Summary Report

for

Maplewood Avenue over North Mill Pond Bridge Rehabilitation and Water Diversion Hydrologic and Hydraulic Analyses

Portsmouth, New Hampshire

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A. Executive Summary

The City of Portsmouth is proposing to temporarily repair the corrugated metal arch bridge which carries Maplewood Avenue over North Mill Pond until such time that a full structure replacement can be implemented. The proposed temporary repair involves the application of a 4.5-inch thick geopolymer liner to the underside of the metal arch. Once applied, the geopolymer liner would reduce the existing waterway opening area, therefore, we have completed hydrologic and hydraulic analyses to: (1) quantify the impact of the liner on flood stages and other hydraulic characteristics and (2) evalute the effectiveness of adding a new culvert through the roadway embankment and removing the sanitary sewer main which passes through the bridge opening on mitigating adverse hydraulic impacts caused by the liner. Five bridge rehabiliation alternatives were evaluated as follows:

- Bridge Rehabilitation Alternative 1 – Geopolymer Liner
- Bridge Rehabilitation Alternative 2.1 – Geopolymer Liner and Twin 48-inch Diameter Culverts
- Bridge Rehabilitation Alternative 2.2 – Geopolymer Liner and Single 60-inch Diameter Culvert
- Bridge Rehabilitation Alternative 2.3 – Geopolymer Liner and Single 72-inch Diameter Culvert
- Bridge Rehabilitation Alternative 3 – Geopolymer Liner and Sewer Main Relocation

Table 1 summarizes the calculated 50- and 100-year flood stages in North Mill Pond on the south side of Maplewood Avenue for each bridge rehabilitation alternative and the change from existing flood levels.

Table 1 – Peak water levels in North Mill Pond on the south side of Maplewood Avenue for Existing Conditions and Bridge Rehabilitation Alternatives

Bridge Rehabilitation Alternative	Peak Water Level in North Mill Pond on South Side of Maplewood Ave. (feet, NAVD88)*		Change from Existing Water Level (feet)	
	50-year	100-year	50-year	100-year
Existing Conditions	7.96	8.41	---	---
1	7.98	8.45	+ 0.02	+ 0.04
2.1	7.94	8.38	- 0.02	- 0.03
2.2	7.94	8.39	- 0.02	- 0.02
2.3	7.93	8.37	- 0.03	- 0.04
3	7.97	8.44	+ 0.01	+ 0.03

* *calculated at the centroid of the waterbody on the south side of Maplewood Ave. (N 211315, E 1224317)*

The bridge opening will need to be dewatered to allow application and curing of the geopolymer liner, which is expected to take about three weeks. Tidal flows into the portion of North Mill Pond south of Maplewood Avenue will need to be excluded or diverted around or through the waterway opening in closed conduits and freshwater outflow from Hodgson Brook will need to be diverted around the work area or potentially stored in the portion of the pond on the south side of the road until construction has been completed. Therefore, we have also completed hydraulic analyses to evaluate the feasibility of several construction dewatering alternatives. Five water diversion alternatives have been studied as follows:

- Water Diversion Alternative 1 – Tidal Flow Exclusion and Freshwater Storage
- Water Diversion Alternative 2.1 – Temporary 48-inch Culverts
- Water Diversion Alternative 2.2 – Temporary 72-inch Culverts
- Water Diversion Alternative 3 – Phased Water Diversion
- Water Diversion Alternative 4 – Permanent Culvert

Table 2 summarizes the calculated flood stages in North Mill Pond on the north and south sides of Maplewood Avenue for each water diversion alternative under the hydrologic and tidal conditions assumed during construction, including runoff from one or more 1-year rainstorms and a 2-year high tide water level.

Table 2 – Peak water levels in North Mill Pond on the north and south sides of Maplewood Avenue for Water Diversion Alternatives

Water Diversion Alternative	Peak Water Level in North Mill Pond on North Side of Maplewood Ave. (feet, NAVD88)*	Peak Water Level in North Mill Pond on South Side of Maplewood Ave. (feet, NAVD88)
1	6.42	5.75
2.1	6.42	4.22
2.2	6.42	4.98
3	6.42	5.36
4	6.42	3.32

* *2-year high tide water level*

B. Hydrologic Analyses

Our approach to the hydrologic analysis was based on the requirements and recommendations included in the following documents:

- *Bridge Design Manual, Chapter 2, Bridge Selection*. January 2015 – v 2.0 (Revised August 2018). NH Department of Transportation (NHDOT); and

- *Sea-level Rise, Storm Surges, and Extreme Precipitation in Coastal New Hampshire: Analysis of Past and Projected Future Trends*. 2014. New Hampshire Coastal Risk and Hazards Commission Science and Technical Advisory Panel (NHCRHC STAP). http://www.nhcrhc.org/wp-content/uploads/2014-STAP-final-report.pdf.

Maplewood Avenue is classified as a Tier 5 highway (i.e. local road). Per the NHDOT Bridge Design Manual, the design flood for calculating freeboard to the superstructure of bridges on local roads is the 50-year event and the design flood for substructure scour analysis is the 100-year event.

The SCS unit hydrograph method was used with the HydroCAD computer program to estimate runoff hydrographs resulting from the 1-, 50-, and 100-year, 24-hour rainfalls. This method, which is an approved hydrologic analysis method listed in the Bridge Design Manual, uses the SCS unit hydrograph (representing the runoff resulting from 1 inch of excess precipitation), synthetic rainfall distribution curve (specifying the distribution of rainfall throughout the storm duration), and the following variables:

- Watershed Area;

- Rainfall depth;

- Runoff Curve Number (measure of the land's capacity to retain precipitation, based on soil and land cover characteristics); and

- Time of Concentration (time required for runoff to travel from the most hydraulically distant point of a watershed to its outlet).

B.1. Watershed Delineation

The main tributary to North Mill Pond is Hodgson Brook, which enters the southwest end of the pond at the outlet of a stone masonry box culvert beneath Bartlett Street. North Mill Pond also receives runoff from areas immediately east and west of the pond which drain directly to it, rather than to Hodgson Brook.

The following data was used to delineate the area draining to North Mill Pond at Maplewood Avenue:

- Digital elevation model (DEM) generated from 2011 LiDAR data downloaded from NHGRANIT (note that the 2011 LiDAR data is the most recent dataset which covers the entire watershed – more recent data only covers a portion of the watershed);

- Stormwater infrastructure GIS data (storm drains and drainage structures) provided by James McCarty, GIS Manager for the City of Portsmouth;

- 1-foot resolution color aerial photography captured in 2017 and 6-inch resolution color aerial photography captured in 2010; and

- Google Maps Street View.

The watershed includes a significant amount of commercial, industrial, and residential development which has altered the natural drainage patterns. Due to these alterations, the stormwater infrastructure GIS data provided by the City was invaluable in determining the current drainage pathways and watershed boundary. However, this data does not include all of the closed drainage pipes and structures nor does it contain other drainage information such as roof drain connections and parking garage stormwater infrastructure. Where the stormwater infrastructure GIS data was incomplete, the LiDAR DEM, aerial photography, and Google Maps Street View were used to estimate flow pathways and delineate the watershed boundary.

The area draining to North Mill Pond at Maplewood Avenue was determined to be 2,628 acres (4.11 square miles). The watershed boundary is shown on the Watershed Relief Map and Drainage Plan in Appendix 1.

