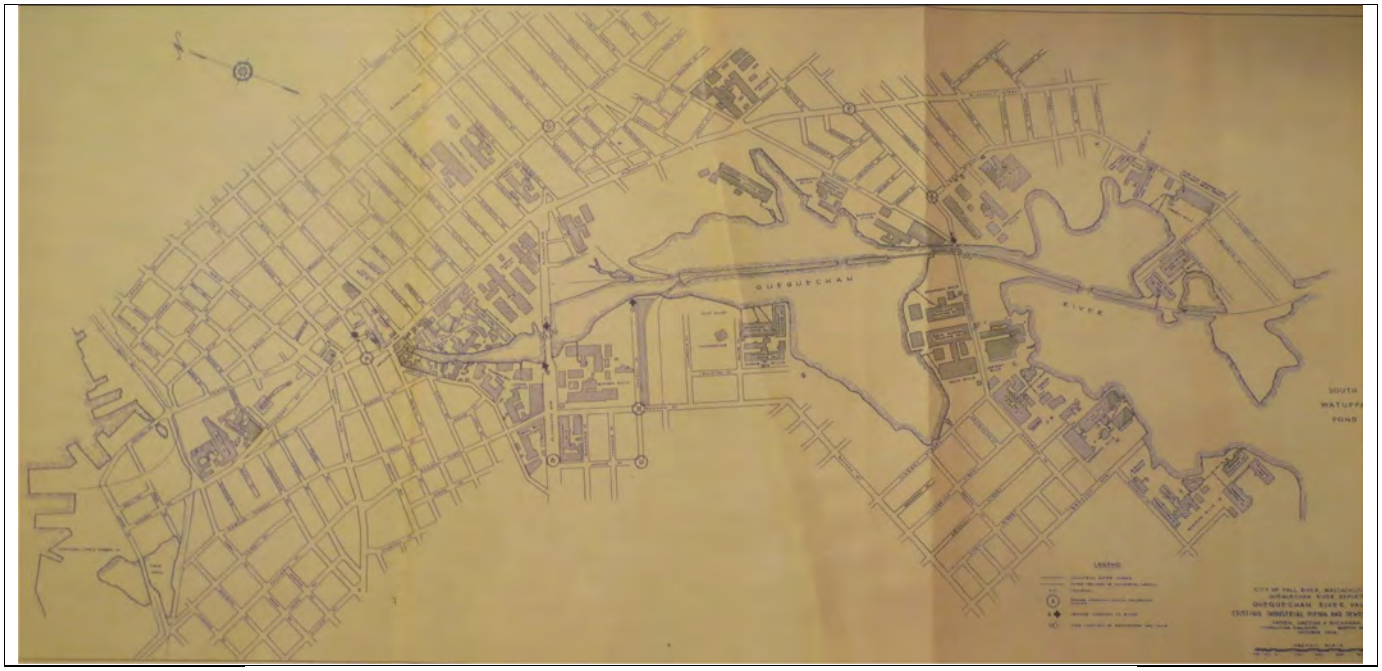


Figure 5.2.5 1954 map of the Quequechan River and Downtown Fall River

Water flowed from the North Pond to the South Pond through the Narrows until the isthmus was fully filled by the construction of the New Bedford Road in the 1880s which later became Rt. 6. Flow between the North and South Ponds has been governed by the Narrows Gatehouse, which was constructed in 1903, and shown below in Figure 5.2.5. The Gatehouse is only rarely used now. During extreme high flow events, flood water from North Watuppa flows into South Watuppa when Adirondack Rd on the east side is overtopped. South Watuppa Pond discharges to the Quequechan River.

Figure 5.2.5a The Narrows Gatehouse

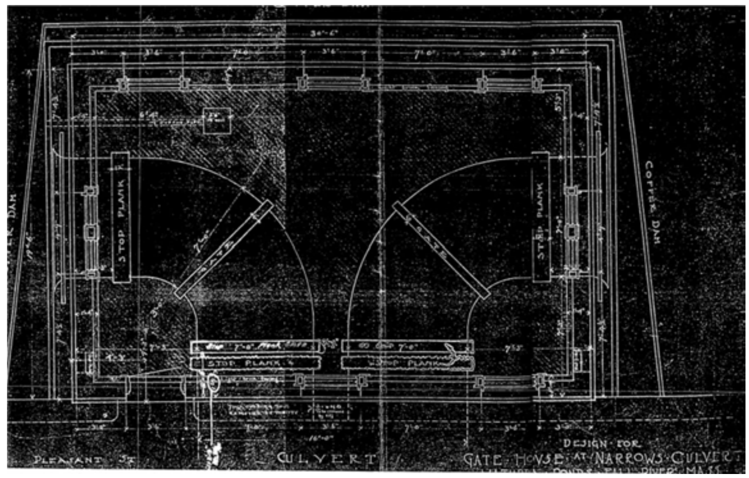


Figure 5.2.5b 1903 Narrows Gatehouse Blue Print

According to a brief prepared by the Watuppa Water Board dated March 15, 1945, in 1937, the Firestone Tire and Rubber company expressed interest in establishing a manufacturing facility in Fall River provided that the City could guarantee a 20 mgd supply. Because North Watuppa Pond was now set-aside solely as a municipal water supply source, the only alternative was South Watuppa Pond. The Watuppa Water Board conducted a study to determine its facility to supply 20 mgd. At that time, the Board discovered that that the Westport Factory property and water rights to Lake Noquochoke and all its tributaries were for sale. Lake Noquochoke is located on the Copicut River which is the border between Fall River and Westport. A vintage postcard of Lake Noquochoke from the early 1900s, is shown below as Figure 5.2.6.

Figure 5.2.6 Lake Noquochoke on the Copicut River

As a matter of detail, Hayden et al. (1954) reported two prior studies; presumably those consulted by the Water Board in 1937. The first conducted in 1910 by Arthur T. Stafford and the second in 1915 by Fay, Spofford and Thorndike. Both established the "safe Yield" of South Watuppa Pond to be 20 mgd "6 days a week, 365 days a year, if a maximum drawdown of eight (8) feet is allowed." Hayden et al. (1954) as part of their water pollution study, recommended that a supply of 30 mgd be developed to ensure short-term levels of high water demand but that on average, 20 mgd should be planned for.

According to the Water Board March 15, 1945 brief, the construction of a pumping station and a raw water transmission line connecting Lake Noquochoke with South Watuppa Pond went on-line in November 1943. By this point in time Firestone Tire and Rubber had fully acquired the water rights of the Watuppa Reservoir Company. Because of the level of pollution in the Quequechan River, Hayden et al. (1954) recommended the construction of a new dam, on the western side of South Watuppa Pond, located to the east of Brayton Avenue. The proposed dam is shown below in Figure 5.2.7

Figure 5.2.7 1954 Proposed S. Watuppa Pond Dam

Interstate Rt. 195 was constructed through the Narrows in the mid-1960s, which is when the North Watuppa Pond Dam was built. The dam proposed by Hayden et al (1954) was never constructed. Instead, Rt 24, which was built at the same time at Rt. 195, crossed the eastern side of the Quequechan River changing the flow patterns in the river. The outlet of South Watuppa Pond and the headwater of the Quequechan River is now a box-culvert located beneath the Rt. 24 Brayton Avenue exit ramp, located near the northwestern corner of South Watuppa Pond. This is shown in yellow-highlight on the left of Figure 5.2.8. The drawing on the right of Figure 5.2.8 is the culvert which is the outlet of the Narrows Gatehouse on North Watuppa Pond, connected with South Watuppa Pond.

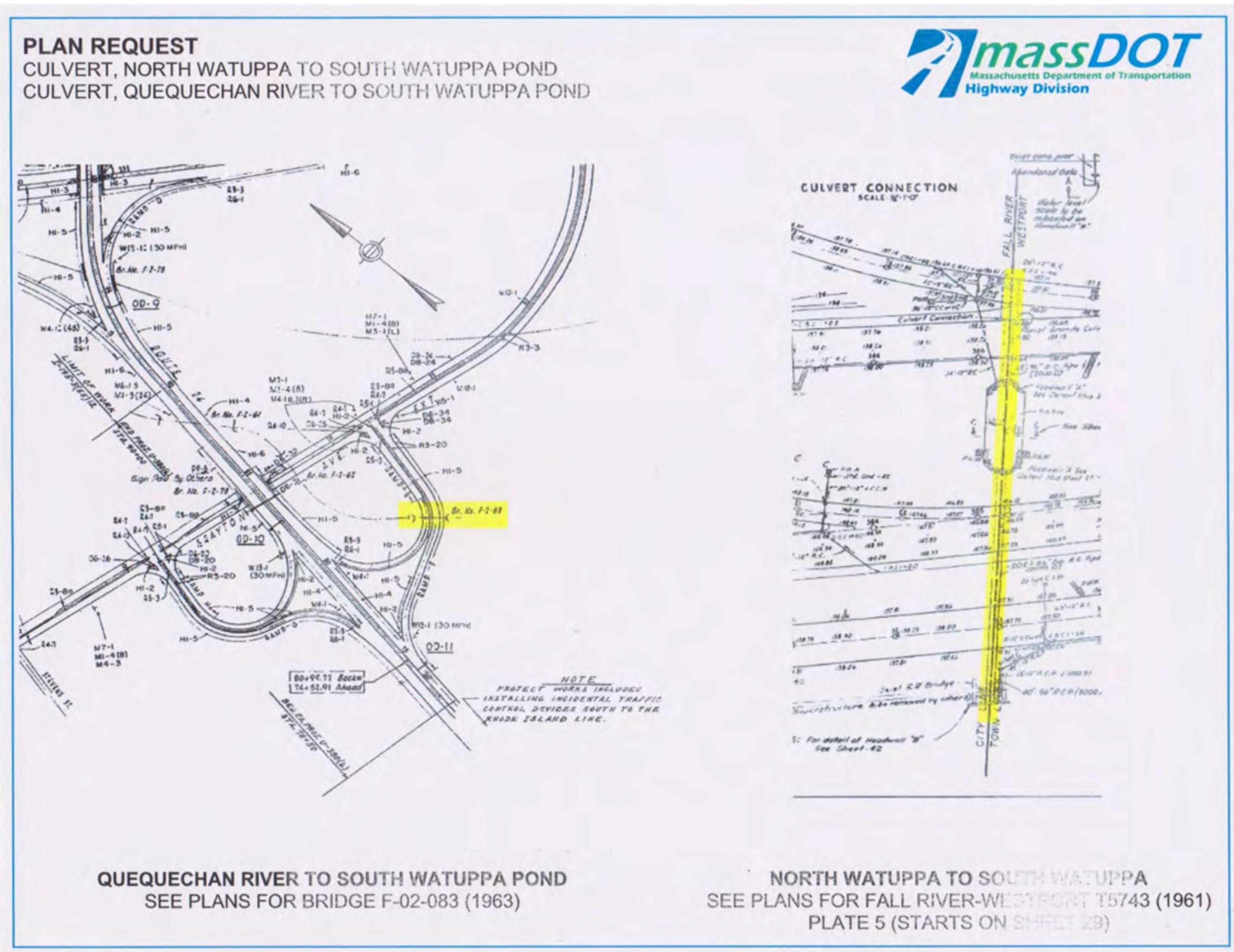


Figure 5.2.8 Rt. 1961 Drawing of the River 24 Box Culvert/S. Watuppa Pond Outlet and Rt. 195/Narrows Gatehouse/ North Watuppa Pond to South Watuppa Pond Culvert

The central to western section of the Quequechan River between Plymouth Avenue and the site of the Watuppa Dam on Pleasant Street and Fourth Street now flows through an eight foot diameter subsurface concrete pipe that was also built at the time of the Rt 195 construction. The Plymouth Ave culvert forebay is shown below as Figure 5.2.9. The river surfaces briefly at what is now referred to as the Fourth Street Gate House, enters another underground pipe which exits to the surface at the Troy Dam site discharging to either Firestone Pond or the Taunton River estuary. The 4th Street Gate House forebay is shown below as Figure 5.2.10.

Figure 5.2.9a. Quequechan R. Plymouth Ave. Forebay

Figure 5.2.9b. Quequechan R. at Plymouth Ave.

Figure 5.2.10a. Quequechan R. 4th Street Gatehouse Forebay

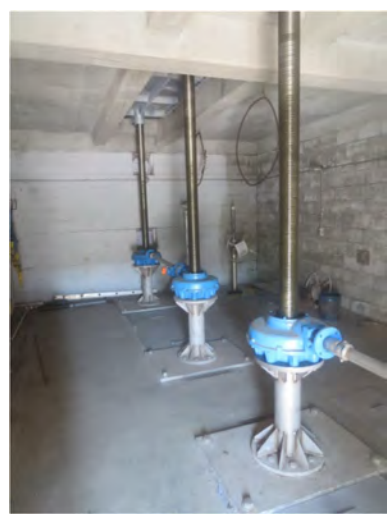


Figure 5.2.10b. 4th Quequechan R. 4th Street Gatehouse Vertical Overflow Weir Sluice Gate Hoists

Figure 5.2.11 is a 1962 design drawing of the 8 foot diameter Reinforced concrete culvert through which the Quequechan River runs and the 4th Street Gatehouse forebay and weir with twin sluice gates.

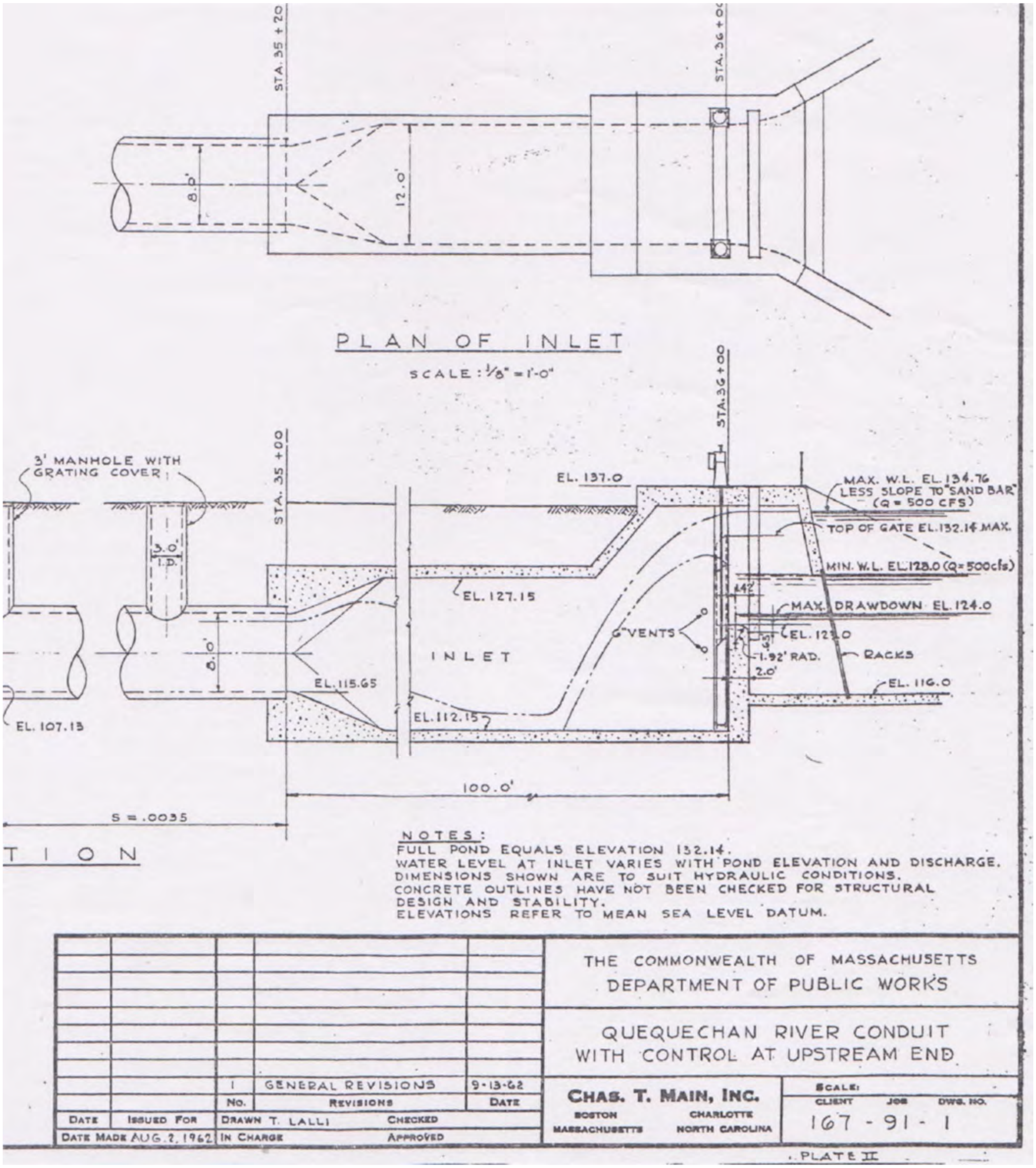


Figure 5.2.10 1962 4th Street Gatehouse Plan View and Facing View Drawings

In a report prepared Whitman and Howard (1958), planning for the construction of the Copicut Reservoir, which is located in Dartmouth, began during the mid-1950s. Construction was completed in 1972, adding extra reliability and high quality water which is pumped over the divide between the North Watuppa and the Copicut watersheds, into Blossom Brook, which flows west into North Watuppa Pond.

Figure 5.2.11a Copicut Reservoir Intake Tower

Figure 5.2.11b Copicut Reservoir Dam

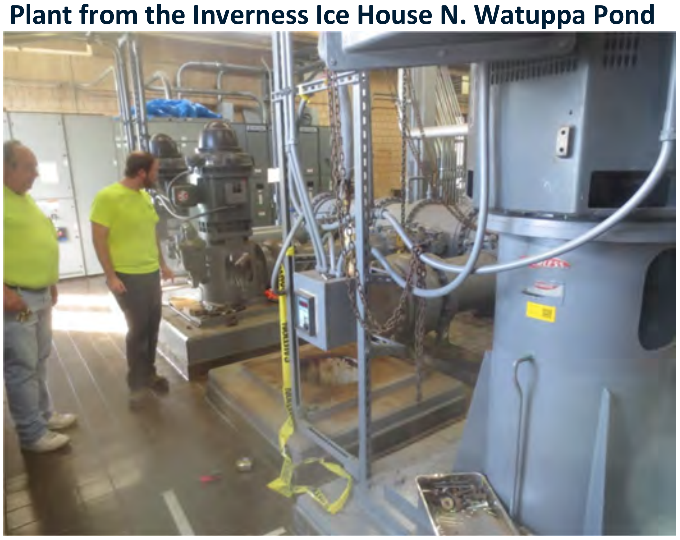
Figure 5.2.11c Copicut Reservoir Spillway

Figure 5.2.11d Copicut Reservoir Pumping Station

The production, treatment and distribution system has expanded significantly over the years, including the construction of a modern filtration plant in 1976. Today the Fall River water supply provides safe drinking water to over 90,000 people in Fall River, the town of Westport, Freetown and Tiverton, RI at an annually averaged daily rate of about 11 million gallons of water per day (mgd) and a peak day capacity of about 20 mgd.

Figure 5.2.12a Old Pumping Plant and 1976 Filtration Plant from the Inverness Ice House N. Watuppa Pond

Figure 5.2.12b N. Watuppa Pond Filtration Plant Raw Water Intake Buoy



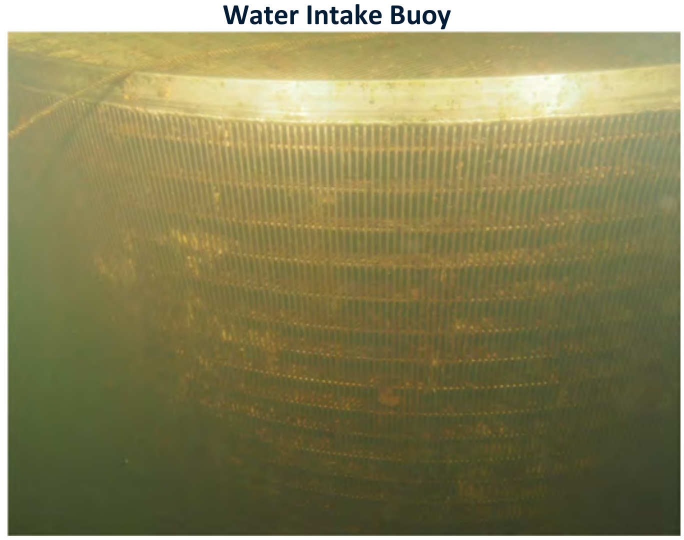


Figure 5.2.12c Filtration Plant Staff and Raw Water Sump Level Survey

Figure 5.2.12d Raw Water intake Screen

One particularly important task undertaken during the study was to establish the reference elevation of the major components of the North Watuppa Pond system. For many years, the Water Department Staff referred to staff gage markings on a prior intake house in North Watuppa Pond but no one knew how that reference tied into the National Geodetic Vertical Datum of 1929 (NGVD 29), commonly known as “mean sea level” (msl). Thanks to Mr. Ferland’s assistance and various Pare Corporation Phase I engineering reports (Pare, 2021a, 2021b, 2020), HYSR conducted a level survey and tied the Full Pond reference to 1962 design drawings of the Rt. 24 Brayton Ave. box culvert, establishing the Full Pond elevation as 132.0 ft (msl). A design drawing, show above as Figure 5.2.13 shown the elevation of the raw water pump intake bell and the floor of the raw water sump at 120.0 ft (msl).