B.2. Rainfall

In accordance with the recommendations in NHDRHC STAP (2014), rainfall depths and distributions at the watershed centroid were obtained from the Northeast Regional Climate Center (NRCC) using their “Extreme Precipitation” web tool (http://precip.eas.cornell.edu) (see Appendix 1). Table 3 summarizes the rainfall depths for the analyzed storms and Figure 1 shows the rainfall distribution curves for these events.

Table 3 – NRCC rainfall data

Storm Frequency	24-hour Rainfall Depth
1-year	2.66”
50-year	7.39”
100-year	8.86”

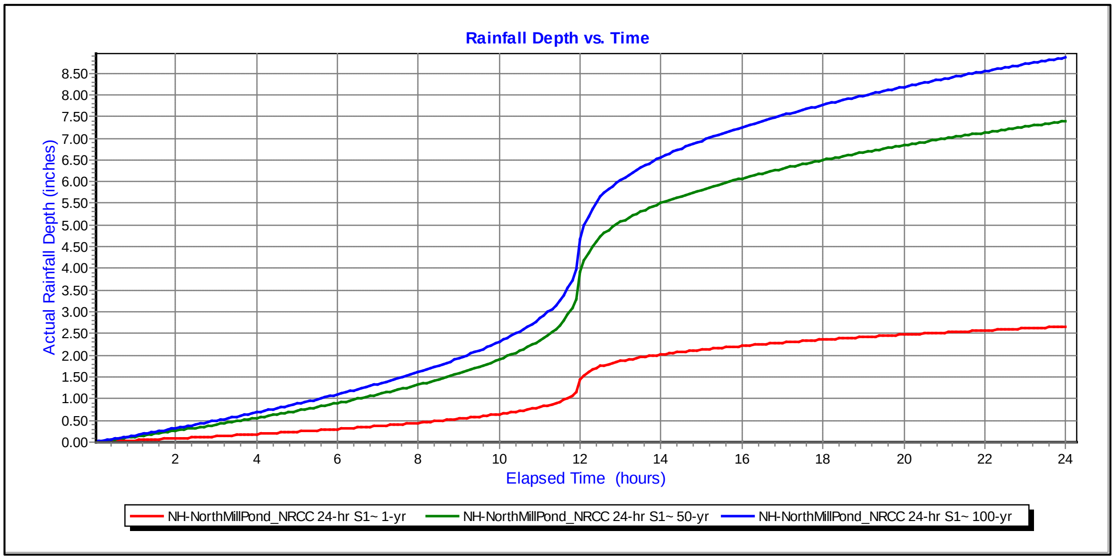


Figure 1 – Rainfall distribution curves for 1-, 50-, and 100-year storms

B.3. <u>Runoff Curve Number</u>

The composite runoff curve number (CN) for the watershed was estimated using the following data sources:

- "Impervious Surfaces in the Coastal Watershed of NH and Maine, High Resolution – 2015" GIS layer downloaded from NHGRANIT;
- "Land Use 2015 - Southeastern New Hampshire" GIS layer downloaded from NHGRANIT;
- 1-foot resolution color aerial photography captured in 2017; and
- digital NRCS soil mapping.

The land use polygons were clipped to remove those portions covered by the impervious layer. The remaining portions of the land use polygons were then assigned one of the land cover types and conditions listed in Table 2-2 of the SCS Technical Release 55 (TR-55) publication by inspecting the ground cover of these polygons shown on 2017 aerial photography. For example, the aerial photography shows that the land use "electric, gas, and other utilities" polygons, which generally cover utility right-of-ways, support predominantly brush and tall herbaceous vegetation over more than 75 percent of the ground surface, which most closely matches the "brush, good" cover type and condition in the TR-55 manual. The "North Mill Pond Watershed Land Cover" table in Appendix 1 summarizes the correlations between the land use layers and TR-55 cover types.

Once the land cover mapping was completed for the entire watershed, it was combined with NRCS soil mapping to create soil-land cover polygons for each combination of hydrologic soil group (HSG) and land cover (e.g. brush, good, HSG B). Each soil-land cover combination was then assigned a CN from Table 2-2 of the TR-55 manual. The "North Mill Pond Watershed Soil – Land Cover Map" in Appendix 1 shows the soil-land cover polygons and the "North Mill Pond Watershed Soil - Land Cover Polygons" table, also in Appendix 1, summarizes the areas and CNs for each soil-land cover combination.

This cumulative area of each soil-land cover combination was determined and used to calculate the area-weighted composite CN for the entire watershed. This value was determined to be 73, which suggests a relatively high runoff potential due to the extent of development in the watershed, approximately 36% of which is covered by impervious surfaces.

B.4. <u>Time of Concentration</u>

The time of concentration (Tc) – the time for runoff to travel from the hydraulically most distant point of the watershed to the bridge – was estimated using the velocity method. The flow path from the uppermost point of the watershed to the bridge was identified using the DEM and storm drain GIS data and has a total length of 23,320 feet (see Drainage Plan in Appendix 1). Twenty-six discreet flow segments were delineated – one sheet flow segment and one shallow concentrated flow segment at the upper end of the watershed followed by alternating pipe and channel flow segments as the drainage path crosses multiple roadways on its way to North Mill Pond.

A terrain profile was cut along the flow path and used to identify the start and end of each channel and pipe segment, the invert elevations at these break points, and the length and slope of each segment. The storm drain GIS data included culvert diameter and material attribute information for a few of the pipe runs; however, most of these features did not include this data. For these pipe segments the pipe diameter and material were estimated. A typical cross-section was cut across each channel flow segment and the ground profile from the DEM was used to determine channel geometry for use in calculating travel time. Geometry was measured at an estimated maximum bankfull depth of one foot. The 2017 aerial photography was used to identify land cover along the channel flow segments from which Manning's roughness coefficients were estimated. Most channel segments have brush or forest cover and were assigned a roughness coefficient of 0.10. The numerous roadway embankments along the flow path likely have restricted outlets which provide floodwater storage and act to increase Tc and lag time between the start of the runoff event and its peak. Although the analysis did not directly account for the storage effects of these manmade basins, the assignment of relatively high roughness coefficients to the channel flow segments does, to some extent, account for these effects.

The total Tc for the watershed was calculated at 564 minutes (9.4 hours). The "North Mill Pond Watershed Time of Concentration" table in Appendix 1 summarizes the data for each flow segment.

B.5. Rainfall Runoff Simulation

The hydrologic model yielded the following peak discharges and freshwater inflow hydrographs to North Mill Pond.

Table 4 – Peak discharge estimates at Maplewood Avenue

Storm Frequency	Peak Discharge (cfs)
1-year	133
50-year	908
100-year	1,179

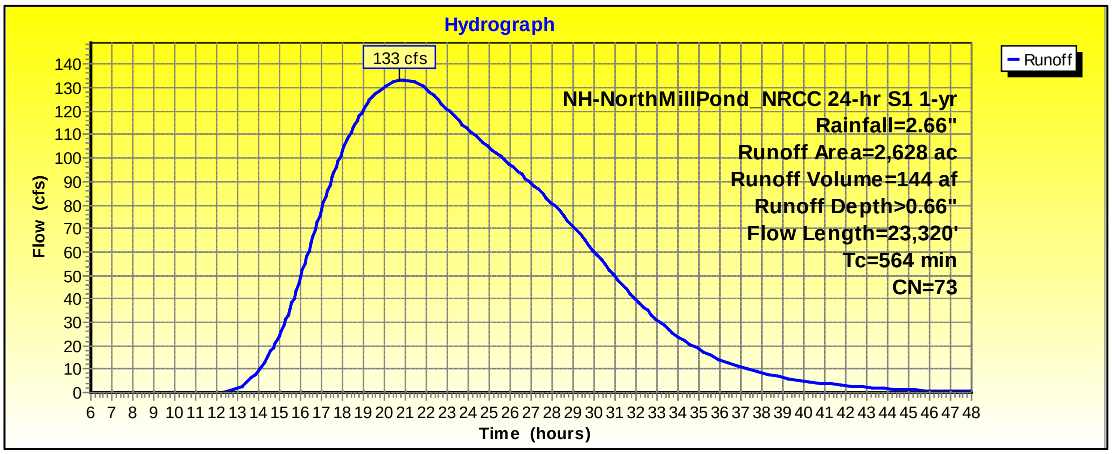


Figure 2 – North Mill Pond 1-year inflow hydrograph

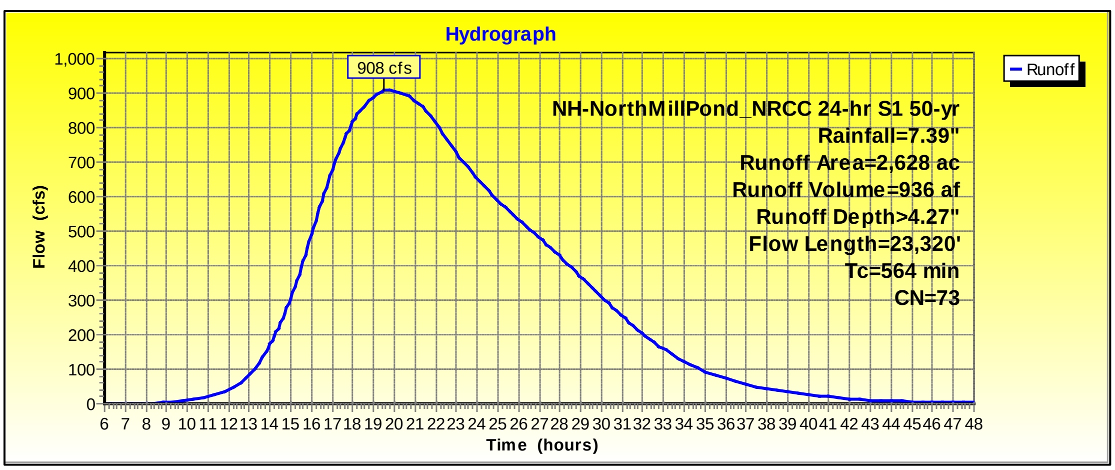


Figure 3 – North Mill Pond 50-year inflow hydrograph

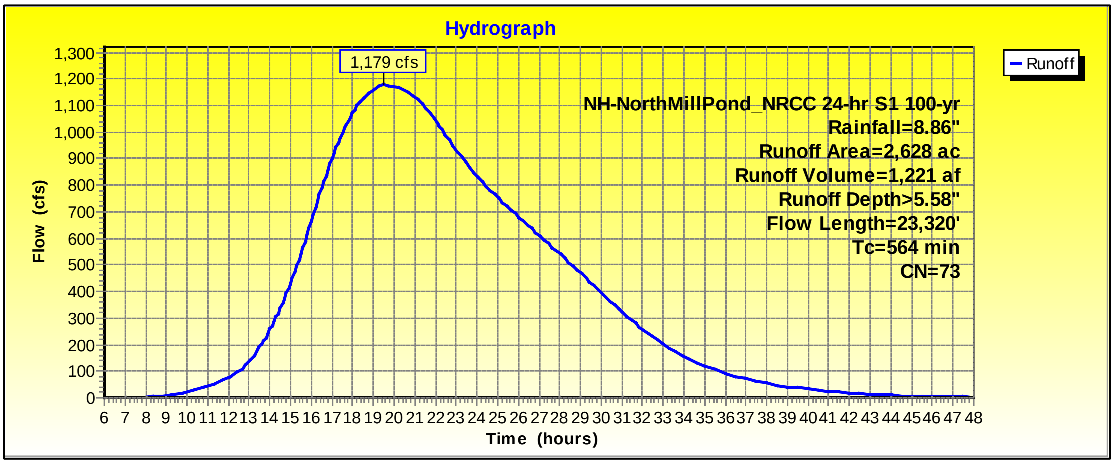


Figure 4 – North Mill Pond 100-year inflow hydrograph

Additional output from the HydroCAD model is included in Appendix 1.

C. Hydraulic Analyses – All Scenarios

Hydraulic analyses for existing conditions and the bridge rehabilitation and water diversion alternatives were completed via the development and execution of two-dimensional (2D) unsteady flow models using the U.S. Army Corps of Engineers HEC-RAS program (version 6.3). Impacts to water levels and other hydraulic characteristics resulting from the proposed bridge repairs were evaluated by comparing the results of the existing conditions models to the results of the bridge rehabilitation models. The water diversion models were developed only to evaluate the feasibility of the various water diversion alternatives during construction.

C.1. Hydraulic Model Geometry

The hydraulic models cover an area from a point on Hodgson Brook about 1,200 feet southwest (upstream) from Bartlett Street to a point approximately 500 feet north of Maplewood Avenue. Geometry for the existing conditions models was developed from a combination of field survey data and publicly-available LiDAR data (Coastal New Hampshire - 2014 data set). With the exception of the area in the vicinity of the bridge, the same geometry was used in all of the other models.

The LiDAR data does not include below-water ground elevations (i.e. bathymetry), geometry of the bridge at Maplewood Avenue, or geometry of the box culvert at Bartlett Street; therefore, this information was field surveyed. Bathymetry of North Mill Pond and the submerged area north of Maplewood Avenue was surveyed by Doucet Survey, LLC in late 2019 and early 2020. The Doucet survey also included topography along about 800 feet of Maplewood Avenue, portions of the shoreline north and south of the road, and other above-water areas in the project vicinity. However, it did not include detailed geometry of the existing bridge, bathymetry at the bridge inlet or outlet, geometry of the box culvert at Bartlett Street, or channel bottom elevations at the box culvert inlet or outlet; therefore, this information was field surveyed by Headwaters Consulting, LLC in September 2020. All field survey data was collected relative to NH State Plane coordinates and NAVD88 elevations, which are the same coordinate system and elevation datum the LiDAR data is referenced to (though the LiDAR data was converted from metric to U.S. customary units). This allowed the field survey data to be merged with the LiDAR data to produce a comprehensive digital elevation model (DEM) of the study area. Figure 5 shows the hydraulic study area DEM with the Doucet field survey area outlined in red and the Headwaters field survey areas outlined in blue. Terrain information in all other areas was generated from the LiDAR data.

As shown in Figure 5, there are many buildings within the hydraulic study area. The building footprints were provided by the City of Portsmouth in GIS format and were uniformly assigned an elevation value of 30 feet in the DEM so that they would be recognized as flow obstructions in the model.