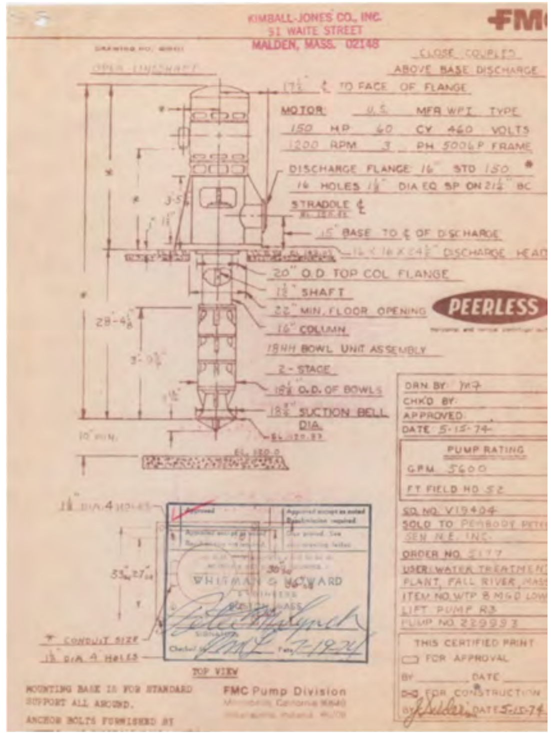


Figure 5.2.13 Fall River Filtration Plant Raw Water Pump Drawing with Elevations

HYSR had the diver photograph and measure dimensions and depth the raw water intake screen (Figure 5.2.12d above) to measure the depth of the raw water intake in North Watuppa pond, which is also approximately 120.0 ft (msl). These elevations are critical to properly analyzing the system Firm Yield.

5.3 The Massachusetts Water Management Act.

The city is presently registered with MA DEP under the auspices of the Massachusetts Water Management Act for an annual average day withdrawal rate of 14.56 million gallons per day (mgd). The City presently uses about 11 mgd. Because the City used South Watuppa Pond during the drought of the early 1970s, as an emergency source, Copicut Reservoir, North and South Watuppa Pond are registered sources. Their registration was renewed by MA DEP a couple of years ago and so the City still has access to South Watuppa Pond. The City also owns Nokachoke Pond, which was created by the construction of a reinforced concrete dam, and is located on the border between Fall River and Dartmouth. Nokachoke Pond is fed by the Copicut River. As discussed in the previous section, a pumping station is located on the west bank of Lake Nokachoke and raw water was pumped via pipeline and discharged to South Watuppa Pond. Because this source was not in use during the 1980-85 MA DEP Water Management Act registration period, the City has no access to this water until they apply for and are granted a withdrawal permit.

5.4 North Watuppa Pond

5.4.1. The North Watuppa Pond Watershed

North Watuppa Pond is located on the eastern side of the Fall River, It is impounded by the North Watuppa Pond Dam and has a watershed area of approximately 8.78 mi.^{2} (5,619 acres). The watershed is delineated by the blackline seen in Figured 5.4.1 below. The outlet, defined as the Narrow Gate House, is located at approximately N 41°40'51" latitude W 71°07'08" longitude. The surface area, at what the City refers to as Full Pond (132 ft msl) is approximately 1779 acres.

Figure 5.4.1.1 North Watuppa Pond Watershed

A summary of the North Watuppa Pond and watershed information is provided in Table 5.4.1. In addition to general information, the table also lists the independent variables of the GPA parameter equations as was described in the QPPQ Transform method used to estimate daily inflows and discussed earlier in this report.

Table 5.4.1.1
Watershed Summary Data
North Watuppa Pond, Fall River, MA

Watershed Area, AREA (mi^2)	8.78
Normal Pool Elevation (ft MSL)	132
Normal Pool Surface Area (acres)	1779
Area of Lakes, Ponds & Reservoirs (%)	27.1
Area of Impervious Surface (%)	4.7
Area of USDA NRC HSG A (%)	6.3
Area of USDA NRC HSG C (%)	29.4
Area of USDA NRC HSG D (%)	17.3
Mean Annual Precipitation, PREC (inches)	46.1
Mean Annual Temperature, TEMP (^{o}F)	51.1
Mean Annual Snowfall, SNOW (in/yr)	42.3
Mean Annual Lake Evaporation, E_p (in/yr)	45.7
Mean Annual Ref. Evapotranspiration, E_t (in/yr)	35.8
Mean Watershed Elevation, ELEV (feet MSL)	162
Main Stream Channel Slope, C-SLOPE (ft/mile)	16.1
Watershed Aspect relative to true north (degrees)	180.5

5.4.2. North Watuppa Pond Bathymetry and Its Stage-Storage-Area.

A bathymetric map of North Watuppa Pond is shown below as Figure 5.4.2.1. Table 5.4.2.1 below is the Stage-Storage-Area data for North Watuppa Pond. The normal pool ("full pond") is at elevation 132 ft (msl). The invert of the raw water sump in the Water Filtration Plant is 120.0 ft (msl). The invert of the raw water intake, which is located approximate 100 feet off-shore is also located at approximately 120 ft (msl). The invert of the North Watuppa Pond Dam gate house, also referred to as the Narrows Gate House, which connects North Watuppa Pond with South Watuppa Pond, is located at 125.7 ft (msl).

One of the Fall River Filtration Plant's four raw water pumps is rated at 5600 gpm. The others are rated for less. Each are equipped with 18.25 in diameter intake bell which is located 10 in above the floor of the raw water sump at elevation 120.8 ft (msl). According to the ANSI/HI 9.8 Pump Intake Design standard, the minimum submergence of each pump's intake is 58 in to avoid vortexing and air entrainment. This places the minimum submergence elevation at approx. 125.7 ft (msl). Using Fall River's MA Water Management Registration of 14.88 mgd (2583 gpm) for comparison, assuming that all four VFD equipped pumps operate simultaneously at this reduced rate, the minimum submergence depth falls to 51 in (4 ft) which corresponds to 125 ft (ms).

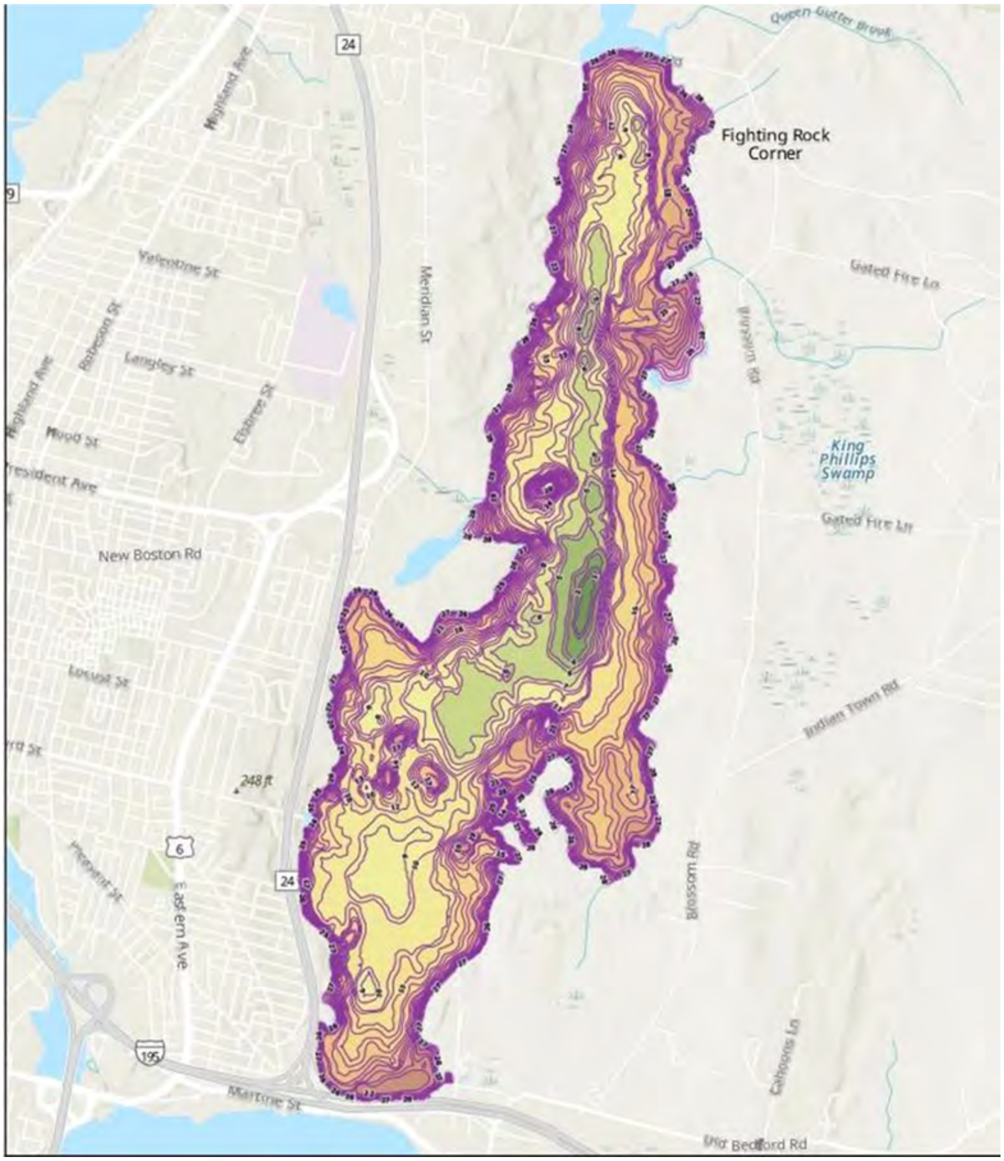


Figure 5.4.2.1 North Watuppa Pond Bathymetry

For this study, HYSR will use the 5600 gpm 125.7 ft (msl) minimum submergence criteria for all four pumps to define the active storage lower limit. The active storage in North Watuppa Pond, which lies in the estimated volume of water between 125.7 and 132 ft (msl) is estimated to be 9953.6 MG, as shown in the last column. This is the range the system is normally operated without risking overtopping the impoundment or damaging the pumps. This data is also shown graphically in Figure 5.4.2.2 below.

Table 5.4.2.1
North Watuppa Pond, Dartmouth, MA
Stage, Storage, Area

Stage (ft msl)	Area (acres)	Storage Vol (acre-ft)	Comment
104	0.0	0.0	
105	9.5	4.8	
106	18.6	18.8	
107	26.2	41.2	
108	53.0	80.8	
109	75.0	144.8	
110	169.7	267.2	
111	242.0	473.1	
112	356.7	772.4	
113	458.5	1180.0	
114	603.5	1711.0	
115	714.1	2369.8	
116	807.2	3130.4	
117	886.0	3977.0	
118	995.1	4917.5	
119	1073.7	5951.9	

Stage (ft msl)	Area (acres)	Storage Vol (acre-ft)	Comment
120	1169.7	7073.7	Raw Water Intake
121	1277.6	8297.3	
122	1347.0	9609.5	
123	1427.4	10996.7	
124	1476.3	12448.6	
125	1523.0	13948.2	
126	1566.0	15492.7	Min Pump Intake
127	1597.4	17074.4	
128	1627.2	18686.7	
129	1657.0	20328.8	
130	1690.3	22002.4	
131	1724.8	23709.9	
132	1748.6	25446.6	Full Pond
137.05	2370.0	35743.1	Dam Crest

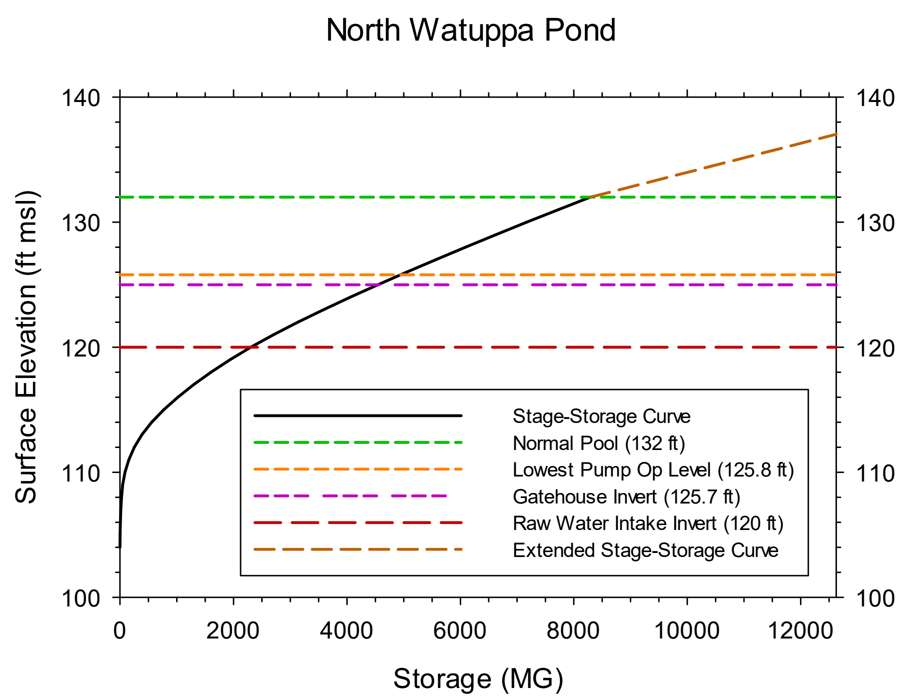


Figure 5.4.2.2 North Watuppa Pond Stage-Storage Curve

5.4.3. The North Watuppa Pond Hydraulics

The North Watuppa Pond system hydraulics are summarized below in Table 5.4.3.1

Table 5.4.3.1
North Watuppa Pond, Dartmouth, MA
System Hydraulics

Dam Crest Elevation (feet msl)	137.05
Normal Pool ("Full Pond") Elevation (feet msl)	132.0
Flood Stage ("Flood Pond") Elevation (feet msl)	134.0
Emergency Spillway Invert (feet msl)	NA
Normal Pool Storage (MG)	8291
Normal Pool Surface Area (acres)	1749
Main Spillway Type	none
Main Spillway Length (feet)	NA
Release Control Type (Narrows Gate House)	Manual sluice gates
Number of Gates	2
Gate Dimensions(s)	48 in wide, 50 in high
Gate Invert Elevation (feet msl)	125.7

5.5 South Watuppa Pond

5.5.1. The South Watuppa Pond Watershed

The South Watuppa Pond watershed area is approximately 18.1 mi.^{2} (11,571 acres). The outlet is located at approximately N 41^{o}40'46" latitude W 71^{o}08'14" longitude. The surface area of the Pond when full, and the Normal Elevation (130-130.5 ft msl) is approximately 1404 acres. A map of the watershed is shown below as Figure 5.5.1.1. The perimeter of the watershed is shown as a black line, an important feature of the South Watuppa Pond watershed are the areas delineated with green lines on the western and eastern sides of North Watuppa Pond and respectively label as the Western and Easter Inceptor Channel watersheds. Both of these sub-watersheds were once part of the North Watuppa Pond watershed but following the construction of the Western and Eastern Interceptor channels for water quality protection purposes in the later 19th century, both now drain into South Watuppa Pond.

South Watuppa Pond itself drains to the Quequechan River, which is located to the west of South Watuppa Pond. The actual outlet of the Pond and headwaters of the Quequechan River, located at the northwestern Corner of South Watuppa Pond is a box culvert, which is located beneath the northbound Rt. 24 exit ramp to Brayton Avenue.

Figure 5.5.1.1 South Watuppa Pond Watershed

A summary of the South Watuppa Pond and watershed information is provided in Table 5.5.1.1

Table 5.5.1.1
Watershed Summary Data
South Watuppa Pond, Fall River, MA

Watershed Area, AREA (mi^2)	18.1
S. Watuppa Pond Normal Pool Elevation (ft MSL)	131
Normal Pool Surface Area (acres)	1404
Area of Lakes, Ponds & Reservoirs (%)	11.8
Area of Impervious Surface (%)	13.3
Area of USDA NRC HSG A (%)	4.3
Area of USDA NRC HSG C (%)	30.2
Area of USDA NRC HSG D (%)	12.6
Mean Annual Precipitation, PREC (inches)	46.1
Mean Annual Temperature, TEMP (^oF)	51.1
Mean Annual Snowfall, SNOW (in/yr)	42.3
Mean Annual Lake Evaporation, E_p (in/yr)	45.7
Mean Annual Ref. Evapotranspiration, E_t (in/yr)	35.8
Mean Watershed Elevation, ELEV (feet MSL)	173
Main Stream Channel Slope, C-SLOPE (ft/mile)	9.8
Watershed Aspect relative to true north (degrees)	181.2

5.5.2. The South Watuppa Pond Bathymetry and Stage-Storage-Area Relationship.

Figure 5.5.2 below is a bathymetric map of South Watuppa Pond. Table 5.5.2.1 below is the Stage-Storage-Area data for South Watuppa Pond developed by an analysis of Figure 5.5.2.1 Because of water quality considerations, South Watuppa is below North Watuppa pond to keep the two hydraulically isolated. According to the Fall River Filtration Plant staff, the South Watuppa Pond Normal Pool typically ranges between -18 in to -24 in below North Watuppa's "Full Pond" (132.0 ft msl), which would be seasonally dependent as 130.5 or 130 ft (msl). In the spring, the operators prefer to maintain South Watuppa Pond at about -30 in below Full Pond in anticipation of potential rapid snow melt and runoff, which would be 129.5 ft (msl).

The invert of the North Watuppa Pond / Narrows Gatehouse is located at approximately 126 ft (msl) which is the lower limit by which water from South Watuppa Pond is able to flow into North Watuppa Pond without emergency pumping. The invert of the Rt. 24 exit ramp culvert, which is the outlet of South Watuppa Pond, marks the beginning of the Quequechan River, and is located at 118 ft (msl). Although discharge control of the Quequechan River is maintained by the 4th Street Gatehouse, the Filtration Plant staff informed HYSR that the positions of its three manually operated gates are only changed when the stage of South Watuppa Pond needs to be altered to maintain the seasonal Normal Pool.

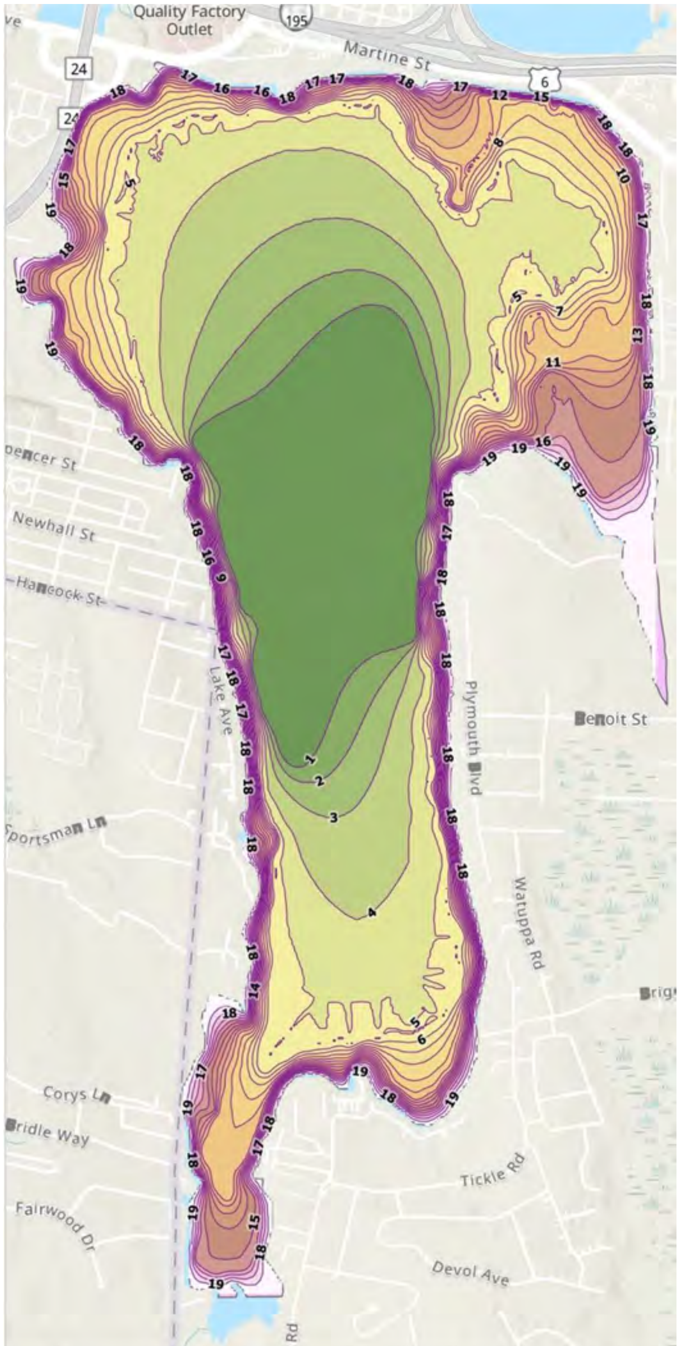


Figure 5.5.2.1 South Watuppa Pond Bathymetric Map

For the present study, since the sole connection between South Watuppa Pond and the Fall River Filtration Plant is through the North Watuppa Pond dam Gate House, the active storage for South Watuppa Pond lies between 126.0 - 130.0 ft (msl). This data is also shown graphically in Figure 5.5.2.2 below.