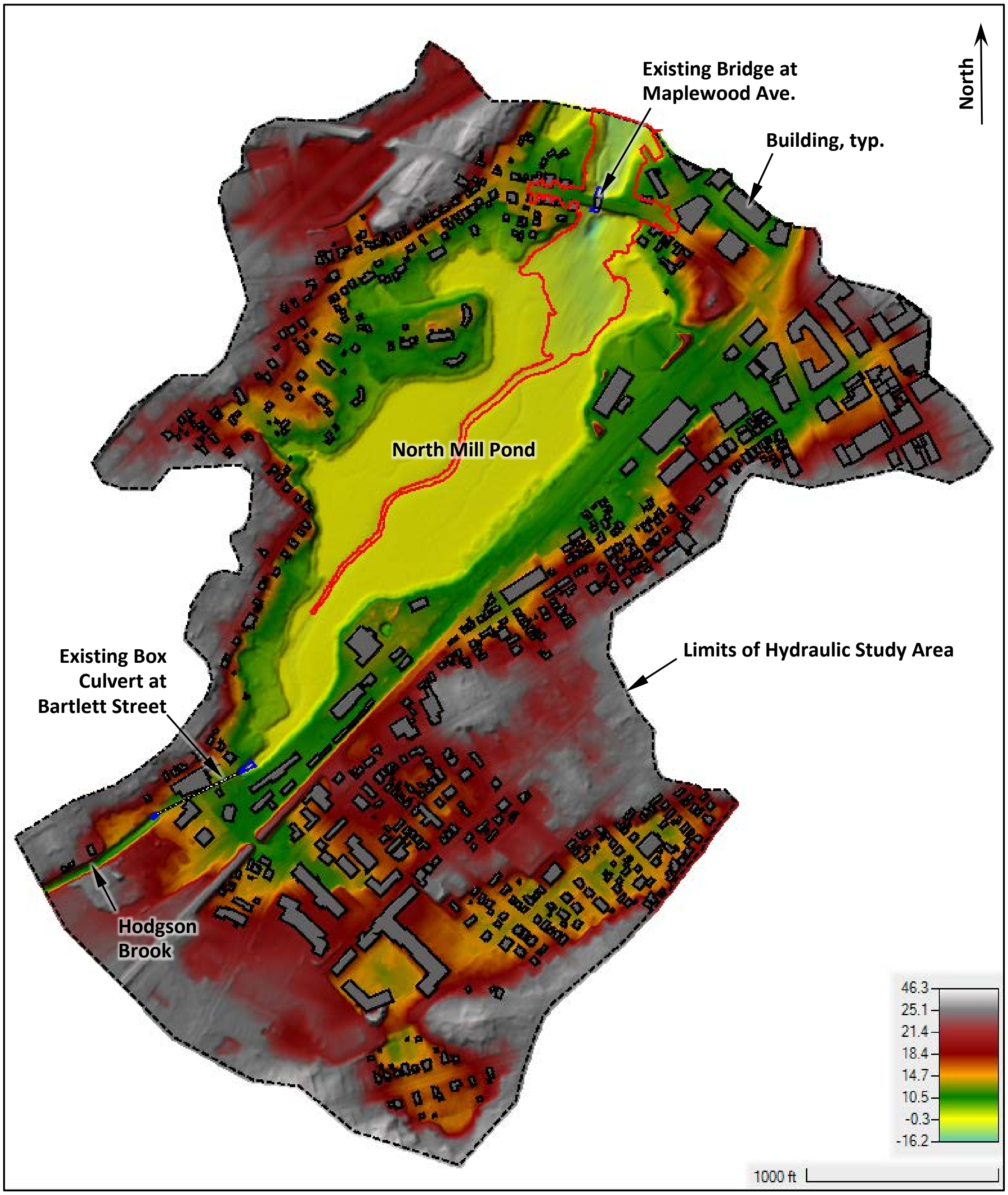


Figure 5 – Existing conditions digital elevation model (DEM) of the hydraulic study area showing areas field surveyed by Doucet Survey, LLC outlined in red and areas field surveyed by Headwaters Consulting, LLC outlined in blue

A 2D computational mesh with a 25-foot x 25-foot cell size was overlaid on the DEM. Breaklines were defined along the tops of embankments and other elevated features which obstruct the flow (e.g. Maplewood Avenue) to prevent the model from calculating flow over them before they are actually overtopped. Figure 6 shows the computational mesh layout in the vicinity of Maplewood Avenue for the existing conditions hydraulic models.

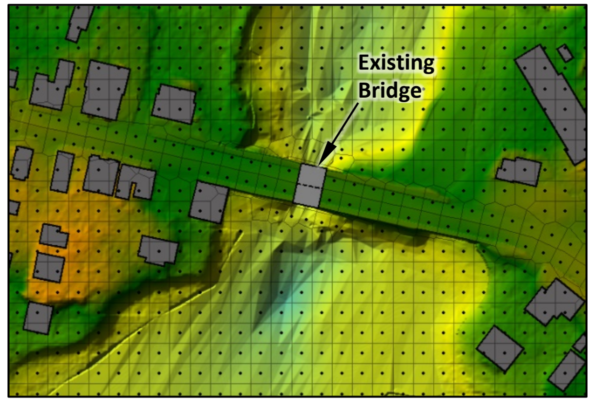


Figure 6 – Computational mesh in the vicinity of Maplewood Avenue used in the existing conditions hydraulic models

C.2. Roughness

2017 aerial photography and the "Impervious Surfaces in the Coastal Watershed of NH and Maine, High Resolution – 2015" and "Land Use 2015 - Southeastern New Hampshire" GIS layers downloaded from NHGRANIT were used to map land cover in the hydraulic study area via the creation of GIS land cover polygons. Manning's n surface roughness coefficients were then assigned to each land cover type for use in the hydraulic modeling. Figure 7 shows the land cover mapping and Table 5 lists the roughness coefficients assigned to the land cover classifications. A full-size copy of the land cover map is included in Appendix 2.

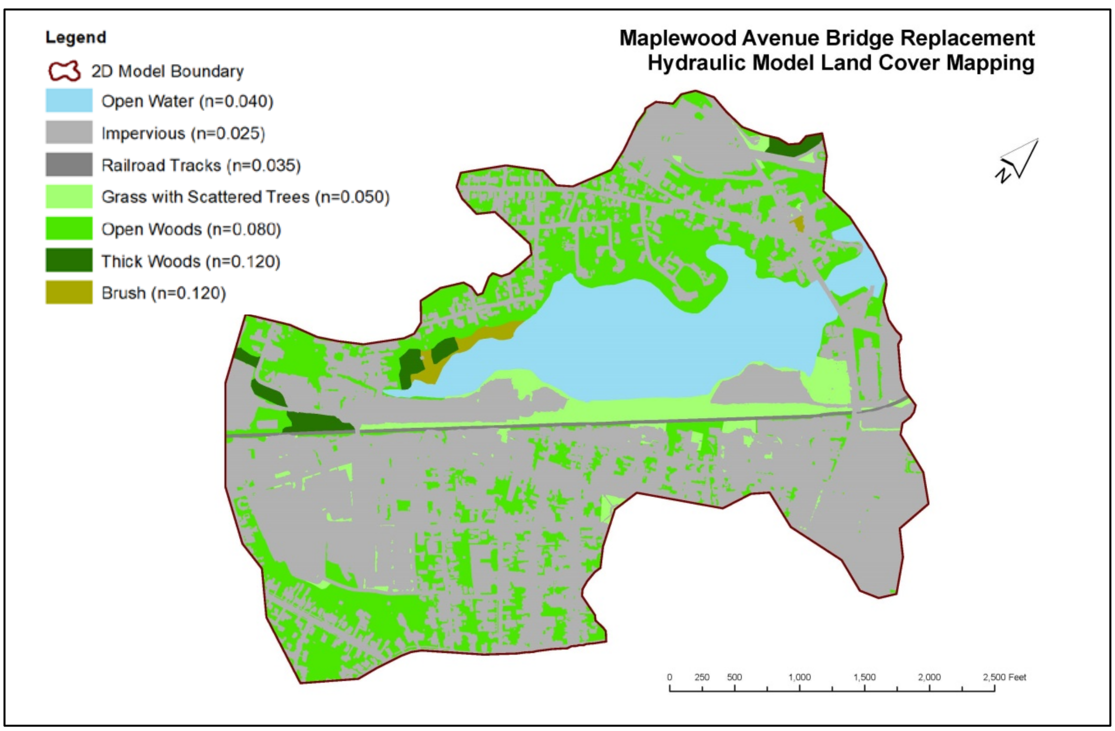


Figure 7 – Land cover mapping

Table 5 – Manning's n roughness coefficients

Land Cover Classification	Manning's n Roughness Coefficient
Open Water	0.040
Impervious Surface	0.025
Railroad Tracks	0.035
Grass with Scattered Trees	0.050
Open Woods	0.080
Thick Woods	0.120
Brush	0.120

Figure 8 shows the hydraulic study area (i.e. 2D model boundary) overlaid on the 2017 aerial photography.

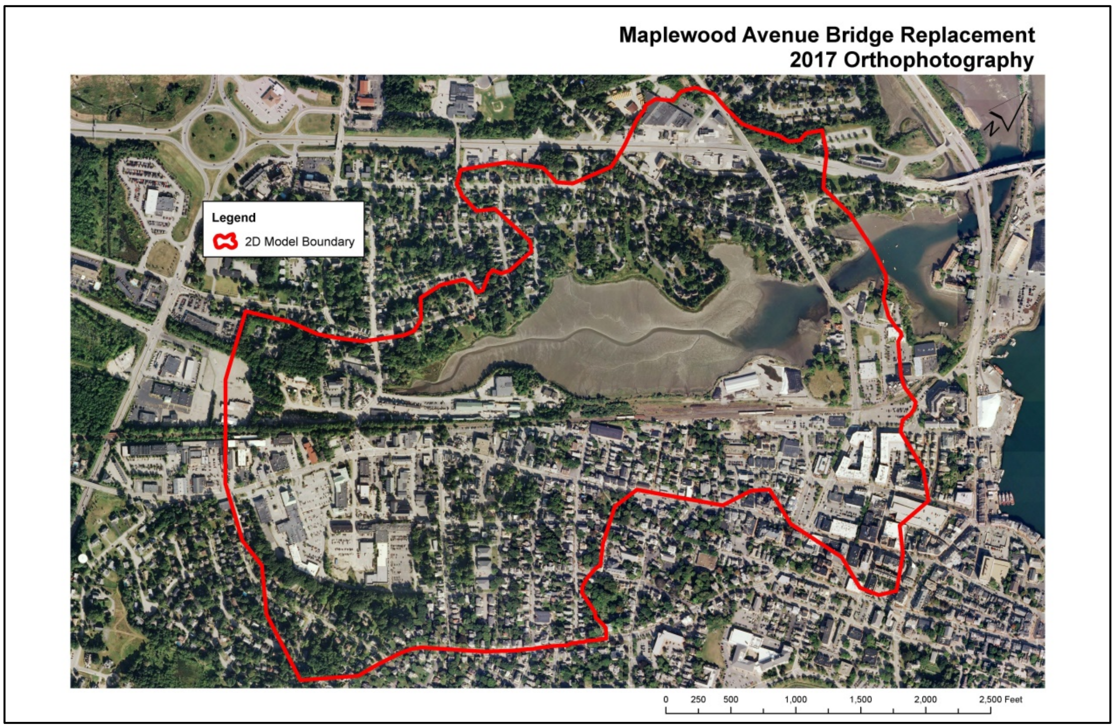


Figure 8 – Hydraulic study area boundary overlaid on 2017 aerial photography

C.3. <u>Boundary Conditions</u>

External boundary conditions were defined at the upstream (south) and downstream (north) limits of the hydraulic study area in each model. These include flow hydrographs at the upstream end of the study area, which represent freshwater inflow to North Mill Pond, and stage hydrographs at the downstream end of the study area to simulate tide fluctuations. 50- and 100-year flow and stage hydrographs were used in the existing conditions and bridge rehabilitation models. These extreme events were used because the rehabilitated bridge is expected to be in service for several years and there is a reasonable probability of experiencing such an infrequent event over its service life. As an example, there is an 18.3% chance that a 50-year event would occur over a 10-year period. By contrast, since the rehabilitation work is only expected to take about three weeks to complete, more frequent, lesser magnitude events, namely 1-year flow hydrographs and 2-year stage hydrographs, were used in the water diversion models.

Runoff hydrographs calculated with the previously described HydroCAD model were used as the upstream boundary condition in the hydraulic models and data from the NOAA Seavey Island tide station (#8419870) were used to develop stage hydrographs for the downstream boundary. The assumed boundary conditions are included in the detailed descriptions of each model below.

C.4. Additional Modeling Parameters

All flood simulations were run with the full momentum SWE-ELM equation set (i.e. Shallow Water Equations, Eulerian-Lagrangian Method) which is appropriate for tidally-influenced conditions as it is capable of modeling the propagation of dynamic tide cycle waves.

Due to the high flows used in the bridge rehabilitation models, the HEC-RAS program was allowed to adjust the computational time step as needed to produce stable model runs with Courant numbers of about one or less to ensure that flow was not propagating through more than one cell at each time step. Due to the high flow velocities, the resulting computational time steps were as short as 1.25 seconds. For the water diversion models which used lesser flows, a uniform computational time step of 5 seconds was used. This was the largest time step found to produce similarly stable model runs with Courant number of approximately one or less.

Hydraulics for the existing bridge were calculated with the energy-based standard step method for low flow conditions (i.e. open channel flow where the water surface is below the highest point of the bridge low chord) and pressure flow (orifice equations) for high flow conditions when the bridge is submerged. The energy-based method was selected as the low flow computational method because there are no piers and this method accounts for friction losses, changes in geometry through the bridge, and losses due to flow transitions and turbulence. Contraction and expansion coefficients of 0.3 and 0.5, respectively, were used in the energy head loss equation. The pressure flow method was used as the high flow computational method because the bridge deck and roadway are significant flow obstructions which create backwater and result in the bridge opening acting like a pressurized orifice.

C.5. Scenario-Specific Modeling Parameters and Results

Hydraulic modeling parameters specific to each scenario, primarily boundary condition assumptions and geometry of the bridge and water diversion structures, and results of each model are described in Sections D through F.

D. Hydraulic Analyses – Existing Conditions

Two separate HEC-RAS 2D flow models were developed for existing conditions – one simulating a 50-year flood occurring coincident with a 50-year tidal storm surge and one simulating a 100-year flood occurring coincident with a 100-year tidal storm surge.

D.1. Boundary Conditions

The 50- and 100-year flood hydrographs calculated with the HydroCAD rainfall runoff model were used as the upstream boundaries in the models (see Figures 3 and 4).

Stage hydrographs representing the tidal storm surge were used as the downstream boundaries. These were developed from water levels measured at NOAA tide station 8419870 at Seavey Island, ME located at the Portsmouth Naval Shipyard about 1.2 miles due east of the bridge (https://tidesandcurrents.noaa.gov/stationhome.html?id=8419870). This tide gage has operated intermittently between 1926 and present with a cumulative record of approximately 57 years. Figure 9 shows the high water level exceedance probability curve generated by NOAA from the gage data.