Table 5.5.2.1
South Watuppa Pond, Fall River, MA
Stage, Storage, Area

Stage (ft msl)	Area (acres)	Storage Vol (acre-ft)	Comment
112	0.0	0.0	
113	261.3	130.7	
114	330.3	426.5	
115	423.4	803.4	
116	591.2	1310.7	
117	822.2	2017.4	
118	951.1	2904.1	Q.R. inlet Rt. 24 Box Culvert Invert
119	991.4	3875.3	
120	1034.4	4888.2	
121	1084.3	5947.5	

Stage (ft msl)	Area (acres)	Storage Vol (acre-ft)	Comment
122	1130.0	7054.7	
123	1188.0	8213.7	
124	1217.6	9416.5	
125	1244.0	10647.3	
126	1272.5	11905.5	Narrows Gate House Invert
127	1317.3	13200.4	
128	1355.2	14536.6	
129	1378.6	15903.5	
130	1404.4	17295.0	Normal Pool
131	1437.7	18716.0	
132	1476.5	20173.1	Flood Pool

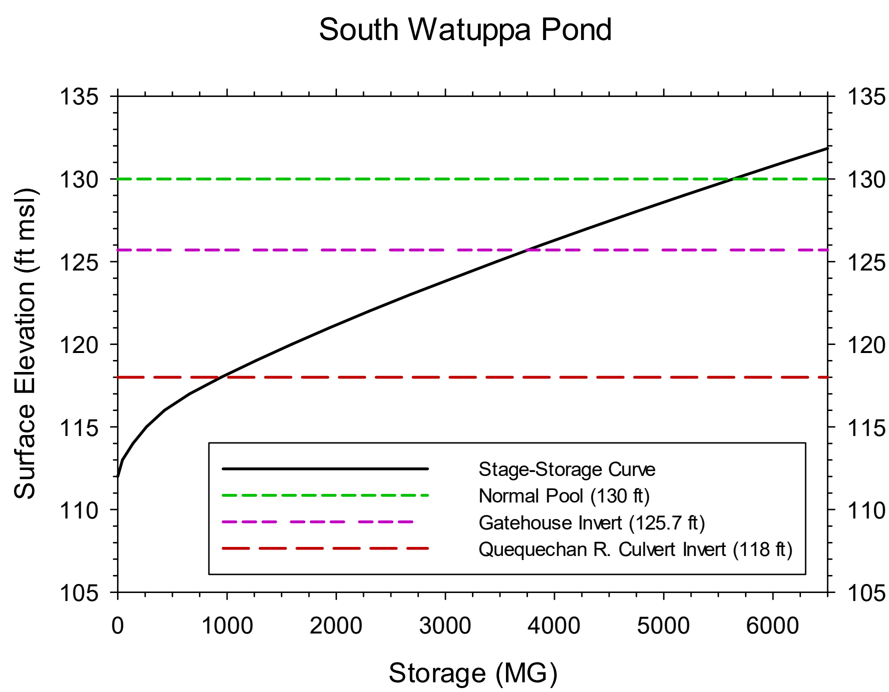


Figure 5.5.2. South Watuppa Pond Stage-Storage Curve

5.5.3. The South Watuppa Pond Hydraulics

The South Watuppa Pond system hydraulics are summarized below in Table 5.5.3.1.

Table 5.5.3.1
South Watuppa Pond, Dartmouth, MA
System Hydraulics

Normal Pool (Weir) Elevation (feet msl)	130.0 – 130.5
Flood Pool Elevation (feet msl)	132.0
Emergency Spillway Invert (feet msl)	NA
Normal Pool Active Storage (MG)	5389
Normal Pool Surface Area (acres)	1404
Main Spillway Type	none
Main Spillway Length (feet)	NA
Primary Outlet	Rt 24 12 ft x15 ft box culvert
Quequechan R. Primary Release Control	Movable overflow weirs (2)
Weir Width (feet)	10
Operating Range (feet msl)	118-131
Quequechan R. Secondary Release Control	Valved outlet pipes (2)
Low Level Outlet Pipe Diameter (ft)	8.0
Low Level Outlet Pipe Invert (feet msl)	108.7

5.6 Copicut Reservoir

5.6.1. The Copicut Reservoir Watershed

The Copicut Reservoir is located on Copicut Brook in Dartmouth, MA. It lies in the Buzzards Bay watershed. The watershed area is approximately 6.63 mi.^{2} (4,243 acres). The outlet is located at approximately N 41°42’02” latitude W 71°02’31” longitude. The Copicut Reservoir watershed is delineated by the black-line on Figure 5.6.1.1. A summary of the Copicut Reservoir and watershed information is provided in Table 5.6.1.1 below.

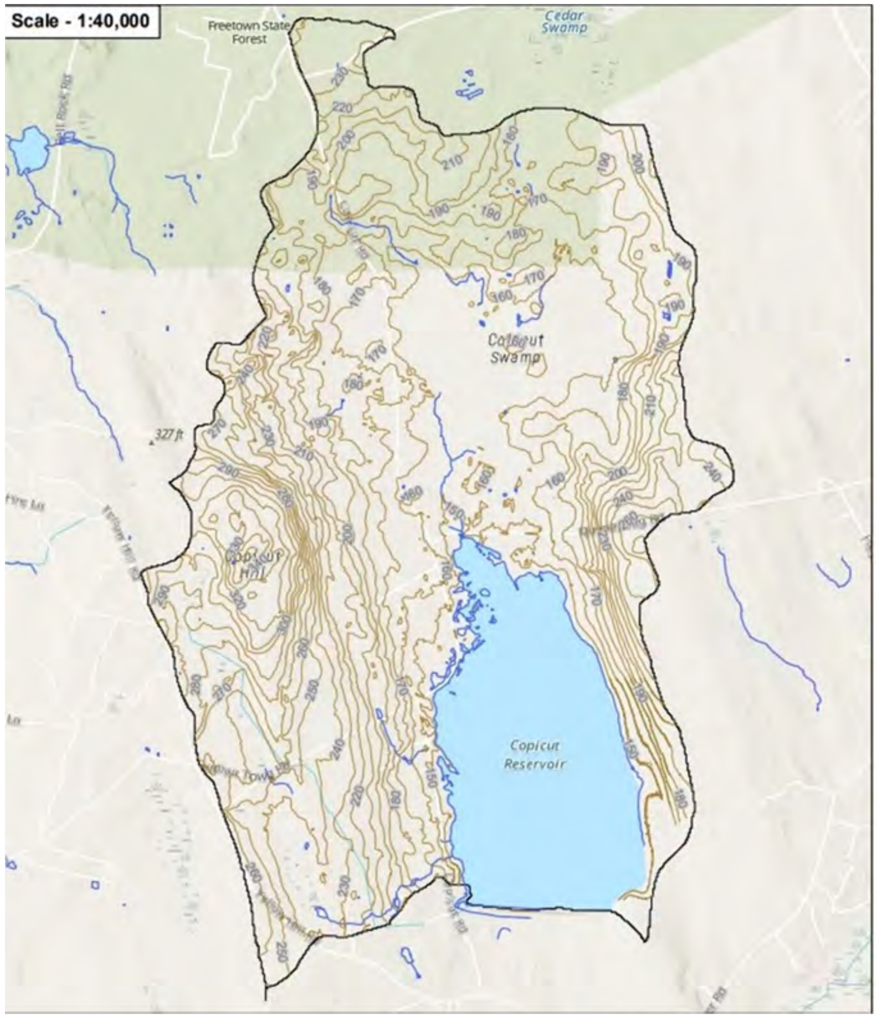


Figure 5.6.1.1 Copicut Reservoir Watershed

Table 5.6.1.1
Watershed Summary Data
Copicut Reservoir, Dartmouth, MA

Watershed Area, AREA (mi^2)	6.63
Normal Pool Elevation (ft)	143
Normal Pool Surface Area (acres)	629
Area of Lakes, Ponds & Reservoirs (%)	14.6
Area of Impervious Surface (%)	0.14
Area of USDA NRC HSG A (%)	9.4
Area of USDA NRC HSG C (%)	39.9
Area of USDA NRC HSG D (%)	20.9
Mean Annual Precipitation, PREC (inches)	46.1
Mean Annual Temperature, TEMP (^{o}F)	51.1
Mean Annual Snowfall, SNOW (in/yr)	42.3
Mean Annual Lake Evaporation, E_p (in/yr)	45.7
Mean Annual Ref. Evapotranspiration, E_t (in/yr)	35.8
Mean Watershed Elevation, ELEV (feet msl)	194
Main Stream Channel Slope, C-SLOPE (ft/mile)	14.9
Watershed Aspect relative to true north (degrees)	179.4

5.6.2. The Copicut Reservoir Bathymetry and Stage-Storage-Area Relationship

Figure 5.5.2 below is a bathymetric map of South Watuppa Pond. Table 5.6.2.1 below is the Stage-Storage-Area data for South Watuppa Pond developed by an analysis of the bathymetry data used to create Figure 5.6.2.1. Copicut Reservoir was built in response to the drought of the 1960s. Construction was finished in 1972. According to the Fall River Filtration Plant personnel, Copicut Reservoir generally operates as a stand-by/supplemental system. The normal pool is at elevation 143 ft (msl). Raw water is withdrawn from Copicut Reservoir by way of three gates 20 in rectangular openings, which are located on the side of a tower structure, which is close to the dam. This system provides flexibility so the operator may choose the elevation from which to withdraw raw water depending on quality considerations and how full the reservoir is.

The high level raw water intake invert is located at elevation 133.0 ft (msl), the mid-level intake which is the one regularly used, is located at elevation 123.0 feet (msl) and the low-level raw water intake invert is located at elevation 113.5 feet (msl). The invert of the low-level outlet, which is used to drain the reservoir for major maintenance or an emergency, is located at elevation 109.5 ft (msl). The usual active storage, lies between 123 and 143 ft (msl), the range the system which can operated without discharging from the dam spillway.

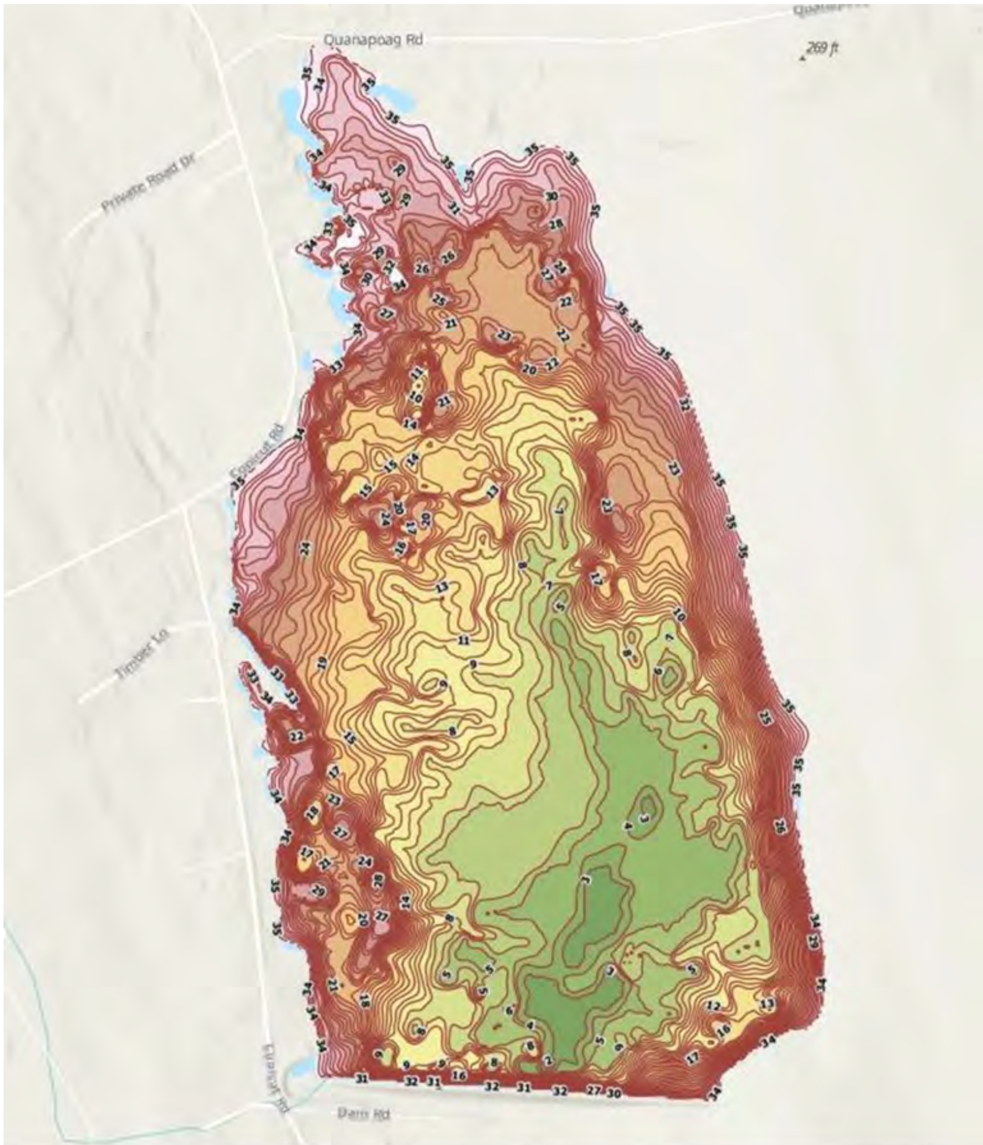


Figure 5.6.2.1 Copicut Reservoir Bathymetry

Table 5.6.2.1
Copicut Reservoir, Dartmouth, MA
Stage, Storage, Area

Stage (ft msl)	Area (acres)	Storage Vol (acre-ft)	Active Store (acre-ft)	Stage (ft msl)	Area (acres)	Storage Vol (acre-ft)	Comment
107	0.0	0.0		126	371.6	3689.7	
108	0.2	0.1		127	384.7	4067.9	
109	11.3	5.8	Low-Level Outlet	128	399.5	4460.0	
110	22.0	22.5		129	421.1	4870.3	
111	58.1	62.5		130	441.0	5301.3	
112	91.5	137.3		131	457.1	5750.4	
113	128.8	247.5	Low-Level Inlet	132	470.0	6214.0	
114	157.5	390.6		133	483.0	6690.5	
115	189.1	563.9		134	496.7	7180.3	
116	209.6	763.3		135	508.9	7683.0	
117	228.7	982.4		136	520.6	8197.8	
118	244.3	1218.9		137	536.6	8726.4	
119	258.8	1470.5		138	552.1	9270.8	
120	276.1	1737.9		139	569.4	9831.6	
121	291.6	2021.8		140	586.1	10409.4	
122	308.3	2321.8		141	598.5	11001.7	
123	326.4	2639.2	Mid-Level Inlet	142	614.3	11608.2	
124	342.7	2973.7		143	621.5	12226.1	Normal Pool
125	358.8	3324.5		145.5	650.0	13815.4	Flood Pool

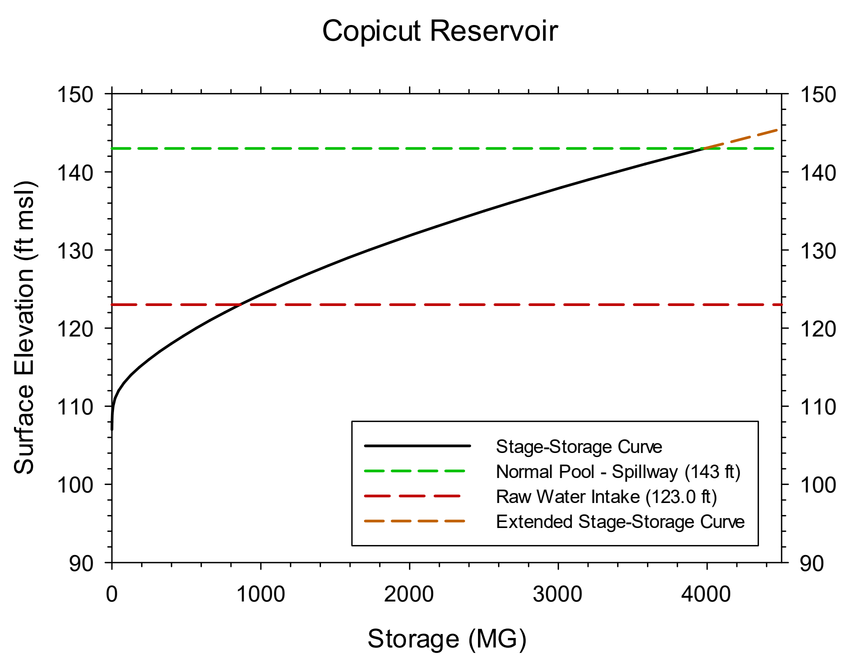


Figure 5.6.2.2 Copicut Reservoir Stage-Storage Curve

5.6.3. The Copicut Reservoir Hydraulics

The Copicut Reservoir system hydraulics are summarized below in Table 5.6.3.1. Raw water is conveyed from the Copicut Reservoir to the North Watuppa Pond watershed by way of a single 24 in diameter pipe. Two 3 mgd pumps are used to lift the raw water from a pumping station located below the dam over the Copicut River-North Watuppa Pond watershed divide. The raw water is discharged from the pipe into a stream which naturally drains into Blossom Brook, which is located a few thousand feet east of North Watuppa Pond.

Table 5.6.3.1
Copicut Reservoir, Dartmouth, MA
System Hydraulics

Dam Crest Elevation (ft msl)	149.5
Normal Pool Elevation (feet msl)	143.0
Emergency Spillway Invert (feet msl)	143.0
Flood Stage Elevation (feet msl)	145.5
Spillway Type	Board-crested weir
Spillway Surface Area (feet)	80 wide x 10
Spillway Geometry	East side: vertical concrete training wall West side: natural cover gradual upward slope
Normal Pool Storage (MG)	3984
Normal Pool Surface Area (acres)	621
Lowest Raw Water Pipe Invert (feet msl)	109.5
Lowest Raw Water Pipe Diameter (feet)	2
Lowest Raw Water Pipe Outlet Control	Manual valve
Low Level Outlet Pipe Diameter (ft)	2
Low Level Outlet Pipe Invert (feet msl)	109.5

5.7 References

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Pare, 2021b, *Quequechan Control Structure Phase I Inspection Evaluation Report*, Report to the City of Fall River, May 6.

Pare, 2020, *North Watuppa Pond Dam Phase I Inspection Evaluation Report*, Report to the City of Fall River, May 6.

Whitman and Howard, 1958, *Report to Watuppa Water Board in Relation to Improving Water Supply by Constructing Copicut Reservoir and Water Supply to Somerset, Taunton (sic) and Dartmouth*. Report prepared for the City of Fall River, April 10.

6.1 Introduction to the Town of Somerset Water Supply System

The Town of Somerset's water supply system consists of the Somerset Reservoir, a pumping station located on the Segreganset River in Dighton and a groundwater well also located in Dighton. Raw water is withdrawn from the Segreganset River during November through May as long as the flows exceed 15 cfs as measured by the US Geological Survey Segreganset River gaging station, which is located about a mile north of the pumping station. Figure 6.1.1 below shows the Somerset Reservoir. Figure 6.1.2 shows the Segreganset River pumping station and Figure 6.1.3 shown the USGS Segreganset River streamgage.

Figure 6.1.1a Somerset Reservoir Dam with New Crest

Figure 6.1.1b Somerset Reservoir Intake Tower

Figure 6.1.1c Segreganset River Pipeline Discharging into the Somerset Reservoir

Figure 6.1.1d Somerset Reservoir Intake Culvert

Figure 6.1.1e Somerset Reservoir Intake By-Pass Channel

Figure 6.1.1e Somerset Reservoir Intake By-Pass Channel Outlets with Blocking Rocks

Figure 6.1.2 Segreganset River Pumping Station

Figure 6.1.2 Segreganset River Pumping Station Dam

Figure 6.1.3a USGS Segreganset River Streamgage

Figure 6.1.3b USGS Segreganset River streamgage Staff Gage

Figure 6.1.3c USGS Segreganset River Streamgage

Figure 6.1.3d USGS Segreganset River streamgage

<u>6.2 A Brief History of the Somerset Water Supply System</u>

The Town of Somerset water supply system has evolved a great deal over time. The town relies primarily on a surface water reservoir to serve residents and businesses in Somerset, parts of Dighton and Swansea. Historically, the town relied on groundwater wells located in Dighton and for some period of time, the City of Fall River. Early distribution system infrastructure dates back to the 1920s when Somerset's Dighton well field was constructed. Most recently, with the upgrade of the Brayton Point Power Plant cooling system, the Somerset Water Department developed a new groundwater well, which is located in Dighton.

The Somerset Reservoir, a compacted earth dam, was constructed in 1965 and recently upgraded with the addition of two feet of structural material to the originally designed crest. An important feature of the system is the Somerset Pumping Station, which was constructed in 1966, is located on the Segreganset River in Dighton. Depending on the flow rate of the Segreganset River and the season, water is pumped from the river through a pipe line which discharges directly into the Somerset Reservoir. Presently the system includes about 95 miles of water distribution mains, a modern water treatment plant and provides water to about 18,000 people each day.

6.3 Somerset Reservoir

6.3.1 Somerset Reservoir Watershed Area

The Somerset Reservoir watershed area is approximately 1.65 mi.^{2} (1,056 acres). The surface area of the reservoir at Normal Pool (full) is approximately 164 acres. The reservoir's low-level outlet is located at approximately N 41°46'38" latitude W 71°08'19" longitude. Figure 6.3.1.1 shows the areal extent of the Somerset Reservoir's watershed by the black line. Table 6.3.1.1 provides a summary of the watershed's characteristics.