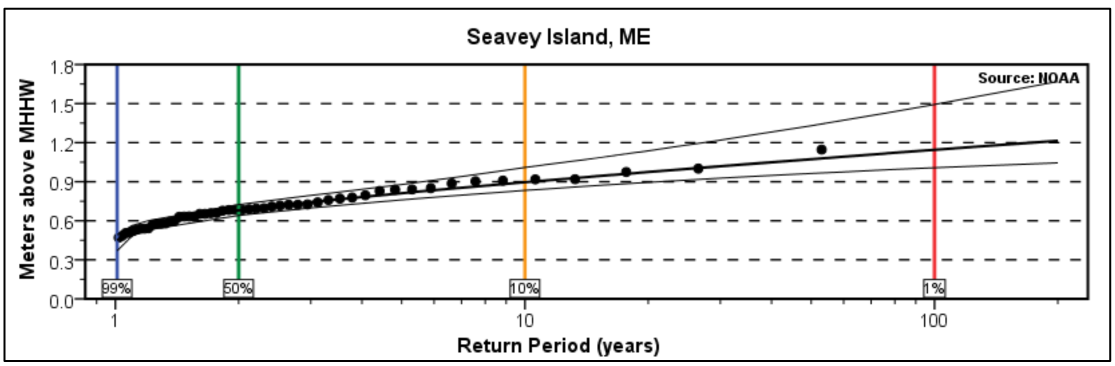


Figure 9 – High water annual exceedance probability curve for NOAA station 8419870 (Seavey Island, ME)

As indicated in Figure 9, the exceedance curve predicts the 100-year high water level is about 1.14 meters (3.74 feet) above mean higher high water (MHHW) and the 50-year high water level is approximately 1.07 meters (3.51 feet) above MHHW. As listed in the tidal datum information for the station, MHHW at the gage for the tidal epoch ending in 2001 is 1.285 meters (4.22 feet) above the North American Vertical Datum of 1988 (NAVD88). Adjusting the exceedence probabiliy water level estimates to fixed elevations relative to NAVD88 results in the following peak tidal storm surge water levels.

Table 6 – Peak tidal storm surge water levels predicted at NOAA station 8419870 (Seavey Island, ME)

Recurrence Interval (years)	Peak Storm Surge Water Level (feet, NAVD88)
50	7.73
100	7.96

Section 3.2 of NHCRHC STAP (2014) suggests that present recurrence intervals of New Hampshire tidal storm surges be basesd upon the preliminary FEMA Flood Insurance Rate Maps (FIRMs) for coastal NH. The prelimary FIRM covering the project area

(#33015C0259F), dated April 9, 2014, shows the Base Flood Elevation (BFE) at elevation 8 feet (NAVD88) (see Figure 10). The effective FIRM, dated January 29, 2021, also shows the BFE at elevation 8. The BFE, which corresponds to the 1% annual chance, or 100-year, flood level, is only 0.04 feet (½ inch) higher than the 100-year peak tidal storm surge water level predicted from the exceedance probability curve for the Seavey Island tide gage.

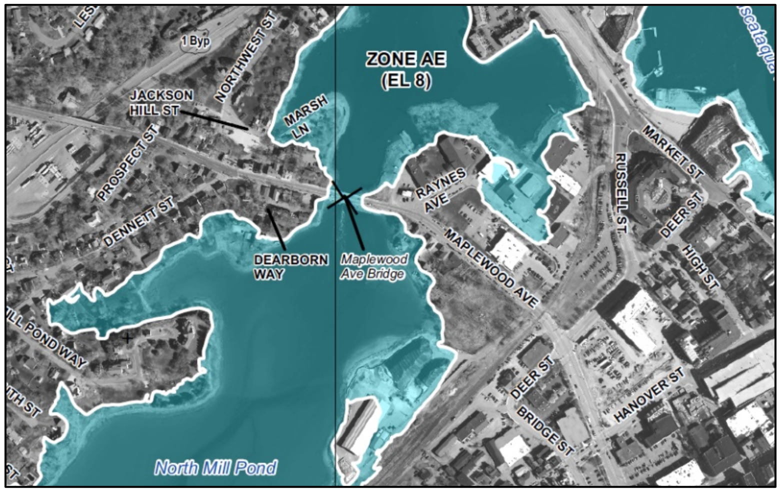


Figure 10 – Preliminary FIRM #33015C0259F

In keeping with the recommendations of NHCRHC STAP (2014), a 100-year peak tidal storm surge elevation of 8 feet was used in the existing conditions 100-year hydraulic model. NHCRHC STAP (2014) does not provide guidance relative to 50-year tidal storm surge water levels and none are published on the FEMA FIRM or in the FEMA Flood Insurance Study (FIS) for Rockingham County. Therefore, the 50-year peak tidal storm surge water level predicted by the exceedance probability curve for the Seavey Island tide gage (7.73 feet) was used in the existing conditions 50-year hydraulic model.

The 50- and 100-year tidal storm surge stage hydrographs used for the downstream boundaries were estimated using water levels measured during the highest tidal storm surge cycle recorded at the Seavey Island gage. This occurred on February 7, 1978 with a peak elevation of 8.06 feet (NAVD88) (see Figure 11), which is 0.33 feet (4 inches) above the estimated 50-year peak tidal storm surge water level and 0.06 feet (¾ inch) above the estimated 100-year peak water level.

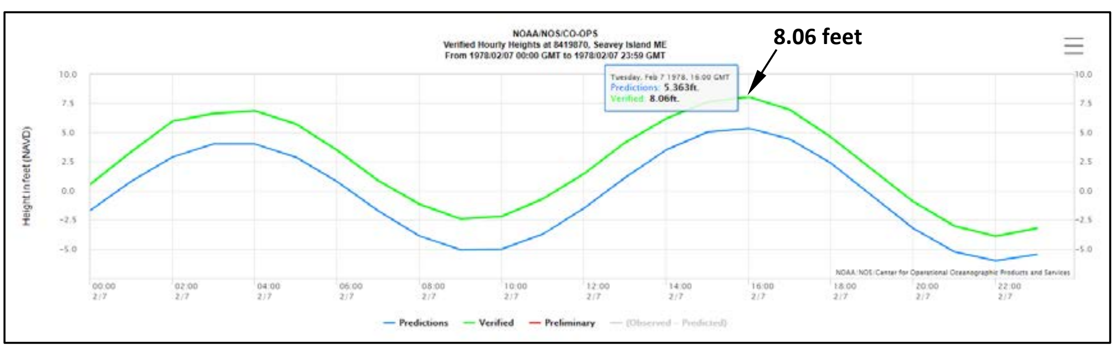


Figure 11 – Stage hydrograph showing water levels measured at the Seavey Island, ME tide gage on February 7, 1978. The green line represents measured water levels and the blue line represents predicted water levels.

Hourly water level data for February 6 through February 8, 1978 were downloaded from the NOAA website. The estimated 50- and 100-year peak tidal storm surge water levels are approximately 95.9% and 99.3% of the peak water level recorded at the gage on February 7,

1978. The measured water levels were multiplied by these percentages to generate the estimated 50- and 100-year tidal storm surge stage hydrographs used as the downstream boundaries in the models.

The 50- and 100-year freshwater inflow hydrographs have a duration of 42 hours with the peak flow occurring at hour 13.5 of the runoff events. The estimated storm surge stage hydrographs were generated so as to have the same 42-hour duration with peak water levels also occurring at hour 13.5. This results in the freshwater inflow hydrographs and the tidal storm surge stage hydrographs peaking concurrently so as to simulate near worst-case scenarios wherein the peak inland runoff enters North Mill Pond at the same time the storm tide reaches its maximum level. Figures 12 and 13 show the estimated 50- and 100-year tidal storm surge stage hydrographs used as the downstream boundaries in the existing conditions models.