Figure 6.3.1.1 Somerset Reservoir Watershed

A summary of the Somerset Reservoir and watershed information is provided in Table 6.3.1.1.

Table 6.3.1.1
Watershed Summary Data
Somerset Reservoir, Somerset, MA

Watershed Area, AREA (mi^2)	1.65
Normal Pool Elevation (ft)	60
Normal Pool Surface Area (acres)	164
Area of Lakes, Ponds & Reservoirs (%)	15.6
Area of Impervious Surface (%)	7.5
Area of USDA NRC HSG A (%)	4.2
Area of USDA NRC HSG C (%)	31.2
Area of USDA NRC HSG C (%)	16.9
Mean Annual Precipitation, PREC (inches)	46.1
Mean Annual Temperature, TEMP (^oF)	51.1
Mean Annual Snowfall, SNOW (in/yr)	42.3
Mean Annual Lake Evaporation, E_p (in/yr)	45.7
Mean Annual Ref. Evapotranspiration, E_t (in/yr)	35.8
Mean Watershed Elevation, ELEV (feet msl)	107
Main Stream Channel Slope, C-SLOPE (ft/mile)	25.3
Watershed Aspect relative to true north (degrees)	178.8

6.3.2 Somerset Reservoir Bathymetry and Stage-Storage-Area Relationship

Figure 6.3.2.1 below is a bathymetry map of the Somerset Reservoir. Table 6.3.2.1 is the Stage-Storage-Area data derived from an analysis of the bathymetry for the Somerset Reservoir. The normal pool is at elevation 56.0 ft (msl). Raw water is withdrawn from the Somerset Reservoir by way of two gates 20 in rectangular openings, which are located on the side of a tower structure, which is located close to the dam. A third gate serves as the low-level outlet intake. This system provides flexibility so the operators may choose the elevation from which to withdraw raw water depending on quality considerations and how full the reservoir is.

The high level raw water intake invert is located at elevation 50.0 ft (msl), the mid-level intake is located at elevation 35.0 feet (msl). The invert of the low-level outlet, which is used to drain the reservoir for major maintenance or an emergency, is located at elevation 18.0 ft (msl). The active storage, which lies between 35.0 and 56 ft (msl), is the range the system which can be operated without discharging from the dam's auxiliary spillway. In the event of a precipitation forecast in excess of 3 inch, the low-level outlet is opened, which discharges to the stream channel below the toe of the dam. Due to potential street flooding, staff report that this discharged is limited to about 5 mgd. Raw water is conveyed from the Somerset Reservoir to the Water Treatment Plant by way of a single 24 in diameter pipe. This data is also shown graphically in Figure 6.2.2.2 below.

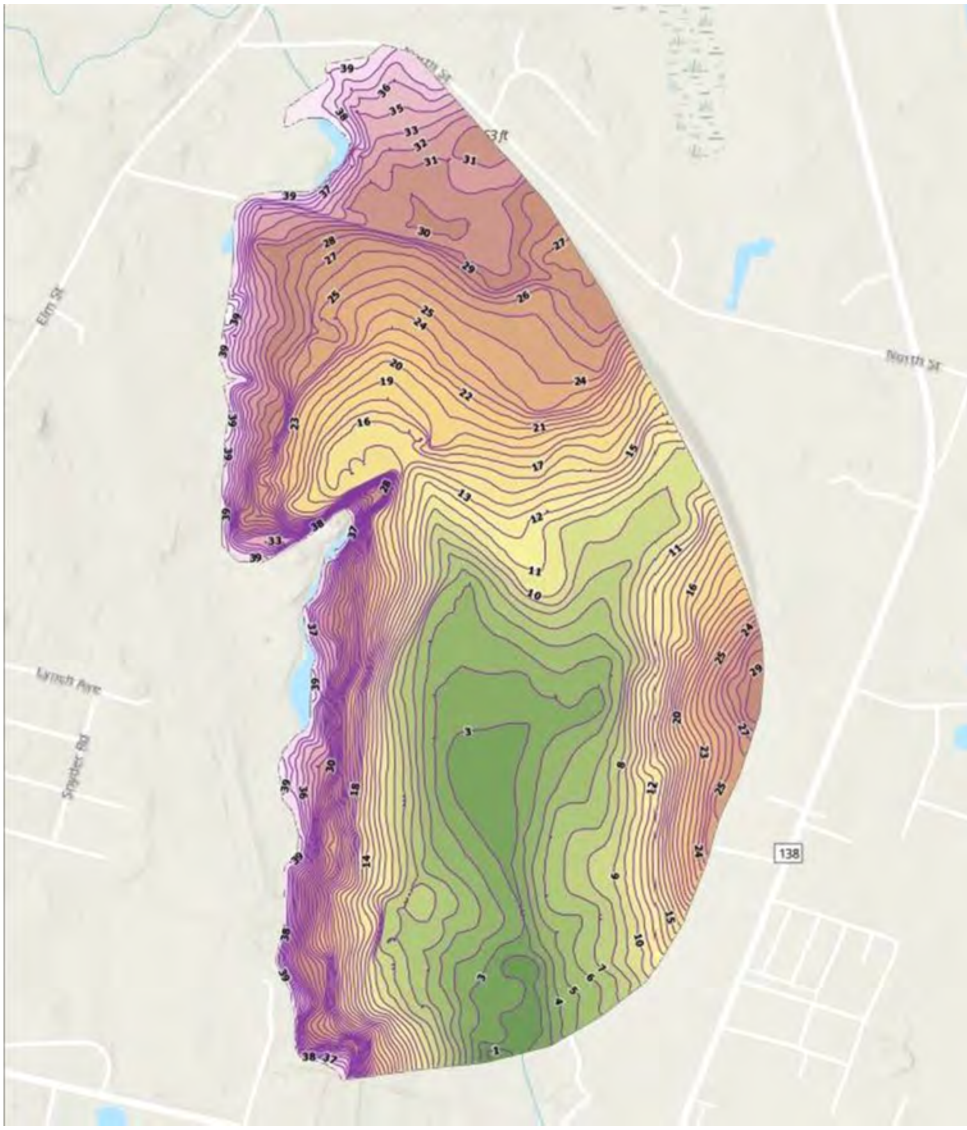

Figure 6.3.2.1 Somerset Reservoir Bathymetry

Table 6.3.2.1
Somerset Reservoir, Somerset, MA
Stage, Storage, Area

Stage (ft msl)	Area (acres)	Storage Vol (acre-ft)	Comment
15	0.0	0.0	
16	1.5	0.8	Low Level Outlet
17	7.6	5.3	
18	16.4	17.4	
19	25.8	38.5	
20	34.0	68.4	
21	42.0	106.4	
22	49.0	152.0	
23	55.7	204.3	
24	62.2	263.3	
25	67.8	328.3	
26	72.7	398.5	
27	76.9	473.3	
28	80.8	552.1	
29	84.6	634.8	
30	89.4	721.8	
31	94.3	813.6	
32	98.1	909.8	
33	102.2	1010.0	
34	106.6	1114.4	
35	110.3	1222.8	Mid-Level Intake
36	113.9	1334.9	
37	117.0	1450.4	
38	120.3	1569.0	
39	124.0	1691.2	
40	128.9	1817.6	
41	134.4	1949.3	
42	139.4	2086.2	
43	143.3	2227.6	
44	147.7	2373.1	
45	151.5	2522.6	
46	155.8	2676.3	
47	163.0	2835.7	
48	167.3	3000.9	
49	170.1	3169.6	
50	173.0	3341.1	High-Level Intake
51	175.7	3515.4	
52	178.1	3692.3	
53	180.9	3871.8	
54	183.8	4054.2	
55	185.4	4238.8	
56	185.5	4424.2	Normal Pool

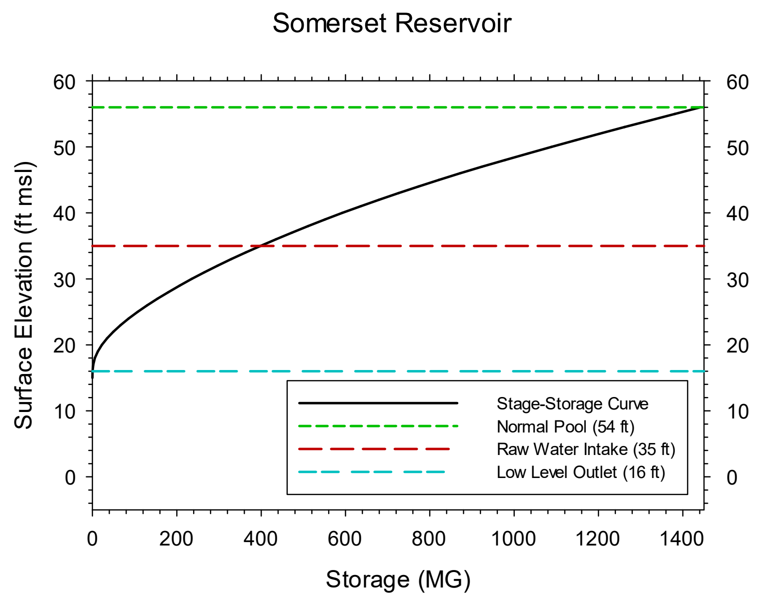

Figure 6.3.2.2 Somerset Reservoir Pond Stage-Storage Curve

6.3.3 Somerset Reservoir Hydraulics

The Somerset Reservoir system hydraulics are summarized below in Table 6.3.3.1.

Table 6.3.3.1
Somerset Reservoir, Somerset, MA
System Hydraulics

Dam Crest Elevation (ft msl)	61.7
Normal Pool Elevation (feet msl)	56.0
Spillway Type	Twin culverts
Spillway Culvert Size (feet)	3
Spillway Culvert Invert (feet msl)	54.7
Flood Stage Elevation (feet msl)	56
Auxiliary Spillway Type	Excavated trapezoidal ditch
Auxiliary Spillway Maximum Depth (feet)	4
Aux. Spillway Top & Bottom Width (feet)	20 & 6
Normal Pool Storage (MG)	922
Normal Pool Surface Area (acres)	1221
Lowest Raw Water Pipe Invert (feet msl)	16
Lowest Raw Water Pipe Diameter (inch)	20
Lowest Raw Water Pipe Outlet Control	Sluice gate
Low Level Outlet Pipe Diameter (inch)	20
Low Level Outlet Pipe Invert (feet msl)	16.0

6.4 The Segreganset River Pumping Station

Raw water is withdrawn on a seasonal basis from the Segregansett River in Dighton, MA. The Town of Somerset's pumping station is equipped with two 6 mgd pumps and the water is conveyed to the Somerset Reservoir by way of a 30-inch diameter steel reinforced concrete pipe. Due to the condition of the pipeline, only one pump is used at a time with the other on-stand-by. The pipe discharges directly into the Somerset Reservoir pool with the invert of its discharge port located below Normal Pool elevation.

6.4.1. The Segreganset River Pumping Station Watershed

The Segreganset River Pumping Station water area is approximately 14.3 mi.^{2} (9,152 acres). . Figure 6.4.1.1 is a map of the Segreganset River watershed, as indicated by the black line. The outlet the southeast corner of the watershed is located at approximately N 41^{o}49'34" latitude W 71^{o}07'39" longitude

Figure 6.4.1.1 Segreganset River Pumping Station Watershed

A summary of Segreganset River Pumping Station watershed information is provided in Table 6.4.1.1.

Table 6.4.1.1
Watershed Summary Data
Segreganset River Pumping Station, Dighton, MA

Watershed Area, AREA (mi^2)	14.3
Normal Pool Elevation (ft)	35
Normal Pool Surface Area (acres)	3
Area of Lakes, Ponds & Reservoirs (%)	0.23
Area of Impervious Surface (%)	4.5
Area of USDA NRC HSG A (%)	10.1
Area of USDA NRC HSG C (%)	17.2
Area of USDA NRC HSG D (%)	22.1
Mean Annual Precipitation, PREC (inches)	46.1
Mean Annual Temperature, TEMP (^oF)	51.1
Mean Annual Snowfall, SNOW (in/yr)	42.3
Mean Annual Lake Evaporation, E_p (in/yr)	45.7
Mean Annual Ref. Evapotranspiration, E_t (in/yr)	35.8
Mean Watershed Elevation, ELEV (feet msl)	98
Main Stream Channel Slope, C-SLOPE (ft/mile)	12.1
Watershed Aspect relative to true north (degrees)	176.1

7.1 The Stone Bridge Fire District Water Supply System Introduction

Stafford Pond, located in Tiverton, RI is a glacial kettle pond with a small dam at its outlet. The pond is located within the South Watuppa Pond watershed. All of its community of users are served by on-site septic systems and for all intents and purposes, most of the water withdrawn from the Pond is recycled. Water treatment plant staff report that the average daily production rates is about 0.8 mgd. In its annual water use reports (see Solley et al, 1988) the US Geological Survey estimates that approximately 10% of the water provided to these users is lost to evapotranspiration by the septic system with the rest being retained in the watershed. The Pond has recently been treated for algae control.

Figure 7.1.1a Stafford Pond Park

Figure 7.1.1b Stafford Pond

Figure 7.1.1c Stone Bridge Fire District Water Tower

Figure 7.1.1d Stafford Pond Dam

7.2 A Brief History of the Stone Bridge Fire District Water Supply System.

The Stone Bridge Fire District water supply system is located in Tiverton, RI. The name "Stone Bridge" refers to the historic stone bridge (and later a steel drawbridge) that once spanned the Sakonnet River, connecting Tiverton and Portsmouth. A vintage postcard of the Stone Bridge in Tiverton, RI is shown below as Figure 7.2.1.

Figure 7.2.1.1 The Stone Bridge, Tiverton, RI

This bridge was eventually replaced by the Sakonnet River Bridge. The Stone Bridge Fire District was established in 1940 to provide water service to the Stone Bridge area of Tiverton. Its sole source of water is Stafford Pond, which is owned by the City of Fall River. The water is treated by a plant that was designed and constructed in the 1980s and distributed through approximately 24 miles of water main, which was constructed during the 1940s and 1950s. The Stone Bridge Fire District is connected to the North Tiverton Fire District which in turn is connected to the City of Fall River. This interconnection safeguards the Stone Bridge Fire District users for periods of time when the Stafford Pond treatment plant has been taken off-line.

7.3. Stafford Pond Watershed

The Stafford Pond watershed area is approximately 2.18 mi.^{2} (1,395 acres). The surface area of the reservoir when full (Normal Pool) is approximately 480 acres. The outlet, located at the north of the watershed, is located at approximately N 41^{o}39’18” latitude W 71^{o}09’38” longitude. A map of the watershed is shown below as Figure 7.3.1.1 where the watershed boundary is shown as the black line. A table of the Stafford Pond watershed characteristics is also shown below as Table 7.3.1.1

Figure 7.3.1.1 Stafford Pond Watershed

Table 7.3.1.1
Watershed Summary Data
Stafford Pond, Tiverton, RI

Watershed Area, AREA (mi^2)	2.17
Normal Pool Elevation (ft)	203
Normal Pool Surface Area (acres)	480
Area of Lakes, Ponds & Reservoirs (%)	36.5
Area of Impervious Surface (%)	6.44
Area of USDA NRC HSG A (%)	1.5
Area of USDA NRC HSG C (%)	39.8
Area of USDA NRC HSG D (%)	7.3
Mean Annual Precipitation, PREC (inches)	46.1
Mean Annual Temperature, TEMP (^oF)	51.1
Mean Annual Snowfall, SNOW (in/yr)	42.3
Mean Annual Lake Evaporation, E_p (in/yr)	45.7
Mean Annual Ref. Evapotranspiration, E_t (in/yr)	35.8
Mean Watershed Elevation, ELEV (feet msl)	218
Main Stream Channel Slope, C-SLOPE (ft/mile)	17.8
Watershed Aspect relative to true north (degrees)	179.3

7.4 Stafford Pond Watershed Bathymetry and Stage-Storage-Area Relationship.

Figure 7.4.1.1 below is a bathymetry map of Stafford Pond. Table 7.4.1.1 which follows is the Stage-Storage-Area data for Stafford Pond derived from analysis of the bathymetry data. The normal pool is at elevation 205 ft (msl). The invert of the raw water intake is located at elevation 185 ft msl. Because design drawings are not available, HYSR assumes that to avoid pump intake vortexing, the minimum operating elevation should be approximately 3 ft above the intake invert (188 ft msl). The active storage, which lies between 188 and 205 ft (msl), is the range of the system can operated without overtopping the impoundment or damaging the pumps. This data is also shown graphically in Figure 7.4.1.2 below.

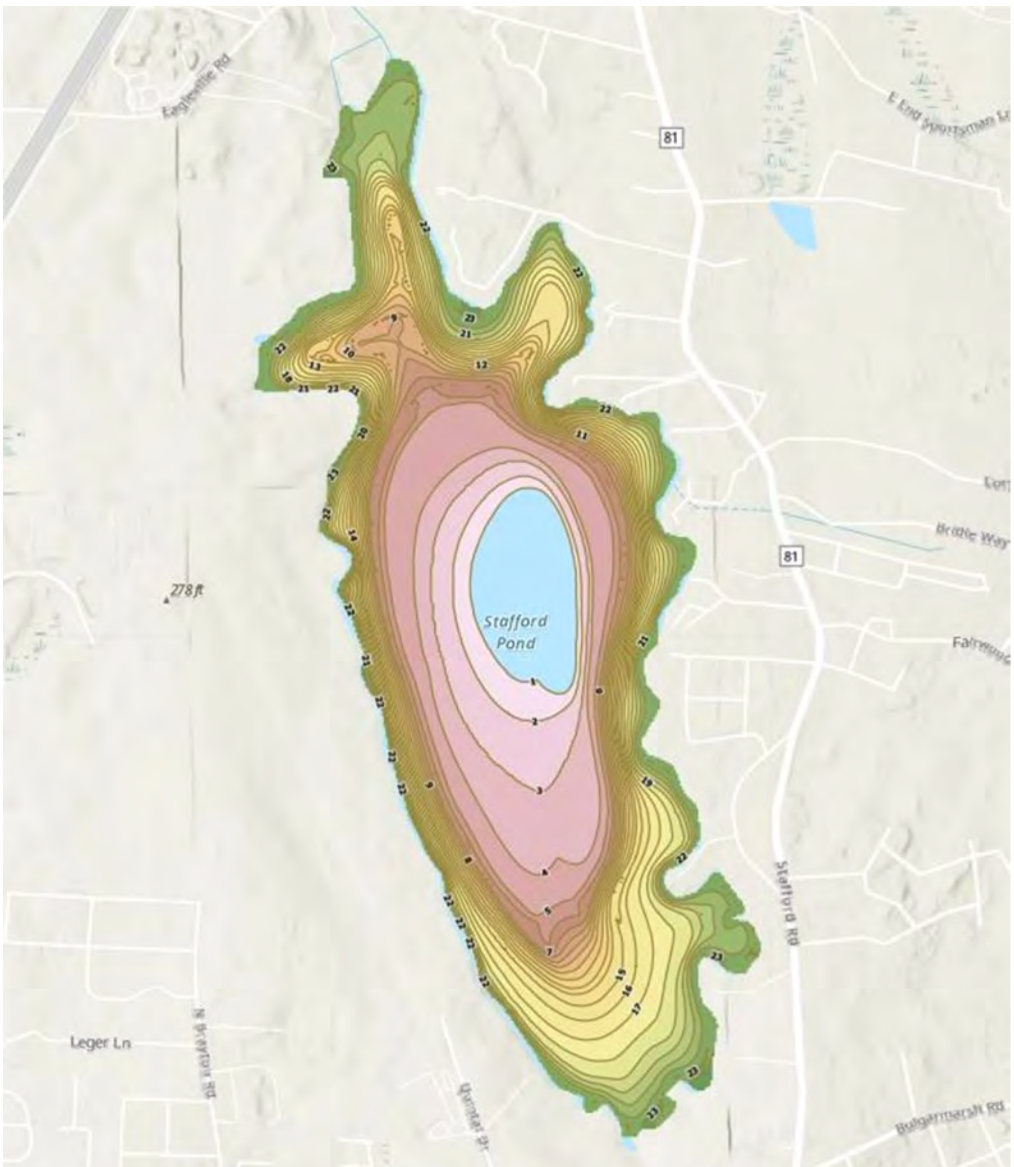

Figure 7.4.1.1 Stafford Pond Bathymetry

Table 7.4.1.1
Stafford Pond, Tiverton, RI
Stage, Storage, Area

Stage (ft msl)	Area (acres)	Storage Vol (acre-ft)	Comment
182	0.0	0.0	
183	33.5	16.7	
184	79.0	72.9	
185	118.1	171.5	Raw Water Intake
186	167.4	314.2	
187	184.0	489.9	
188	204.5	684.2	Min. Intake Submergence
189	214.3	893.6	
190	224.1	1112.7	
191	237.3	1343.5	
192	247.3	1585.8	
193	258.8	1838.9	
194	271.9	2104.2	
195	281.9	2381.1	
196	296.5	2670.3	
197	316.0	2976.5	
198	337.3	3303.2	
199	359.8	3651.7	
200	381.8	4022.5	
201	399.4	4413.2	Stop-Log Gate Invert
202	418.3	4822.1	
203	443.8	5253.2	
204	485.2	5717.7	
205	488.4	6204.4	Normal Pool

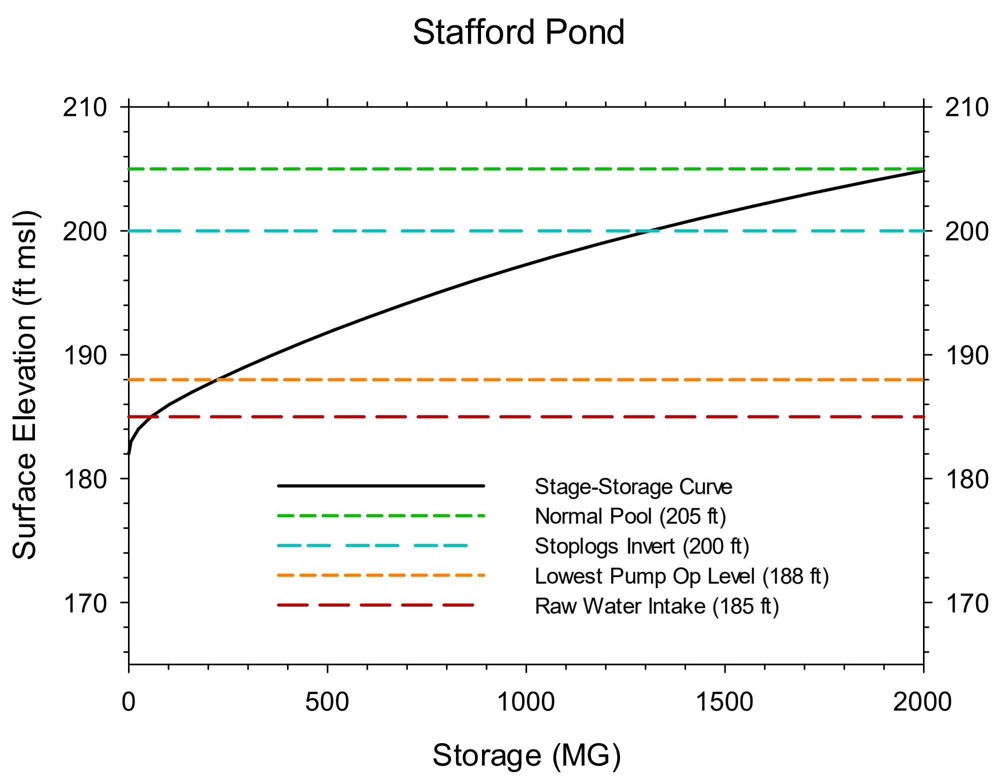

Figure 7.4.1.2 Stafford Pond Stage-Storage Curve

7.5 Stafford Pond Hydraulics

The Stafford Pond system hydraulics are summarized below in Table 7.5.1.

Table 7.5.1
Stafford Pond, Tiverton, RI
System Hydraulics

Normal Pool (Weir) Elevation (feet msl)	205
Flood Pool Elevation (feet msl)	206 est.
Emergency Spillway Invert (feet msl)	201
Normal Pool Storage (MG)	2022
Normal Pool Surface Area (acres)	488
Main Spillway Type	Reinforced Concrete
Main Spillway Length (feet)	35
Primary Outlet	Stop-Log Gate
Primary Outlet Invert (feet msl)	201
Primary Outlet Dimensions (feet)	5 wide x 4 deep
Raw Water Intake Invert Elevation (feet msl)	185
Assumed Raw Water Intake Minimum Submergence Elevation (feet msl)	188

8.1 Introduction to the Study Systems Firm Yield Assessments

Firm Yield was defined and extensively discussed in Chapter 2 of this report. The focus here is to summarize the firm yields of the Fall River, Somerset and Tiverton surface water supply systems. Using the methods described by Fennessey (1994), the estimation process involves solving the Mass Balance Equation day by day over the course of a period-of-record from 1940-1980. Equation 8.1.1 is the model of the Mass Balance Equation employed in this study.

$$\begin{aligned} S(j+1) = S(j) &+ A_r(j)\left[P(j) - E_p(j)\right] + A_w(j)\left[Q_{si}(j)\right] + Q_T(j) \\ &- \left[Q_{so}(j) + \alpha(mon)Q_w(j) + \alpha(mon)Q_{ow}(j) + Q_r(j)\right] \end{aligned} \quad (8.1.1)$$

where S(J+1) is the is the instantaneous volume of water in active storage and the end of the day J; S(j) is the volume of water in storage at the beginning of day j, which also equals the volume of water in storage at the end of the previous day; $A_r(j)$ is the area of the lake surface; $A_w(j)$ is the area of the contributing watershed; P(j) is the total depth of water on that fell on $A_r(t)$; $E_p(j)$ is the total depth of water evaporated from $A_r(t)$; $Q_T(j)$ is the total volume of water transferred from another source; $Q_{si}(j)$ is the total volume of surface (streamflow) water per unit area that flowed from $A_w(t)$; $Q_{so}(t)$ is the volume of uncontrolled spillway discharge rate; $Q_w(j)$ is the total volume of water to determine the Firm Yield withdrawn; α(mon) is the particular water supplier's dimensionless monthly water use factor and $Q_{ow}(j)$ is the total volume of water to withdrawn by others or for some other purpose that water supply system being examined.

An earlier chapter in this report describes how the daily P and E_p time series were constructed. Other chapters describe how the quantitative relationship between S and A_r was established. Those same chapters describe how $A_w(j)$ is determined from $A_r(j)$ and summarize the specific systems hydraulics necessary to estimate $Q_r(j)$.

The solution of Equation 8.1.1 requires an initial condition, which would generally be the volume of water in storage at the beginning of the very first day of the simulation: S(1). Because both Fall River and Somerset operate their systems in part based on how full their reservoirs are, they express that in terms of elevation. In the case of Fall River, the normal pool, referred to by the operators as "Full Pond" with a "local elevation" = 0 is 132 ft (msl). Somerset seems to operate their system in terms of Percent Full which they easily convert to feet (msl) by way of their vintage bathymetry map. Given this preference, HYSR constructed each system's firm yield model using reservoir elevation(s) as the initial condition. Because of the relationship between elevation and storage, also discussed during an earlier chapter in this report, this was not difficult to implement.

Because P(j), E(j) and $Q_{si}(j)$ need to be provided to solve Equation 8.1.1, the starting point of the simulation was dictated by the available data from the USGS Adamsville Brook streamgage station, which began Oct 1,

1940. To match this condition, all the reservoir model input time series begin on Oct 1, 1941. They end on Sept. 30, 1980. Although this would seem unusual given our general dependence on the Gergorian calendar, from a hydrological standpoint this makes a great deal of sense. The reason being that early streamflow gaging was initially for water supply assessment and flood assessment. Because the lowest flows generally occur in the early fall, Oct 1 is when the USGS Water Year begins. For example, the 1941 Water Year began Oct 1, 1940. The 1979 Water Year ended on Sept. 30, 1979. Although the Base Climate for this study is established as 1950-1980, the simulations were started in 1940 to allow the consequences of the initial condition to vanish by 1950. Earlier simulations with a monthly time-step firm yield model determined that it does.

8.2 Fall River Surface Water Supply System Firm Yield

As described earlier, the Fall River system consists of North Watuppa Pond, South Watuppa Pond and the Copicut Reservoir. The primary source is South Watuppa Pond, where is where the Water Filtration Plant is located. Copicut Reservoir is a secondary source, located in a neighboring watershed, and due to the expense of pumping, is presently used as a stand-by source. In other words, if South Watuppa Pond is drawn down to a threshold which is seasonally dependent, the two 3 mgd pumps are brought on-line and the raw water is discharged to Blossom Brook at a location that is a few thousand feed east of North Watuppa Pond. Some of that transferred water is lost due to summertime evapotranspiration but how much of it has never been quantified. For the present study, HYSR assumes that all 6 mgd of Copicut raw water is added to the North Watuppa storage. South Watuppa Pond is operated only on an emergency basis, otherwise except in the case of an extreme flood in North Watuppa Pond, the Narrows Gatehouse vertical gates remain closed. HYSR assumes that in the event of a repeat of past practice of placing bags of copper sulfate in the narrows Gatehouse channel, the operators will allow at most a 5 mgd from South Watuppa to "adequately" treat the water and not blow the bags into North Watuppa due to excessive hydraulic forces.

Figure 8.2.1 is a flow chart of the Fall River system operating rules. HYSR created it to assist with writing the Firm Yield model computer code for Fall River and to provide a visual display of the rules for the Filtration Plant Operator's reference.

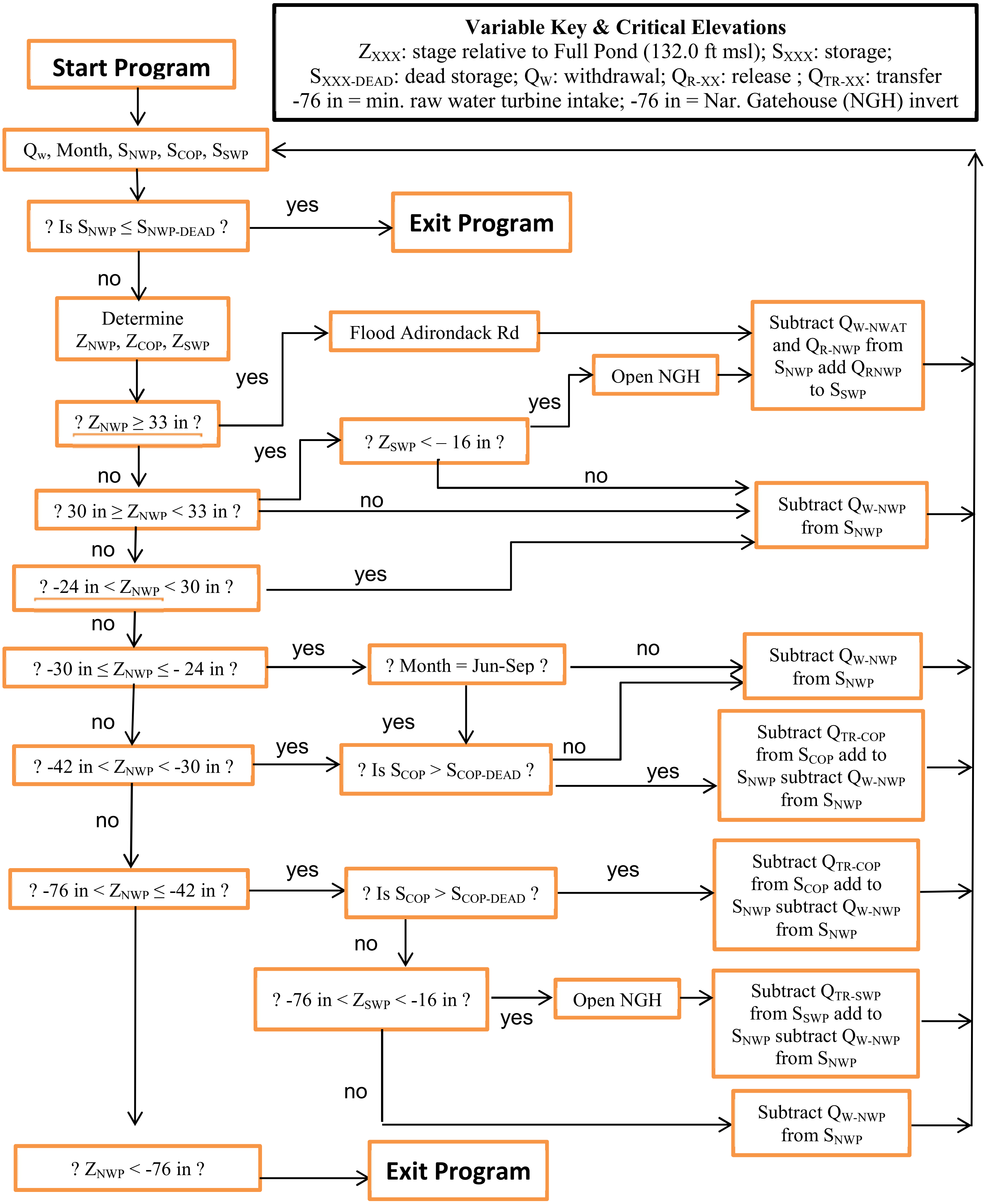

Figure 8.2.1. North Watuppa Pond Operating Rules Flow Chart.

The initial elevation and boundary conditions used for the Fall River system reservoirs are shown in Table 8.2.1. All three were given a somewhat drawn down initial condition, as one would tend to expect in October. The Normal Pool elevation and Minimum Pool elevation are given as well. When the reservoir or pond surface falls below the minimum elevation, the active storage has been fully depleted and the Firm Yield model's system no longer has access.

Table 8.2.1
Initial and Boundary Conditions for the Fall River Reservoir System

Source	Initial Elevation (ft-msl)	Full Pond Elevation (ft msl)	Minimum Elevation (ft-msl)
North Watuppa Pond	131.0	132.0	125.67
South Watuppa Pond	130.5	130.5	125.67
Copicut Reservoir	140.0	142.0	123.0

Table 8.2.3 summarizes the results of the Firm Yield assessment of the Fall River surface water supply system. The system was operated according to the rules as shown in Figure 8.2.1. No independent withdrawals were made from either South Watuppa Pond or Copicut Reservoir. Those scenarios will be discussed in a subsequent chapter of the report.

Table 8.2.3
Estimated Firm Yield of the Fall River Reservoir System

Sources	Firm Yield (mgd)
North Watuppa Pond, South Watuppa Pond and the Copicut Reservoir	11.0

8.3 Somerset Surface Water Supply System Firm Yield

As described earlier, the Town of Somerset's surface water supply system consists of the Somerset Reservoir and a pumping station located on the Segreganset River in Dighton. The Somerset Water Filtration plant is located at the toe of the Somerset Reservoir. The Segreganset River pumping station, which is operated on a seasonal basis, is located about a mile north of the reservoir. The pumping station has two 6 mgd pumps but due to the condition of the pipeline, the system Superintendent reports that at this time, only one pump is used. The Firm Yield is estimated for a single 6 mgd pump and both pumps. In addition to the seasonal rules, Somerset does not run the pumps until flows at the USGS Segreganset River streamgage exceed 15 cfs.

The initial elevation and boundary conditions used for the Somerset Reservoir are shown as Table 8.3.1. It was given a somewhat drawn down initial condition, as one would tend to expect in October. The Normal Pool elevation and Minimum Pool elevation are given as well. When the reservoir or pond surface falls below the minimum elevation, the active storage has been fully depleted and the Firm Yield model's system no longer has access.

Table 8.3.1
Initial and Boundary Conditions for the Somerset Reservoir

Source	Initial Elevation (ft-msl)	Normal Pool Elevation (ft msl)	Minimum Elevation (ft-msl)
Somerset Reservoir	50.0	56.0	35.0

Table 8.3.2 summarizes the results of the Firm Yield assessment of the Somerset Reservoir surface water supply system. Figure 8.3.1 below is the Operating Rules flow chart for the Somerset system. The Firm Yield model was operated according to the rules and no independent withdrawals were made by other users.

Table 8.3.2
Estimated Firm Yield of the Somerset Reservoir

Sources	Firm Yield One 6 mgd Pump (mgd)	Firm Yield Two 6 mgd Pumps (mgd)
Somerset Reservoir and the Segregansett River	2.54	4.88

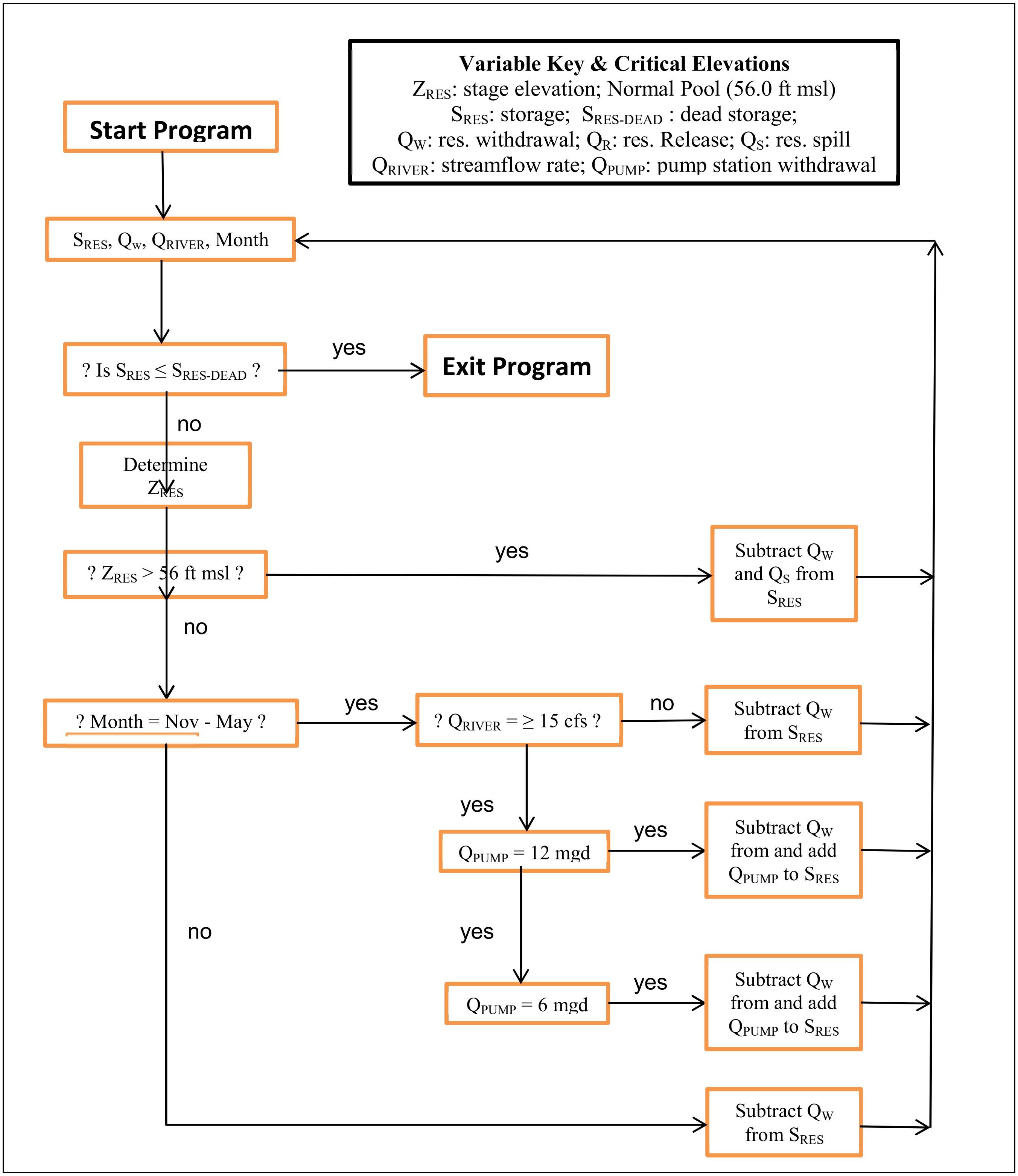

Figure 8.3.1 Somerset Reservoir Operating Rules Flow Chart.

8.4 Stone Bridge Fire District Stafford Pond Firm Yield

As described earlier, the Town of Tiverton's Stone Bridge Fire District's sole-source surface water supply system consists of Stafford Pond. The Stone Bridge Fire District's water filtration plant is located on the western side of Stafford Pond. The system is very simple and doesn't require an Operating Rules flow chart. The initial elevation and boundary conditions used for Stafford Pond are shown as Table 8.4.1. It was given a somewhat drawn down initial condition, as one would tend to expect in October. The Normal Pool elevation and Minimum Pool elevation are given as well. When the pond surface falls below the minimum elevation, the active storage has been fully depleted and the Firm Yield model's system no longer has access.

Table 8.4.1
Initial and Boundary Conditions for Stafford Pond

Source	Initial Elevation (ft-msl)	Full Pond Elevation (ft msl)	Minimum Elevation (ft-msl)
Stafford Pond	203.0	205.0	188.0

Table 8.4.2 summarizes the results of the Firm Yield assessment of Stafford Pond surface water supply system.

Table 8.4
Estimated Firm Yield of Stafford Pond

Source	Intake Depth (ft)	Firm Yield (mgd)
Stafford Pond	20	1.59

8.5 References

Fennessey, N.M., *Estimating the Firm Yield of a Surface Water Reservoir Supply System in Massachusetts: a Guidance Manual*, Prepared for the Massachusetts Department of Environmental Protection, UMass-Dartmouth Department of Civil and Environmental Engineering, Hydrology and Water Resources Group Publication January 1996.

9.1 Introduction to Impacts of Climate Change on Firm Yield

A key focus of the present study is to explore the potential impact of climate change on the Firm Yield of the water supply reservoirs of Fall River, Somerset and Tiverton. As discussed in depth elsewhere, key mass balance fluxes include precipitation on and evaporation from the reservoir surface as well as inflows from tributary streams. Reservoir surface precipitation and evaporation are modeled as estimates of decades of daily processes. Estimates of historic direct daily precipitation are used in the Firm Yield modeling process. As discussed earlier, decades of historic daily air temperature are used to develop estimates of mean monthly air temperature for that period-of-record. These in turn are used to develop twelve estimates of mean monthly free surface evaporation. The twelve monthly estimates of evaporation are then transformed to fixed 365/366 days-per-year cycle of estimated daily evaporation. In other words, the estimated reservoir evaporation rate that occurs on July 17 for each year of the base period is the same year in and year out. Decades of estimated historic daily inflows are developed using the QPPQ Transform whose parameters include mean annual temperature and mean annual precipitation.