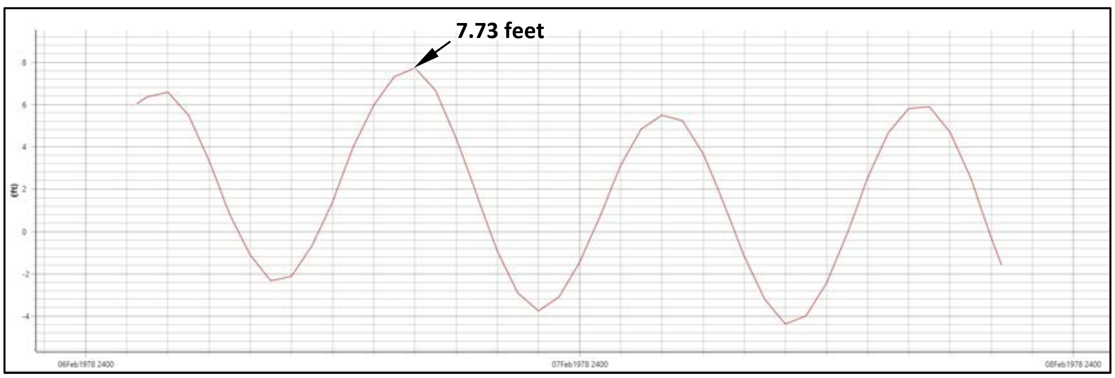


Figure 12 – Estimated 50-year tidal storm surge stage hydrograph

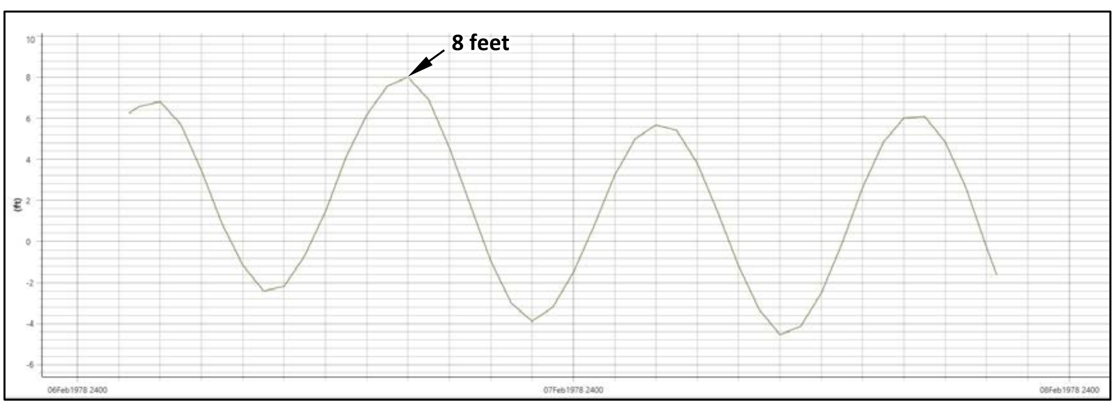


Figure 13 – Estimated 100-year tidal storm surge stage hydrograph

D.2. <u>Bridge Geometry</u>

Figure 14 shows a photo of the existing bridge inlet and Figures 15 and 16 show cross-sections at the existing bridge inlet and outlet. [Note that although there is bi-directional flow through the bridge, for the purposes of our study the bridge inlet is on the south side of Maplewood Avenue and the bridge outlet is on the north side of the road.] Geometries of the metal arch, concrete footings, and channel bottom are based on field survey data collected by Headwaters Consulting, LLC collected in September 2020. The roadway embankment geometries were determined from the Doucet Survey, LLC survey information.

A 24-inch diameter sanitary sewer main passes through the bridge opening about 15 feet south of the bridge outlet (see Figures 14 and 17). The size, location, and elevation of the sewer main were estimated from a 2009 plan by Haight Engineering, PLLC[1] and superimposed on the existing bridge outlet section (Figure 16).

Figure 14 – View north at the existing bridge inlet (09-23-20)

[1] Existing Profile Plan, Maplewood Ave Culvert Replacement & North Mill Pond Restoration, Portsmouth, NH, prepared by Haight Engineering, PLLC, Sheet C-4, date: 12-30-2009