In order to explore the potential impacts due to climate change, these estimated historic daily reservoir inflows and streamflows, historic daily precipitation, precipitation and reservoir surface evaporation from the 1950-1980 Base Period, must be re-scaled by climate change projections. This is accomplished by using results from General Circulation Models or GCMs. A GCM is a mathematical, physics-based representation of the atmosphere, land and oceans of planet Earth on a computer platform.

Fennessey and Kirshen (1994) and Kirshen and Fennessey (1995) conducted a potential climate change impact Firm Yield assessment of the MWRA water supply system of greater Boston. They used climate change projections due to doubling the atmospheric carbon dioxide (2xCO_2) as generated by the first generation of GCMs. Climate modeling detail, modeling techniques and digital computing power have evolved enormously over the past 30 years. The present study is using the results from five different GCMs to re-scale the Firm Yield model variables mentioned above. The goal is to develop a range of system firm yield solutions for these three water supply systems to gain an understanding as to how sensitive the Firm Yield is to climate change due to Green House Emissions (GHGs).

9.2 Introduction to General Circulation Models

A major focus of the present study is to perform an assessment of how the surface water ponds and reservoirs of Fall River, Somerset and Tiverton might be impacted due to climate change. There is strong evidence that global warming is occurring and the scientific community has reached a majority consensus about this. To investigate potential impacts, researchers and scientists use the results from various GCMs which are computer-based mathematical models of dynamic (time varying) global-scale physical processes. GCMs were originally developed from large-scale atmospheric weather-forecasting models which first incorporated land surface features and then later ocean circulation models. The type of GCM used for this study is a coupled atmosphere-land-ocean circulation model because each component dynamically influences the others.

The GCM Earth is modeled as something akin to a multi-layer cake with toothpicks penetrating the many model layers at a horizontal spacing of 1 degree latitude and a vertical spacing of 1 degree longitude. A grid-square this size at the equator would be a rectangle of 70 statute miles by 70 miles (4,900 mi^2 = 13,000 km^2). Other GCMs might employ grid squares of 2 degrees x 2 degrees or 1 degree latitude x 2 degrees longitude. Some have different spacing for the atmosphere, land and ocean grids. A GCM model atmosphere might consist of 30 layers, the ocean might have 20 layers and the Earth's surface and subsurface might have 10 layers.

Mathematical equations from physics are solved where each toothpick penetrates a cake layer. For a single grid "cube" these equations are solved at each of the eight corners once every 15 minutes which is an enormous computational task. The first generation of GCMs with 9 atmospheric layers, 2 land layers and a single ocean layer with 4 deg x 5 deg grid squares took a full year for a 100-year 2xCO2 simulation on a Cray Supercomputer. Today's GCMs not only have smaller grid squares, as mentioned, they also have dozens of layers and run on massive parallel processing computers.

The collection of stacked grids which comprise a GCM is shown in Figure 9.2.1. Figure 9.2.1 also shows the physical processes of the GCM models, including the vertical and lateral transfer of heat and water mass, atmospheric divergence and convergence, wind patterns, ocean circulation, among others.

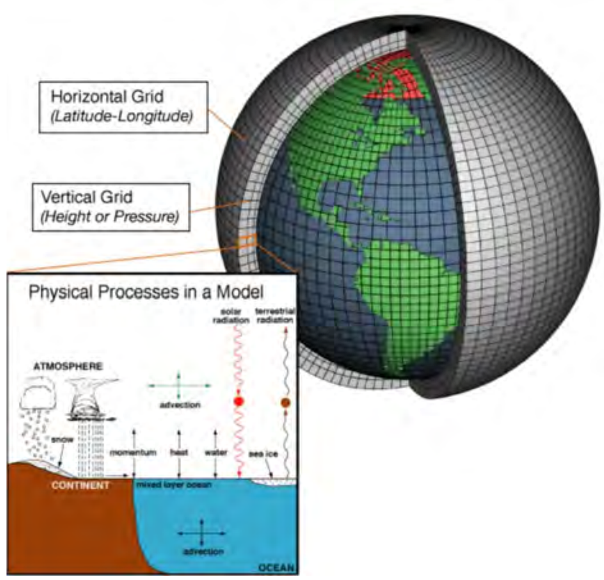

Figure 9.2.1 Conceptual Global Circulation Model

9.3. GCM Models Used

HYSR is using data generated by five GCMs that were designed to investigate climate change by following the growth rate of greenhouse gases (GHCs: carbon dioxide, methane, etc.) over time subject to various emission scenarios. ChatGTP AI (Artifical Intellegence) was used to select the "best" seven GCMs to use for a water supply study in New England. Daily air temperature and precipitation data for the period of 1950-2100 were provided to HYSR by the Lawrence Livermore Laboratory. It was found that two of the seven either had missing data or didn't have the data needed. The five GCMs used and some information about each one are listed below in Table 9.3.1.

Table 9.3.1
GCM Simulation Models Used

ID	GCM Run	Host/Developer
GCM 1	**CanESM5 ssp245** **CanESM5 ssp585**	Canadian Centre for Climate Modeling and Analysis is located in Toronto, Canada. https://www.canada.ca/en/environment-climate-change/services/climate-change/science-research-data/modeling-projections-analysis/centre-modelling-analysis.html
GCM 2	**CNRM-CM6-1 ssp245** **CNRM-CM6-1 ssp585**	CNRM-CERFACS is located in Toulouse, France. Specifically, it is a joint effort between Centre National de Recherches Météorologiques (CNRM) and Centre Europeen de Recherche et de Formation Avancée en Calcul Scientifique (CERFACS). https://www.umr-cnrm.fr/cmip6/spip.php?article11
GCM 3	**EC-Earth3 ssp245** **EC-Earth3 ssp585**	EC Earth Consortium is a collaborative European community Earth System Model with key members from across Europe including Sweden, the Netherlands, Denmark, Spain, Ireland, Italy and Finland. The model is hosted at the Swedish Meteorological and Hydrological Institute in Norrköping, Sweden. **https://ec-earth.org/**
GCM 4	**GFDL-ESM4 ssp245** **GFDL-ESM4 ssp585**	The Geo Fluid Dynamics Laboratory (GFDL), which is part of NOAA, is located in Princeton, New Jersey USA https://www.gfdl.noaa.gov/
GCM 5	**NorESM5 ssp245** **NorESM5 ssp585**	Norwegian Centre for Climate Services (NCCS). The main partners are the NCCS are the Norwegian Meteorological Institute (MET) and the Norwegian Research Centre (NORCE), located in Norway. https://klimaservicesenter.no/kss/om-oss/nccs?locale=en

9.4. CMIP6 GCM Simulation Data

HYSR used GCM simulation results from the 6th phase of the Coupled Model Intercomparison Project (CMIP6). HYSR used CMIP6 data for two emission scenarios: ssp245 and ssp585 respectively precipitation and temperature data where ssp stands for "Shared Socio-economic Pathway." ssp245 is a "moderate" GHC emission scenario and ssp585 is an unchecked GHC emission scenario. The ssp245 scenario assumes that GHCs peak around 2040 and then decline thanks largely to global cooperation among fossil fuel users. The ssp585 recognizes the possibility that GHC-emissions scenario will continue to rise throughout the 21st century due to increased economic competition among fossil fuel users.

Figure 9.4.1 compares the GCM model near surface Earth's temperature that might result from alternative emission scenarios. As this figure suggests, some of the GCMs' models begin their simulation in the early 19th century, which is when the Industrial Revolution began and the extensive use of fossil fuels (coal) began in earnest. The piControl black line is a simulation using estimated values of the various pre-Industrial Revolution GHC concentrations, including carbon dioxide (CO_2) and methane (CH_4), among others.

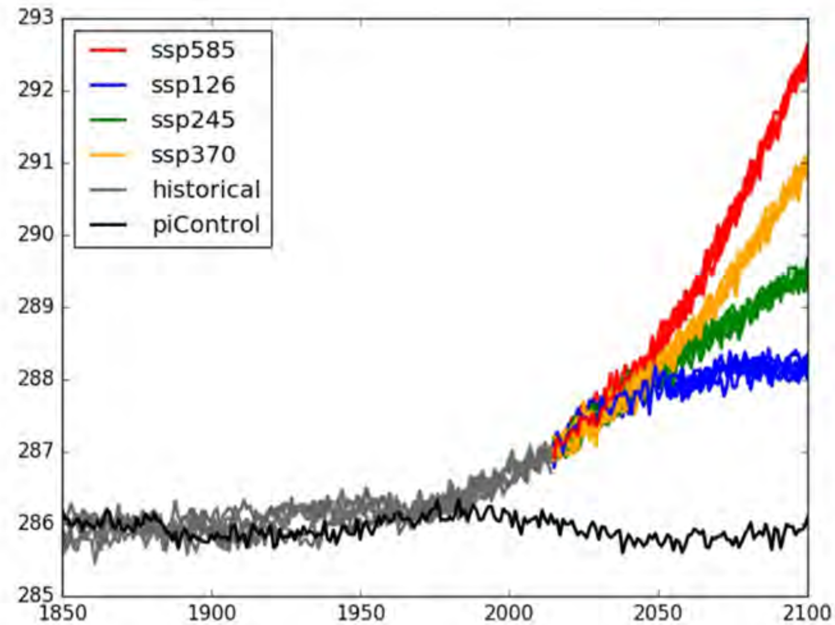

Figure 9.4.1 CMIP6 CO_2 Concentrations from Gas Emission Scenarios

9.5 Downscaling GCM Simulation Results

Of particular interest for the present study is precipitation (as rainfall) and air temperature near the land surface of the earth (2 meters) near Fall River. The process HYSR is using is called "down-scaling." Consider that the total area of Massachusetts is about 10,500 miles. It's about twice as "wide" as it "tall" or roughly 140 miles x 70 miles (9800 mi^2). A single 2 deg x 1 deg GCM grid-square would contain the entire state but everyone knows that the landscape of the Berkshires is very different from that of Cape Cod. Because the study focus is on Fall River area water supply reservoirs, surface interpolation is necessary. Fortunately, one may specify a near-specific location that is an interpolation of precipitation and surface temperature data (citation needed) from the large grid square and its neighbors as appropriate to a 1/16 deg x 1/16 deg. area which would be half the size of the smallest red square box that's shown in Figure 9.5.1

Figure 9.5.1 1/8 deg x 1/8 deg Grid-Square Centered on North Watuppa Pond

9.6. Downscaled GCM Data

The Base Period of the present study has been chosen to be January 1, 1950 - December 31, 1980 for reasons discussed earlier. Statistics made from the study area of estimated daily precipitation and air temperature daily data are compared with statistics generated by the 5 GCMs for the same period of time, using GHC concentrations from that same period. HYSR has chosen the period of January 1, 2025 - December 31, 2055 as the mid-century period-of-record, because 2040 is when the positive impacts of ssp245 reduced emissions were assumed to begin to take effect. The late-century period-of-record has been chosen to be January 1, 2070 - December 31, 2100. Note that all of CMIP6 simulations end on December 31, 2100.

Table 9.6.1 lists the ssp245 mean annual daily temperature (^oF) for Fall River, GCM base period, mid-century and late century. Similarly, Table 9.6.2 lists the ssp245 mean annual precipitation (in/year) for Fall River, GCM base period, mid-century and late century.

Table 9.6.1
ssp245
Mean Annual Temperature (^oF) by GCM

Period	Obs	GCM 1	GCM 2	GCM 3	GCM 4	GCM 5
1950-1980	73.1	71.2	72.0	71.1	72.0	71.7
2025-2055	-	77.7	75.7	76.0	76.2	76.6
2070-2100	-	81.3	78.7	77.7	79.0	78.9

Table 9.6.2
ssp245
Mean Annual Precipitation (in/yr) by GCM

Period	Obs	GCM 1	GCM 2	GCM 3	GCM 4	GCM 5

1950-1980	46.1	46.8	46.3	46.4	48.7	47.1
2025-2055	-	49.2	50.2	49.6	51.9	50.3
2070-2100	-	53.5	52.2	53.0	54.3	53.9

Figures 9.6.1, 9.6.2 and 9.6.3 respectively show the July ssp245 mean monthly results for air temperature, precipitation and evaporation.

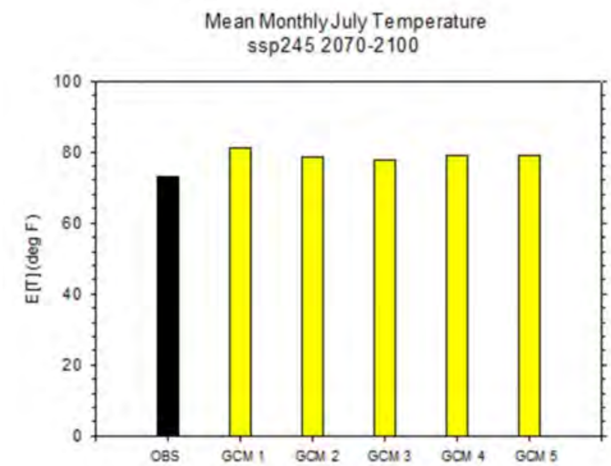

Figure 9.6.1 ssp245 July Mean Monthly Temperature

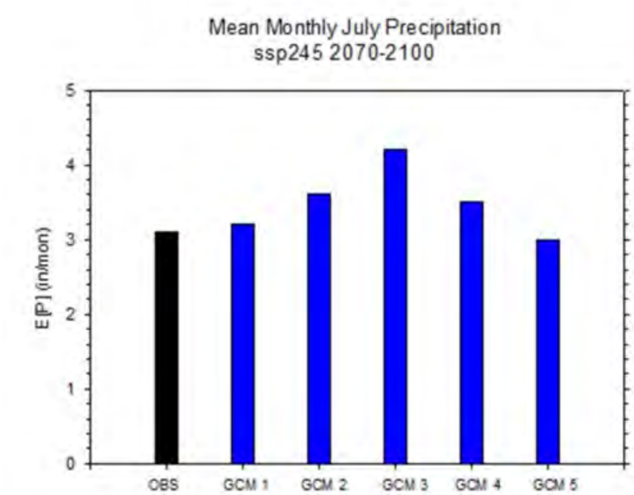

Figure 9.6.2 ssp245 July Mean Monthly Precipitation

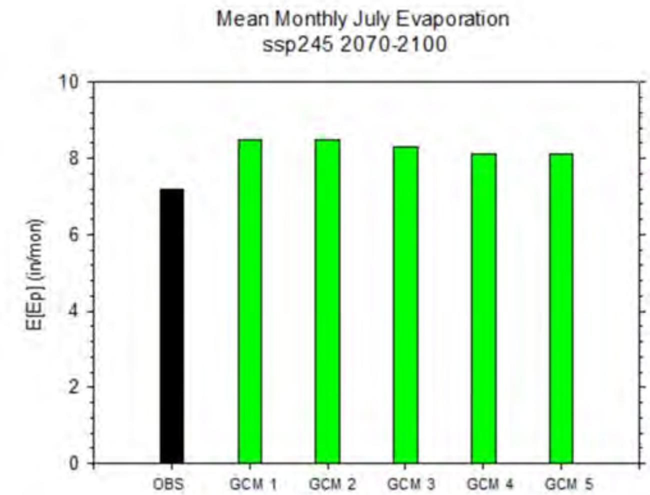

Figure 9.6.3 ssp245 July Mean Monthly Evaporation

Appendix II and Appendix III of this report provide the reader with many tables of monthly and annual mean values of temperature, precipitation and reservoir evaporation by time period (1950-80, 2025-55 and 2070-00), by GCM and by emission scenario (ssp245 and ssp858).

9.7 Scaling the Historic Baseline Climate Data.

In the above discussion, the estimates of 1950-1980 historical base-line daily precipitation and temperature data must be scaled to explore the potential impacts in estimated firm yield due to ssp245 and ssp585 GHC-induced climate change. Daily reservoir surface fluxes, P(j) and E_p(j) are perturbed by the multiplying each by a GCM specific ratio. For example, Equation 9.7.1 illustrates how this process is accomplished for an analysis involving GCM1, ssp245 and 2025-50:

$$P_{GCM1\,ssa245\text{-}2025\text{-}55}(\text{jday}) = \text{Pratio}(\text{month})P(\text{jday})$$

$$\text{where Pratio}(\text{month}) = \left[\frac{E\left[P_{GCM1\,ssp245\text{-}2055}(\text{month})\right]}{E\left[P_{GCM1\,1950\text{-}80}(\text{month})\right]}\right] \tag{9.7.1}$$

A similar approach is used to perturb the daily evaporation, as shown by Equation 9.7.2.

$$E_{p\ \ GCM1\,ssa245\text{-}2025\text{-}55}(\text{jday}) = E_p\text{ratio}(\text{month})P(\text{jday})$$

$$\text{where } E_p\text{ratio}(\text{month}) = \left[\frac{E\left[E_{p\ \ GCM1\,ssp245\text{-}2055}(\text{month})\right]}{E\left[E_{p\,GCM1\,1950\text{-}80}(\text{month})\right]}\right] \tag{9.7.2}$$

Scaling the daily reservoir surface inflows, Q_{si}(jday) is handled a different way. Because the QPPQ Transform method assumes that the probability function that best describes daily streamflow is the Generalized Pareto Distribution function, the GPA, as discussed earlier, has an explicit formula for the mean value, E[Q], as shown below by Equation 9.7.3.

$$E[Q] = \xi[E(P), E(T)] + \frac{\alpha[E(P)]}{\kappa[E(P), E(T)]}$$

(9.7.3)

where ξ, α and κ are the three GPA parameters. The regional equations for both ξ and κ require the annual mean values of precipitation and temperature, E(P) and E(T), as independent variables among others including watershed soil and topography, as discussed in an earlier chapter. This mean value is estimated for each GCM for each emission scenario and for each time period. Each daily value of Q_{si}(jday) is scaled according to Equation 9.7.4 using ratios of the mean value of Q, E(Q).

$$Q_{GCM\ ssp245\ 2025\text{-}55}(\mathrm{jday}) = Q(\mathrm{day}) \frac{E_{GCM\ ssp245\ 2025\text{-}55}(Q)}{E_{GCM\ 1950\text{-}80}(Q)} \qquad (9.7.4)$$

Appendix III lists the annual mean values, E(T) and E(T) for both emission scenarios, and 1950-80, 2025-55 and 2070-00 periods of time, for each of the five GCMs.

9.8 Results of the Firm Yield Assessment Under Climate Change

The Firm Yield Mass Balance model discussed earlier had the daily reservoir surface flux terms, P(jday) and E_p(jday) scaled by the ratio of the monthly mean values of P, and T and the daily inflows scaled according to Equation 9.7.4 as discussed above. The results are shown in Tables 9.8.1 , 9.8.2 and 9.8.3. In each case, the Firm Yield is estimated using the current operating rules and equipment conditions. For example, the Somerset Reservoir assessment uses only one 6 mgd pump to transfer water from the Segreganset River.

Table 9.8.1
Fall River Surface Water Source Firm Yield Under Climate Change

Emission Scenario	Historic (mgd)	GCM 1 (mgd)	GCM 2 (mgd)	GCM 3 (mgd)	GCM 4 (mgd)	GCM 5 (mgd)	GCM Ave (mgd)
1950-80	11.0	11.1	11.0	11.1	11.1	11.1	11.1
ssp245 2025-55	-	11.0	11.4	11.1	11.3	11.3	11.2
ssp585 2025-55	-	8.8	11.0	10.2	11.1	11.1	10.4
ssp245 2070-00	-	11.4	9.8	11.1	11.3	9.3	10.6
ssp585 2070-00	-	10.0	11.0	11.1	11.3	11.2	10.9

Table 9.8.2
Somerset Reservoir Firm Yield Under Climate Change

Emission Scenario	Historic (mgd)	GCM 1 (mgd)	GCM 2 (mgd)	GCM 3 (mgd)	GCM 4 (mgd)	GCM 5 (mgd)	GCM Ave (mgd)
1950-80	2.53	2.56	2.53	2.55	2.56	2.56	2.55
ssp245 2025-55	-	2.48	2.69	2.51	2.58	2.54	2.58
ssp585 2025-55	-	2.53	2.49	2.45	2.54	2.54	2.47
ssp245 2070-00	-	2.60	2.44	2.51	2.57	2.42	2.51
ssp585 2070-00	-	2.41	2.46	2.48	2.55	2.54	2.49

Table 9.8.3
Stafford Pond Firm Yield Under Climate Change

Emission Scenario	Historic (mgd)	GCM 1 (mgd)	GCM 2 (mgd)	GCM 3 (mgd)	GCM 4 (mgd)	GCM 5 (mgd)	GCM Ave (mgd)
1950-80	1.59	1.63	1.58	1.62	1.65	1.61	1.62
ssp245 2025-55	-	1.57	1.80	1.62	1.68	1.65	1.66
ssp585 2025-55	-	1.30	1.64	1.51	1.57	1.66	1.54
ssp245 2070-00	-	1.81	1.57	1.65	1.73	1.40	1.63
ssp585 2070-00	-	1.47	1.60	1.63	1.69	1.69	1.62

The results are somewhat surprising. The estimated changed climate Firm Yields didn't change much at all. In several instances the firm yield increased. This is likely due to the process being dominated more by the increased mean annual streamflow in which its sensitivity is greater to increased precipitation than increased air temperature.

9.9 References

Fennessey, N.M. and P.H. Kirshen, 1994, *Evaporation and Evapotranspiration Under Climate Change in New England*, ASCE J. of Water Resources Planning and Management, Vol. 120, No. 1, pp. 48-70.

Kirshen, P.H. and N.M. Fennessey, 1995, *Possible Climate-change Impacts on Water Supply of Metropolitan Boston*, ASCE J. of Water Resources Planning and Management, Vol. 121, No. 1, pp. 61-70.

10.1 Introduction to Copicut Reservoir and South Watuppa Pond Alternative Operating Rules Firm Yield Assessment

In a report written in 1958, Whitman & Howard described an analysis of the water supply requirements of Somerset, Tiverton and Dartmouth in light of the proposed Copicut Reservoir. Water demand in Fall River by that point was about 10.0 mgd with a maximum day demand of 14.78 mgd. Fall River had agreed to provide Somerset with 0.6 to 1.2 mgd depending on the water level in North Watuppa Pond. With Somerset's well field deteriorating, Whitman & Howard expected that Somerset would need 1.0 mgd in the near future. The North Tiverton Fire District would require 1.2 mgd from Fall River. At that time, the safe yield of North Watuppa Pond was estimated to be 8.5 mgd and that with commitments approaching 11.0 mgd, something needed to be done other than pump "highly colored water" from Lake Noquochoke into North Watuppa Pond.

Legislative Acts of 1924 gave Fall River the right to construct and maintain water works on Long Pond in Freetown. Long Pond is part of the Assawompset Pond Complex (APC), which is the source of water supply for Taunton and New Bedford. Whitman & Howard wrote that Long Pond is nine air-miles from Fall River and perhaps it might be best to relinquish those rights to Taunton and New Bedford, the latter providing treated water to the Town of Dartmouth. They wrote that it was six miles from the Copicut Reservoir site to Dartmouth plus another four miles to South Dartmouth. Given the cost of treatment, the cost to build a pipeline and pump water to Dartmouth, the result would be very expensive water and they recommended that Dartmouth continue to buy their water from the City of New Bedford. The Town of Dartmouth still does today. Ultimately, they recommended that the Copicut Reservoir be built.

Since the late 1950s, the cost of cost of fuel and electricity has changed significantly. Whitman & Howard anticipated that delivering 1.2 mgd from Copicut Reservoir would cost the City approximately $3,000 per year. HYSR was informed by the City that the monthly electricity bill for operating the Copicut Reservoir pumping station full-time was $30,000 per month. Add to this, an interest by Somerset, Tiverton, Dartmouth, Swansea and Westport, it might be time to take another look.

Given the historic operation of Copicut Reservoir as a stand-by source of raw water to be pumped to North Watuppa Pond during extended dry periods, HYSR suggested examining alternative roles and operating rules. HYSR examined the Copicut Reservoir as a potential independent source of raw water for the Towns of Dartmouth and/or Westport. One advantage of operating the Copicut Reservoir this way is to avoid the need to apply for an for an Interbasin-Transfer Permit, which might trigger unanticipated requirements, such as requiring releases, for example, to augment streamflow in the Copicut Rover. Fall River might also choose to consider building a treatment plant in the vicinity of the reservoir or Lake Noquochoke, adding value to the water sold.

As discussed in a prior section of this report, HYSR estimated that the Fall River water supply system firm yield was about 11.0 mgd. HYSR examined the impact on the Fall River Firm Yield as a consequence of providing water to other users using the Copicut Reservoir and or South Watuppa Pond as independent sources while maintaining the same North Watuppa Pond system operating rules. Those scenario results are shown below in various tables.

<u>10.2 Assessing the Firm Yield of the Fall River Water Supply System as-is and Build a New Intake for North Watuppa Pond</u>

Scenario 1) Present Rules: Copicut and South Watuppa used to augment North Watuppa. As a reminder, elevation 126.67 is the invert of the Narrows Gatehouse channels and the elevation of deposits at the entrance of the Rt. 24 box culvert off Brayton Avenue in Fall River. That culvert is an outlet of South Watuppa Pond and the headwater of the Quequechan River. Result: 11.0 mgd.

Table 10.2.1a
Initial and Boundary Conditions for the Fall River Reservoir System

Source	Initial Elevation (ft-msl)	Full Pond Elevation (ft-msl)	Minimum Elevation (ft-msl)	Maximum Drawdown (ft)
North Watuppa Pond	131.0	132.0	125.67	5.25
South Watuppa Pond	130.5	130.5	125.67	4.83
Copicut Reservoir	140.0	142.0	123.0	19.0

Table 10.2.1b
Estimated Firm Yield of the Fall River Reservoir System

Sources	Firm Yield (mgd)
North Watuppa Pond, South Watuppa Pond and the Copicut Reservoir	11.0

Scenario 2) Present Rules: Copicut and South Watuppa used to augment North Watuppa. Build a new intake structure in the northeast side of North Watuppa Pond and drop the intake invert to elevation 114.0 (msl) and specify submersible pumps. Result: the Firm Yield rises to 13.0 mgd.

Table 10.2.2a
Initial and Boundary Conditions for the Fall River Reservoir System

Source	Initial Elevation (ft-msl)	Full Pond Elevation (ft-msl)	Minimum Elevation (ft-msl)	Maximum Drawdown (ft)
North Watuppa Pond	131.0	132.0	114.0	18.0
South Watuppa Pond	130.5	130.5	122.5	4.83
Copicut Reservoir	140.0	142.0	123.0	19.0

Table 10.2.2b
Estimated Firm Yield of the Fall River Reservoir System

Sources	Firm Yield (mgd)
North Watuppa Pond, South Watuppa Pond and the Copicut Reservoir	13.0

10.3 Assessing the Firm Yield of the Fall River Water Supply System as-is and Use South Watuppa Pond as a Separate Source

Scenario 1) Allow an independent 4.0 mgd withdrawal from South Watuppa Pond. Use present Operating Rules: Copicut and South Watuppa used to augment North Watuppa. Result: the original system Firm Yield remains the same at 11.0 mgd but the combined Firm Yield, assuming that the City of Fall River is treating and distributing from both sources, rises to 15.0 mgd.

Table 10.3.1a
Initial and Boundary Conditions for the Fall River Reservoir System

Source	Initial Elevation (ft-msl)	Full Pond Elevation (ft-msl)	Minimum Elevation (ft-msl)	Maximum Drawdown (ft)
North Watuppa Pond	131.0	132.0	125.67	5.25
South Watuppa Pond	130.5	130.5	125.67	4.83
Copicut Reservoir	140.0	142.0	123.0	19.0

Table 10.3.1b
Estimated Firm Yield of the Fall River Reservoir System

Sources	Firm Yield (mgd)
North Watuppa Pond, South Watuppa Pond and the Copicut Reservoir	11.0
Independent South Watuppa Pond	4.0
Combined Firm Yield	15.0

Scenario 2) Allow an independent 7.0 mgd withdrawal from South Watuppa Pond. Use present Operating Rules: Copicut and South Watuppa used to augment North Watuppa. Result: the original system Firm Yield remains the same at 11.0 mgd but the combined Firm Yield, assuming that the City of Fall River is treating and distributing both sources, rises to 18.0 mgd.

Table 10.3.2a
Initial and Boundary Conditions for the Fall River Reservoir System

Source	Initial Elevation (ft-msl)	Full Pond Elevation (ft-msl)	Minimum Elevation (ft-msl)	Maximum Drawdown (ft)
North Watuppa Pond	131.0	132.0	125.67	5.25
South Watuppa Pond	130.5	130.5	125.67	4.83
Copicut Reservoir	140.0	142.0	123.0	19.0

Table 10.3.2b
Estimated Firm Yield of the Fall River Reservoir System

Sources	Firm Yield (mgd)
North Watuppa Pond, South Watuppa Pond and the Copicut Reservoir	11.0
Independent South Watuppa Pond	7.0
Combined Firm Yield	18.0

Scenario 3) Present Rules: Copicut and South Watuppa used to augment North Watuppa. An independent withdrawal of 9.5 mgd is made from South Watuppa. Result: the original system Firm Yield falls slightly to 10.6 mgd but the combined Firm Yield, assuming that the City of Fall River is treating and distributing both sources, rises to 20.1 mgd.

Table 10.3.3a
Initial and Boundary Conditions for the Fall River Reservoir System

Source	Initial Elevation (ft-msl)	Full Pond Elevation (ft-msl)	Minimum Elevation (ft-msl)	Maximum Drawdown (ft)
North Watuppa Pond	131.0	132.0	125.67	5.25
South Watuppa Pond	130.5	130.5	125.67	4.83
Copicut Reservoir	140.0	142.0	123.0	19.0

Table 10.3.3b
Estimated Firm Yield of the Fall River Reservoir System

Sources	Firm Yield (mgd)
North Watuppa Pond, South Watuppa Pond and the Copicut Reservoir	10.6
Independent South Watuppa Pond	9.5
Combined Firm Yield	20.1

10.4 Assessing the Firm Yield of the Fall River Water Supply System as-is and using Copicut Reservoir as Separate Source

Because Copicut Reservoir is located in the Buzzards Bay watershed, raw (or treated) water from this source could be sold to one or more communities also located in that watershed. In this case, an Inter-basin Transfer Permit application would not be necessary. Previous analysis has shown that using South Watuppa Pond as an independent source significantly enhances the Firm Yield of the original system of North Watuppa Pond being augmented by South Watuppa Pond and Copicut Reservoir as needed.

Scenario 1) Allow an independent 2.0 mgd withdrawal from Copicut Reservoir. Use present Operating Rules: Copicut and South Watuppa used to augment North Watuppa. Result: the original system Firm Yield falls to 7.5 mgd. The combined Firm Yield, assuming perhaps that the Copicut Reservoir source water is sold to someone in the Buzzards Bay watershed falls to 9.5 mgd but the City would have a significant shortfall for itself.

Table 10.4.1a
Initial and Boundary Conditions for the Fall River Reservoir System

Source	Initial Elevation (ft-msl)	Full Pond Elevation (ft-msl)	Minimum Elevation (ft-msl)	Maximum Drawdown (ft)
North Watuppa Pond	131.0	132.0	125.67	5.25
South Watuppa Pond	130.5	130.5	125.67	4.83
Copicut Reservoir	140.0	142.0	123.0	19.0

Table 10.4.1b
Estimated Firm Yield of the Fall River Reservoir System

Sources	Firm Yield (mgd)
North Watuppa Pond, South Watuppa Pond and the Copicut Reservoir	7.5
Independent Copicut Reservoir	2.0
Combined Firm Yield	9.5

Scenario 2) Allow an independent 4.0 mgd withdrawal from Copicut Reservoir. Use present Operating Rules: Copicut and South Watuppa used to augment North Watuppa. Result: the original system Firm Yield falls to 6.7 mgd. The combined Firm Yield, assuming perhaps that the Copicut Reservoir source water is sold to someone in the Buzzards Bay watershed falls to 10.7 mgd but the City would have a very significant shortfall for itself.

Table 10.4.2a
Initial and Boundary Conditions for the Fall River Reservoir System

Source	Initial Elevation (ft-msl)	Full Pond Elevation (ft-msl)	Minimum Elevation (ft-msl)	Maximum Drawdown (ft)
North Watuppa Pond	131.0	132.0	125.67	5.25
South Watuppa Pond	130.5	130.5	125.67	4.83
Copicut Reservoir	140.0	142.0	123.0	19.0

Table 10.4.2b
Estimated Firm Yield of the Fall River Reservoir System

Sources	Firm Yield (mgd)
North Watuppa Pond, South Watuppa Pond and the Copicut Reservoir	6.7
Independent Copicut Reservoir	4.0
Combined Firm Yield	10.7

<u>10.5 Assessing the Firm Yield of the Fall River Water Supply System as-is and using Copicut Reservoir and South Watuppa Pond as Separate Sources</u>

Because Copicut Reservoir is located in the Buzzards Bay watershed, raw (or treated) water from this source could be sold to one or more communities also located in that watershed. It this case, an Inter-basin Transfer Permit application would not be necessary. Previous analysis has shown that using South Watuppa Pond as an independent source significantly enhances the Firm Yield of the original system of North Watuppa Pond being augmented by South Watuppa Pond and Copicut Reservoir as needed.

Scenario 1) Given the supply shortfall for Fall River in Scenarios (1) and (2) above, allow an independent 2.0 mgd withdrawal from Copicut Reservoir and a 4.0 mgd withdrawal from South Watuppa Pond. Use present Operating Rules: Copicut and South Watuppa used to augment North Watuppa. Result: the original system Firm Yield falls to 7.5 mgd but assuming that the South Watuppa Pond withdrawal is pumped to the Filtration Plant, the combined Firm Yield, assuming perhaps that the Copicut Reservoir source water is sold to someone in the Buzzards Bay watershed, would be 11.5 mgd. Total yield of the original system, independent South Watuppa Pond and independent Copicut Reservoir would be 13.5 mgd.

Table 10.5.1a
Initial and Boundary Conditions for the Fall River Reservoir System

Source	Initial Elevation (ft-msl)	Full Pond Elevation (ft-msl)	Minimum Elevation (ft-msl)	Maximum Drawdown (ft)
North Watuppa Pond	131.0	132.0	125.67	5.25
South Watuppa Pond	130.5	130.5	125.67	4.83
Copicut Reservoir	140.0	142.0	123.0	19.0

Table 10.5.1b
Estimated Firm Yield of the Fall River Reservoir System

Sources	Firm Yield (mgd)
North Watuppa Pond, South Watuppa Pond and the Copicut Reservoir	7.5
Independent South Watuppa Pond	4.0
Independent Copicut Reservoir	2.0
Combined Firm Yield	13.5

Scenario 2) Given the supply shortfall for Fall River in Scenarios (1) and (2) above, allow an independent 2.0 mgd withdrawal from Copicut Reservoir and a 7.0 mgd withdrawal from South Watuppa Pond. Use present Operating Rules: Copicut and South Watuppa used to augment North Watuppa. Result: the original system Firm Yield still falls to 7.5 mgd but assuming that the South Watuppa Pond withdrawal is pumped to the Filtration Plant, the combined Firm Yield, assuming that the Copicut Reservoir source water is sold to someone in the Buzzards Bay watershed, would be 14.5 mgd. Total yield of the original system, independent South Watuppa Pond and independent Copicut Reservoir would be 16.5 mgd.

Table 10.5.2a
Initial and Boundary Conditions for the Fall River Reservoir System

Source	Initial Elevation (ft-msl)	Full Pond Elevation (ft-msl)	Minimum Elevation (ft-msl)	Maximum Drawdown (ft)
North Watuppa Pond	131.0	132.0	125.67	5.25
South Watuppa Pond	130.5	130.5	125.67	4.83
Copicut Reservoir	140.0	142.0	123.0	19.0

Table 10.5.2b
Estimated Firm Yield of the Fall River Reservoir System

Sources	Firm Yield (mgd)
North Watuppa Pond, South Watuppa Pond and the Copicut Reservoir	7.5
Independent South Watuppa Pond	7.0
Independent Copicut Reservoir	2.0
Combined Firm Yield	16.5

Scenario 3) Given the supply shortfall for Fall River in Scenarios (1) and (2) above, allow an independent 2.0 mgd withdrawal from Copicut Reservoir and a 9.5 mgd withdrawal from South Watuppa Pond. Use present Operating Rules: Copicut and South Watuppa used to augment North Watuppa. Result: the original system Firm Yield still falls to 7.5 mgd but assuming that the South Watuppa Pond withdrawal is pumped to the Filtration Plant, the combined Firm Yield, assuming that the Copicut Reservoir source water is sold to someone in the Buzzards Bay watershed, would be 17.0 mgd. Total yield of the original system, independent South Watuppa Pond and independent Copicut Reservoir would be 19.0 mgd.

Table 10.5.3a
Initial and Boundary Conditions for the Fall River Reservoir System

Source	Initial Elevation (ft-msl)	Full Pond Elevation (ft-msl)	Minimum Elevation (ft-msl)	Maximum Drawdown (ft)
North Watuppa Pond	131.0	132.0	125.67	5.25
South Watuppa Pond	130.5	130.5	125.67	4.83
Copicut Reservoir	140.0	142.0	123.0	19.0

Table 10.5.3b
Estimated Firm Yield of the Fall River Reservoir System

Sources	Firm Yield (mgd)
North Watuppa Pond, South Watuppa Pond and the Copicut Reservoir	7.5
Independent South Watuppa Pond	9.5
Independent Copicut Reservoir	2.0
Combined Firm Yield	19.0

Scenario 4) Allow an independent 4.0 mgd withdrawal from Copicut Reservoir and a 7.0 mgd withdrawal from South Watuppa Pond. Use present Operating Rules: Copicut and South Watuppa used to augment North Watuppa. Result: the original system Firm Yield falls to 6.7 mgd but assuming that the South Watuppa Pond withdrawal is pumped to the Filtration Plant, the combined Firm Yield, assuming that the Copicut Reservoir source water is sold to someone in the Buzzards Bay watershed, would be 13.7 mgd. Total yield of the original system, independent South Watuppa Pond and independent Copicut Reservoir would be 17.7 mgd.

Table 10.5.4a
Initial and Boundary Conditions for the Fall River Reservoir System

Source	Initial Elevation (ft-msl)	Full Pond Elevation (ft-msl)	Minimum Elevation (ft-msl)	Maximum Drawdown (ft)
North Watuppa Pond	131.0	132.0	125.67	5.25
South Watuppa Pond	130.5	130.5	125.67	4.83
Copicut Reservoir	140.0	142.0	123.0	19.0

Table 10.5.4b
Estimated Firm Yield of the Fall River Reservoir System

Sources	Firm Yield (mgd)
North Watuppa Pond, South Watuppa Pond and the Copicut Reservoir	6.7
Independent South Watuppa Pond	7.0
Independent Copicut Reservoir	4.0
Combined Firm Yield	17.7

Scenario 5) Allow an independent 4.0 mgd withdrawal from Copicut Reservoir and a 9.5 mgd withdrawal from South Watuppa Pond. Use present Operating Rules: Copicut and South Watuppa used to augment North Watuppa. Result: the original system Firm Yield falls to 6.7 mgd but assuming that the South Watuppa Pond withdrawal is pumped to the Filtration Plant, the combined Firm Yield, assuming that the Copicut Reservoir source water is sold to someone in the Buzzards Bay watershed, would be 16.2 mgd. Total yield of the original system, independent South Watuppa Pond and independent Copicut Reservoir would be 20.2 mgd.

Table 10.5.5a
Initial and Boundary Conditions for the Fall River Reservoir System

Source	Initial Elevation (ft-msl)	Full Pond Elevation (ft-msl)	Minimum Elevation (ft-msl)	Maximum Drawdown (ft)
North Watuppa Pond	131.0	132.0	125.67	5.25
South Watuppa Pond	130.5	130.5	125.67	4.83
Copicut Reservoir	140.0	142.0	123.0	19.0

Table 10.5.5b
Estimated Firm Yield of the Fall River Reservoir System

Sources	Firm Yield (mgd)
North Watuppa Pond, South Watuppa Pond and the Copicut Reservoir	6.7
Independent South Watuppa Pond	9.5
Independent Copicut Reservoir	4.0
Combined Firm Yield	20.2

10.6 Summary of Results

Results from the previous sections are summarized in Table 10.5.1.

Table 10.6.1
Fall River System Firm Yield with Independent
South Watuppa Pond and/or Copicut Reservoir Withdrawals

Operation Scenario	N. Watuppa System With Firm Yield (mgd)	Independent S. Watuppa (mgd)	Sub-System Firm Yield (mgd)	Independent Copicut Res. (mgd)	Total Firm Yield (mgd)
N. & S Watuppa Ponds and Copicut Res. Current Rules	11.0	0	11.0	0	11.0
N. & S Watuppa Ponds and Copicut Res. Current Rules + 18 ft N. Watuppa Drawdown (new Intake)	10.9	0	10.9	0	10.9

Table 10.6.1 (cont'd)
Fall River System Firm Yield with Independent
South Watuppa Pond and/or Copicut Reservoir Withdrawals

Operation Scenario	N. Watuppa System With Firm Yield (mgd)	Independent S. Watuppa (mgd)	Sub-System Firm Yield (mgd)	Independent Copicut Res. (mgd)	Total Firm Yield (mgd)
N. & S Watuppa Ponds and Copicut Res. Current Rules + Independent S. Watuppa	11.0	4.0	15.0	0	15.0
N. & S Watuppa Ponds and Copicut Res. Current Rules + Independent S. Watuppa	11.0	7.0	18.0	0	18.0
N. & S Watuppa Ponds and Copicut Res. Current Rules + Independent S. Watuppa	10.6	9.5	20.1	0	20.1
N. & S Watuppa Ponds and Copicut Res. Current Rules + Independent Copicut Reservoir	7.5	0	7.5	2.0	9.5
N. & S Watuppa Ponds and Copicut Res. Current Rules + Independent Copicut Reservoir	6.7	0	6.7	4.0	10.7
N. & S Watuppa Ponds and Copicut Res. Current Rules + Independent S. Watuppa and Copicut Reservoir	7.5	4.0	11.5	2.0	13.5

Table 10.6.1 (cont'd)
Fall River System Firm Yield with Independent
South Watuppa Pond and/or Copicut Reservoir Withdrawals

Operation Scenario	N. Watuppa System With Firm Yield (mgd)	Independent S. Watuppa (mgd)	Sub-System Firm Yield (mgd)	Independent Copicut Res. (mgd)	Total Firm Yield (mgd)
N. & S Watuppa Ponds and Copicut Res. Current Rules + Independent S. Watuppa and Copicut Reservoir	7.5	7.0	14.5	2.0	16.5
N. & S Watuppa Ponds and Copicut Res. Current Rules + Independent S. Watuppa and Copicut Reservoir	7.5	9.5	17.0	2.0	19.0
N. & S Watuppa Ponds and Copicut Res. Current Rules + Independent S. Watuppa and Copicut Reservoir	6.7	7.0	13.7	4.0	17.7
N. & S Watuppa Ponds and Copicut Res. Current Rules + Independent S. Watuppa and Copicut Reservoir	6.7	9.5	16.2	4.0	20.2

Appendix I

Monthly and Annual CMIP6 Climate Change Data Tables

This Appendix is to provide future readers estimates of monthly and annual mean precipitation, near-surface air temperature and reservoir surface evaporation at different periods of time between 1950 and 2100, with an atmosphere subject to alternative GHG emission scenarios as generated by five different CMIP6 General Circulation Models (GCMs).

Table AI-1a
Mean Daily Temperature
1950-1980

Month	Obs (^oF)	GCM1 (^oF)	GCM2 (^oF)	GCM3 (^oF)	GCM4 (^oF)	GCM5 (^oF)
1	29.0	28.0	29.9	27.8	29.2	28.8
2	30.3	30.3	31.4	29.4	30.4	30.4
3	37.9	37.1	37.6	37.5	37.8	36.7
4	48.1	47.4	47.5	46.9	47.4	46.8
5	58.0	56.1	57.4	57.1	56.7	57.3
6	67.3	66.0	66.4	65.8	66.5	66.2
7	73.1	71.2	72.0	71.1	72.0	71.7
8	72.2	70.3	70.8	69.8	71.1	70.7
9	64.5	62.9	63.7	62.7	63.8	63.2
10	54.3	52.5	53.8	52.7	53.3	53.0
11	44.0	42.4	44.1	42.6	43.9	42.9
12	33.1	33.6	34.4	33.4	33.9	33.2
Year	51.1	49.8	50.7	49.7	50.5	50.1

Table AI-1b
Mean Monthly Precipitation
1950-1980

Month	Obs (inches)	GCM1 (inches)	GCM2 (inches)	GCM3 (inches)	GCM4 (inches)	GCM5 (inches)
1	4.1	4.1	4.4	4.2	3.9	4.2
2	3.9	3.5	3.5	3.9	3.7	3.8
3	4.5	4.3	4.4	4.5	5.2	4.6
4	3.9	4.3	4.0	4.3	4.2	4.2
5	3.7	3.7	3.1	3	3.7	3.5
6	2.9	3.8	3.8	3.4	3.4	3.5
7	3.1	3.4	3.3	3.7	3.4	3.3
8	4.0	4.3	4.0	3.7	4.3	3.9
9	3.4	3.4	3.6	3.1	3.9	3.5
10	3.7	3.6	3.1	3.8	3.5	3.9
11	4.3	4.1	4.5	4.5	4.5	4.2
12	4.6	4.3	4.6	4.3	4.9	4.4
Year	46.1	46.8	46.3	46.4	48.7	47.1

Table AI-1c
Mean Monthly Reservoir Evaporation
1950-1980

Month	Obs (inches)	GCM1 (inches)	GCM2 (inches)	GCM3 (inches)	GCM4 (inches)	GCM5 (inches)
1	1.2	1.2	1.3	1.2	1.3	1.3
2	1.6	1.6	1.6	1.5	1.6	1.6
3	2.8	2.7	2.7	2.6	2.7	2.7
4	4.0	3.9	4.0	3.9	4.0	4.0
5	5.7	5.6	5.8	5.7	5.7	5.8
6	6.7	6.6	6.9	6.7	6.7	6.8
7	7.2	7.1	7.4	7.2	7.2	7.3
8	6.4	6.2	6.4	6.2	6.3	6.3
9	4.6	4.4	4.5	4.3	4.5	4.4
10	3.0	2.8	2.9	2.8	2.9	2.8
11	1.7	1.6	1.7	1.6	1.6	1.6
12	1.1	1.2	1.3	1.2	1.2	1.2
Year	45.7	44.6	46.5	44.9	45.7	45.6

Table AI-2a
Mean Daily Temperature
ssp245: 2025-2055

Mon	Obs (^oF)	GCM1 (^oF)	GCM2 (^oF)	GCM3 (^oF)	GCM4 (^oF)	GCM5 (^oF)
1	29	34.9	32.9	35.1	33.9	34.5
2	30.3	35.9	34.0	33.5	35.6	36.0
3	37.9	43.7	40.0	40.7	42.0	42.6
4	48.1	51.7	50.7	51.1	51.2	52.3
5	58.0	61.0	60.4	60.2	60.8	62.0
6	67.3	71.1	70.2	70.2	70.3	71.2
7	73.1	77.7	75.7	76.0	76.2	76.6
8	72.2	76.7	74.7	74.9	74.4	75.8
9	64.5	69.5	68.2	68.0	68.3	67.8
10	54.3	58.9	57.2	58.8	57.0	57.5
11	44.0	49.5	48.2	48.1	47.9	48.5
12	33.1	39.3	38.0	39.6	38.4	38.8
Year	51.1	55.8	54.2	54.7	54.7	55.3

Table AI-2b
Mean Monthly Precipitation
ssp245: 2025-2055

Month	Obs (inches)	GCM1 (inches)	GCM2 (inches)	GCM3 (inches)	GCM4 (inches)	GCM5 (inches)
1	4.1	4.5	4.5	5.0	4.5	4.9
2	3.9	3.6	4.0	4.6	4.9	3.8
3	4.5	5.5	5.0	5.9	5.1	4.6
4	3.9	4.7	5.2	4.1	4.9	4.7
5	3.7	4.0	3.9	3.7	4.1	4.3
6	2.9	3.0	3.9	3.6	3.7	3.8
7	3.1	3.4	2.9	2.9	3.4	3.3
8	4	4.2	4.4	3.6	4.9	3.9
9	3.4	3.4	3.6	3.2	3.7	3.4
10	3.7	3.3	3.4	3.7	3.7	4.0
11	4.3	3.9	4.7	4.1	3.9	4.5
12	4.6	5.6	4.7	5.3	5.1	5.2
Year	46.1	49.2	50.2	49.6	51.9	50.3

Table AI-2c
Mean Monthly Reservoir Evaporation
ssp245: 2025-2055

Month	Obs (inches)	GCM1 (inches)	GCM2 (inches)	GCM3 (inches)	GCM4 (inches)	GCM5 (inches)
1	1.2	1.5	1.5	1.6	1.6	1.6
2	1.6	1.9	1.8	1.9	1.9	2.0
3	2.8	3.3	3.0	3.1	3.1	3.2
4	4.0	4.5	4.4	4.5	4.5	4.6
5	5.7	6.2	6.3	6.5	6.3	6.5
6	6.7	7.1	7.4	7.7	7.3	7.5
7	7.2	7.7	8.0	8.2	7.8	8.0
8	6.4	7.0	7.0	7.2	6.9	7.0
9	4.6	5.3	5.1	5.2	5.0	5.0
10	3.0	3.6	3.4	3.4	3.4	3.3
11	1.7	2.1	2.0	2.0	2.0	2.0
12	1.1	1.5	1.5	1.6	1.5	1.6
Year	45.7	51.5	51.2	52.6	50.9	52.2

Table AI-3a
Mean Daily Temperature
ssp585: 2025-2055

Mon	Obs (^{o}F)	GCM1 (^{o}F)	GCM2 (^{o}F)	GCM3 (^{o}F)	GCM4 (^{o}F)	GCM5 (^{o}F)
1	29	35.9	32.1	35.4	34.5	35.8
2	30.3	37.2	34.1	34.9	35.3	37.6
3	37.9	43.6	40.4	42.1	40.8	43.1
4	48.1	53.3	49.9	52.7	50.3	52.3
5	58.0	61.1	61.0	61.7	59.5	62.8
6	67.3	72.7	71.6	70.9	69.0	71.7
7	73.1	78.6	76.8	76.8	74.9	77.5
8	72.2	77.6	75.7	76.3	74.2	75.4
9	64.5	70.6	68.8	68.4	66.9	69.1
10	54.3	61	58.0	59.7	56.3	59.0
11	44.0	50.8	48.5	49.3	47.2	49.5
12	33.1	40.8	38.6	40.6	37.7	40.1
Year	51.1	56.9	54.6	55.7	53.9	56.2

Table AI-3b
Mean Monthly Precipitation
ssp585: 2025-2055

Month	Obs (inches)	GCM1 (inches)	GCM2 (inches)	GCM3 (inches)	GCM4 (inches)	GCM5 (inches)
1	4.1	4.2	4.5	5.1	4.4	4.8
2	3.9	4.5	3.7	4.8	4.7	4.3
3	4.5	4.9	4.8	4.9	4.8	5.7
4	3.9	4.6	5.2	4.4	4.4	5.1
5	3.7	3.9	3.7	3.8	4.0	3.6
6	2.9	2.5	3.5	2.8	3.6	4.4
7	3.1	3.2	3.0	3.7	3.4	2.9
8	4.0	3.4	4.3	3.0	4.6	4.0
9	3.4	3.0	3.3	3.5	3.5	4.0
10	3.7	3.5	3.3	3.1	3.9	2.4
11	4.3	4.5	3.9	5.1	3.9	4.1
12	4.6	5.3	5.1	4.5	5.2	5.4
Year	46.1	47.5	48.3	48.7	50.4	50.8

Table AI-3c
Mean Monthly Reservoir Evaporation
ssp585: 2025-2055

Month	Obs (inches)	GCM1 (inches)	GCM2 (inches)	GCM3 (inches)	GCM4 (inches)	GCM5 (inches)
1	1.2	1.6	1.5	1.6	1.6	1.7
2	1.6	2.0	1.8	1.9	1.9	2.0
3	2.8	3.3	3.0	3.2	3.1	3.3
4	4.0	4.6	4.5	4.6	4.4	4.7
5	5.7	6.4	6.5	6.6	6.1	6.6
6	6.7	7.3	7.7	7.7	7.2	7.7
7	7.2	8.0	8.2	8.2	7.7	8.2
8	6.4	7.2	7.2	7.2	6.8	7.2
9	4.6	5.5	5.2	5.2	4.9	5.2
10	3.0	3.8	3.4	3.5	3.3	3.5
11	1.7	2.2	2.0	2.1	1.9	2.1
12	1.1	1.5	1.5	1.6	1.5	1.7
Year	45.7	53.3	52.3	53.4	50.1	53.6

Table AI-4a
Mean Daily Temperature
ssp245: 2070-2100

Month	Obs (°F)	GCM1 (°F)	GCM2 (°F)	GCM3 (°F)	GCM4 (°F)	GCM5 (°F)
1	29.0	39.3	35.2	36.1	37.9	37.9
2	30.3	39.5	35.3	36.9	39.2	39.2
3	37.9	47.3	41.9	43.5	45.1	44.9
4	48.1	56.3	52.3	53.5	54.8	54.4
5	58.0	63.7	62.4	62.6	63.4	63.4
6	67.3	75.3	72.8	72.3	73.0	73.3
7	73.1	81.3	78.7	77.7	79.0	78.9
8	72.2	79.7	77.6	76.9	77.3	77.8
9	64.5	72.1	70.9	69.5	70.3	70.3
10	54.3	62.5	59.9	60.1	59.9	60.8
11	44.0	53.0	49.9	50.2	50.7	51.1
12	33.1	43.7	40.8	41.8	40.9	40.4
Year	51.1	59.5	56.5	56.8	57.6	57.7

Table AI-4b
Mean Monthly Precipitation
ssp245: 2070-2100

Month	Obs (inches)	GCM1 (inches)	GCM2 (inches)	GCM3 (inches)	GCM4 (inches)	GCM5 (inches)
1	4.1	5.1	4.1	4.4	4.3	5
2	3.9	4.4	3.8	4.5	5.7	4.1
3	4.5	6.0	4.9	5.2	6.2	5.0
4	3.9	5.2	5.2	4.7	4.5	4.3
5	3.7	4.2	3.2	3.9	4.3	4.2
6	2.9	3.5	3.8	3.2	3.8	3.1
7	3.1	3.2	3.6	4.2	3.5	3.0
8	4.0	4.5	3.5	3.9	5.4	3.7
9	3.4	4.0	3.3	3.1	3.8	3.5
10	3.7	4.1	4.6	3.8	3.5	3.5
11	4.3	4.4	4.4	3.8	3.7	4.5
12	4.6	5.2	4.4	6.1	5.1	5.1
Year	46.1	53.9	48.7	50.7	53.7	49.1

Table AI-4d
Mean Monthly Reservoir Evaporation
ssp245: 2070-2100

Month	Obs (inches)	GCM1 (inches)	GCM2 (inches)	GCM3 (inches)	GCM4 (inches)	GCM5 (inches)
1	1.2	1.8	1.6	1.7	1.8	1.7
2	1.6	2.2	1.9	2.0	2.2	2.1
3	2.8	3.6	3.2	3.3	3.5	3.5
4	4.0	5.0	4.7	4.8	4.8	4.8
5	5.7	6.9	6.7	6.7	6.6	6.6
6	6.7	7.9	7.9	7.8	7.6	7.5
7	7.2	8.5	8.5	8.3	8.1	8.1
8	6.4	7.6	7.5	7.3	7.2	7.2
9	4.6	5.7	5.4	5.3	5.3	5.4
10	3.0	3.9	3.6	3.6	3.6	3.7
11	1.7	2.4	2.2	2.2	2.2	2.2
12	1.1	1.8	1.6	1.7	1.7	1.7
Year	45.7	57.2	54.6	54.6	54.4	54.3

Table AI-5a
Mean Daily Temperature
ssp585: 2070-2100

Month	Obs (^oF)	GCM1 (^oF)	GCM2 (^oF)	GCM3 (^oF)	GCM4 (^oF)	GCM5 (^oF)
1	29	42.8	39.5	42	39.1	40.9
2	30.3	43.8	39.8	42.2	40.6	41.6
3	37.9	50.6	46.2	47.5	46.0	48.2
4	48.1	59.4	55.4	58.1	55.3	58.3
5	58.0	68.2	66.6	67.0	64.5	68.0
6	67.3	79.9	77.3	77.3	74.3	77.2
7	73.1	86.4	84.0	83.0	80.0	82.8
8	72.2	84.6	83.2	82.6	79.1	81.6
9	64.5	77.4	75.5	75.6	72.4	74.7
10	54.3	68.6	64.1	65.4	61.4	64.0
11	44.0	58.9	54.6	53.7	52.7	54.3
12	33.1	48.9	44.5	45.3	42.0	44.1
Year	51.1	64.1	60.9	61.7	58.9	61.3

Table AI-5b
Mean Monthly Precipitation
ssp585: 2070-2100

Month	Obs (inches)	GCM1 (inches)	GCM2 (inches)	GCM3 (inches)	GCM4 (inches)	GCM5 (inches)
1	4.1	5.9	5.1	5.6	5.9	4.7
2	3.9	5.6	4.6	5.0	5.4	4.2
3	4.5	7.0	5.9	5.4	5.8	6.5
4	3.9	6.0	5.9	5.3	5.4	5.5
5	3.7	3.9	3.4	3.5	4.5	4.0
6	2.9	2.9	3.6	3.2	3.8	4.0
7	3.1	2.8	3.0	3.8	3.9	3.5
8	4.0	4.2	3.8	3.6	3.7	3.1
9	3.4	3.2	3.4	3.2	3.2	4.4
10	3.7	1.9	3.4	2.6	2.9	3.0
11	4.3	4.2	4.4	5.2	5.1	4.9
12	4.6	6.1	5.8	6.6	4.7	6.2
Year	46.1	53.5	52.2	53	54.3	53.9

Table AI-5c
Mean Monthly Reservoir Evaporation
ssp585: 2070-2100

Month	Obs (inches)	GCM1 (inches)	GCM2 (inches)	GCM3 (inches)	GCM4 (inches)	GCM5 (inches)
1	1.2	2.0	1.8	1.9	1.8	1.9
2	1.6	2.5	2.2	2.3	2.2	2.3
3	2.8	4.0	3.7	3.8	3.6	3.8
4	4.0	5.6	5.2	5.2	4.9	5.3
5	5.7	7.6	7.2	7.0	6.7	7.2
6	6.7	8.7	8.4	8	7.6	8.1
7	7.2	9.3	9.1	8.6	8.2	8.6
8	6.4	8.5	8.2	7.9	7.4	7.7
9	4.6	6.4	6.1	6.0	5.6	5.8
10	3.0	4.6	4.2	4.2	3.9	4.1
11	1.7	2.8	2.5	2.6	2.4	2.5
12	1.1	2.1	1.8	1.8	1.7	1.9
Year	45.7	63.8	60.1	59.3	55.8	59.2

Appendix II

Annual Mean Daily Streamflow CMIP6 Climate Change Data Tables

This Appendix is to provide future readers estimates of annual average daily inflows at the outlet of each of the surface water supply sources including the Town of Somerset's Segreganset River pumping station. These data are provided for different periods of time between 1950 and 2100, with an atmosphere subject to alternative GHG emission scenarios as generated by five different CMIP6 General Circulation Models (GCMs). Because the annually averaged daily streamflow model used in this study is estimated using multivariate regional parameter model equations, each set of annual mean values is unique to one of the six watersheds analyzed during the course of this study. The six watersheds are: North Watuppa Pond; South Watuppa Pond; Copicut Reservoir; Somerset Reservoir; the Town of Somerset Pumping Station sited on the Segreganset River and Stafford Pond.

Table AII-1
North Watuppa Pond
Annual Mean Daily Streamflow

GHG Scenario	Obs (ft^3/s)	GCM1 (ft^3/s)	GCM2 (ft^3/s)	GCM3 (ft^3/s)	GCM4 (ft^3/s)	GCM5 (ft^3/s)
Historic: 1950-1980	14.2	14.9	14.4	14.7	16	15.1
ssp245: 2025-2055	-	15.2	16.2	15.7	17.2	16.0
ssp585: 2025-2055	-	13.9	14.9	14.9	16.4	16.1
ssp245: 2070-2100	-	17.5	14.7	15.9	17.7	14.8
ssp585: 2070-2100	-	16.3	16.1	16.4	17.9	17.1

Table AII-2
South Watuppa Pond
Annual Mean Daily Streamflow

GHG Scenario	Obs (ft^3/s)	GCM1 (ft^3/s)	GCM2 (ft^3/s)	GCM3 (ft^3/s)	GCM4 (ft^3/s)	GCM5 (ft^3/s)
Historic: 1950-1980	28.1	29.4	28.5	28.9	31.3	29.5
ssp245: 2025-2055	-	29.9	31.6	30.7	33.4	31.3
ssp585: 2025-2055	-	27.8	29.3	29.4	31.9	31.6
ssp245: 2070-2100	-	33.9	29.2	31.3	34.5	29.1
ssp585: 2070-2100	-	31.9	31.5	32.2	34.6	33.3

Table AII-3
Copicut Reservoir
Annual Mean Daily Streamflow

GHG Scenario	Obs (ft^3/s)	GCM1 (ft^3/s)	GCM2 (ft^3/s)	GCM3 (ft^3/s)	GCM4 (ft^3/s)	GCM5 (ft^3/s)
Historic: 1950-1980	12.2	12.8	12.4	12.6	13.8	12.9
ssp245: 2025-2055	-	13.0	13.9	13.5	14.8	13.7
ssp585: 2025-2055	-	12.0	12.8	12.8	14.1	13.9
ssp245: 2070-2100	-	15.0	12.7	13.7	15.3	12.7
ssp585: 2070-2100	-	14.0	13.8	14.1	15.4	14.7

Table AII-4
Somerset Reservoir
Annual Mean Daily Streamflow

GHG Scenario	Obs (ft^3/s)	GCM1 (ft^3/s)	GCM2 (ft^3/s)	GCM3 (ft^3/s)	GCM4 (ft^3/s)	GCM5 (ft^3/s)
Historic: 1950-1980	2.3	2.5	2.4	2.5	2.8	2.5
ssp245: 2025-2055	-	2.5	2.8	2.7	3.1	2.7
ssp585: 2025-2055	-	2.2	2.5	2.5	2.9	2.8
ssp245: 2070-2100	-	3.1	2.4	2.7	3.2	2.4
ssp585: 2070-2100	-	2.8	2.7	2.8	3.2	3.0

Table AII-5
Segreganset River Pumping Station
Annual Mean Daily Streamflow

GHG Scenario	Obs (ft^3/s)	GCM1 (ft^3/s)	GCM2 (ft^3/s)	GCM3 (ft^3/s)	GCM4 (ft^3/s)	GCM5 (ft^3/s)
Historic: 1950-1980	21.2	22.3	21.5	21.9	23.9	22.5
ssp245: 2025-2055	-	22.7	24.1	23.4	25.6	23.9
ssp585: 2025-2055	-	20.8	22.2	22.3	24.4	24.1
ssp245: 2070-2100	-	26.0	22.0	23.8	26.4	22.1
ssp585: 2070-2100	-	24.3	24.0	24.5	26.6	25.4

Table AII-6
Stafford Pond
Annual Mean Daily Streamflow

GHG Scenario	Obs (ft^3/s)	GCM1 (ft^3/s)	GCM2 (ft^3/s)	GCM3 (ft^3/s)	GCM4 (ft^3/s)	GCM5 (ft^3/s)
Historic: 1950-1980	21.2	22.3	21.5	21.9	23.9	22.5
ssp245: 2025-2055	-	22.7	24.1	23.4	25.6	23.9
ssp585: 2025-2055	-	20.8	22.2	22.3	24.4	24.1
ssp245: 2070-2100	-	26.0	22.0	23.8	26.4	22.1
ssp585: 2070-2100	-	24.3	24.0	24.5	26.6	25.4