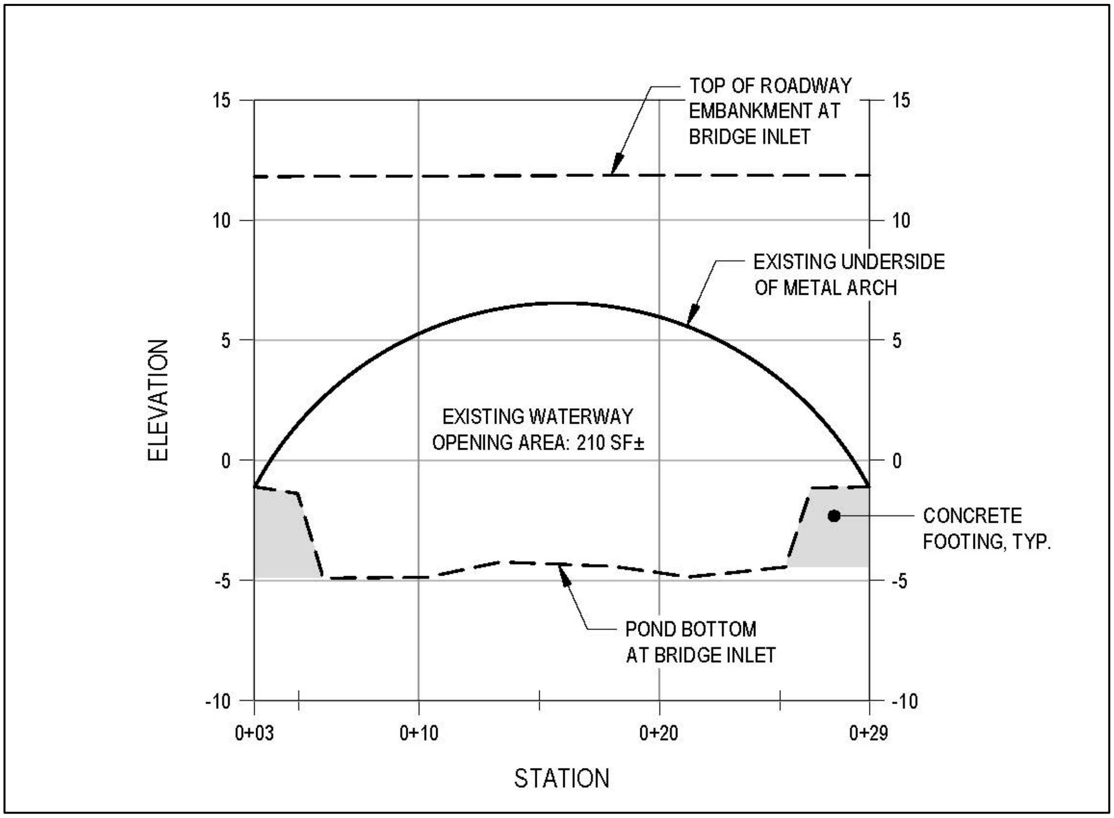


Figure 15 – Existing bridge inlet cross-section

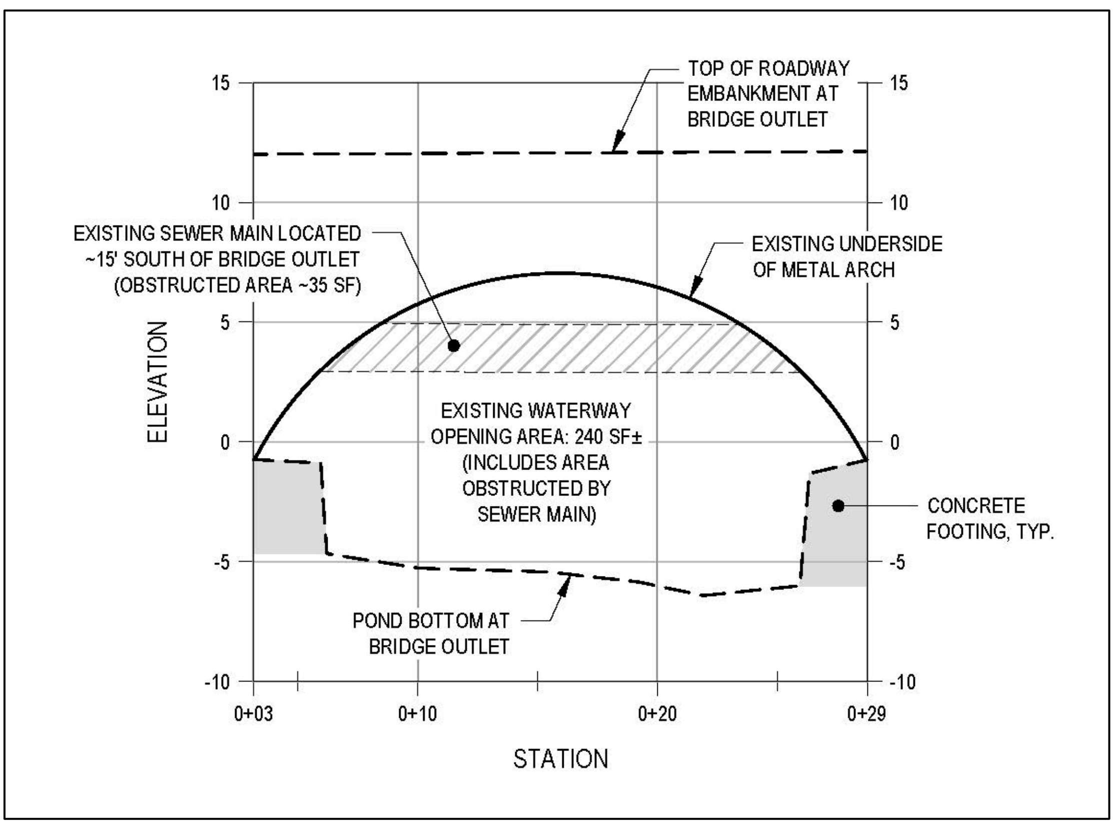


Figure 16 – Existing bridge outlet cross-section

Since the HEC-RAS bridge hydraulics routine computes flow through the bridge only at the inlet and outlet, the true effect of the sewer main cannot be modelled directly. Therefore, in an attempt to estimate its impact, the waterway opening at the bridge outlet was reduced by an area equal to the area obstructed by the sewer main, which is shown to be approximately 35 square feet on the 2009 Haight Engineering plan. Figure 18 shows the bridge outlet section as coded in the existing conditions models to account for the sewer main. The waterway opening area at the bridge outlet is approximately 240 square feet when the sewer main obstruction is disregarded. The modeled waterway opening area at the bridge outlet is about 205 square feet.

Figure 17 – View north within the existing bridge opening showing the sewer main (09-23-20)

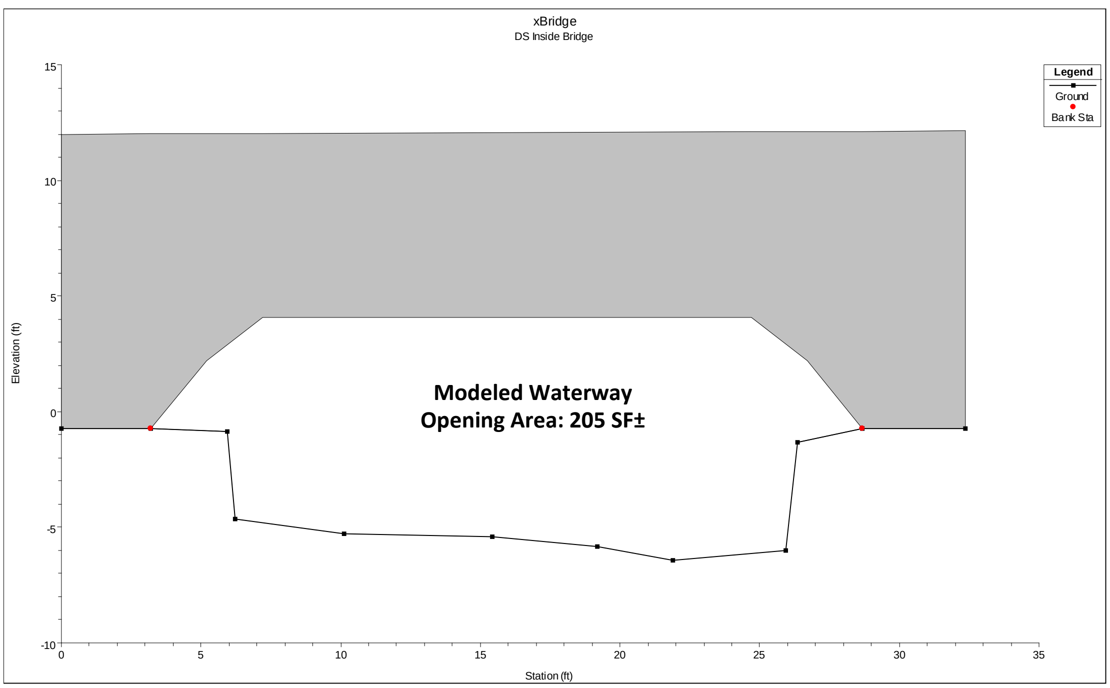


Figure 18 – Existing bridge outlet cross-section as modeled to account for sewer main obstruction

D.3. <u>Results</u>

Table 7 summarizes the peak water levels in North Mill Pond south of Maplewood Avenue calculated with the 50- and 100-year existing conditions models. Note that maximum water levels at the south end of the pond below the outlet of the Bartlett Street culvert are slightly higher (<0.1’) than in the majority of the pond. Similarly, maximum water levels at the bridge inlet are slightly lower (<0.1’) than in the majority of the pond. The peak water levels listed in Table 7, and in subsequent tables which report maximum water levels, have been calculated at the centroid of the portion of North Mill Pond on the south side of Maplewood Avenue and represent the peak water levels in the majority of the waterbody on the south side of the road.

Table 7 – Peak water levels in North Mill Pond on the south side of Maplewood Avenue calculated with existing conditions models

Recurrence Interval (years)	Peak Water Level in North Mill Pond on South Side of Maplewood Ave. Existing Conditions* (feet, NAVD88)
50	7.96
100	8.41

* *calculated at the centroid of the waterbody on the south side of Maplewood Ave. (N 211315, E 1224317)*

Figures 19 and 20 show the areas inundated when water levels calculated with the 50- and 100-year existing conditions models are at their maximum.

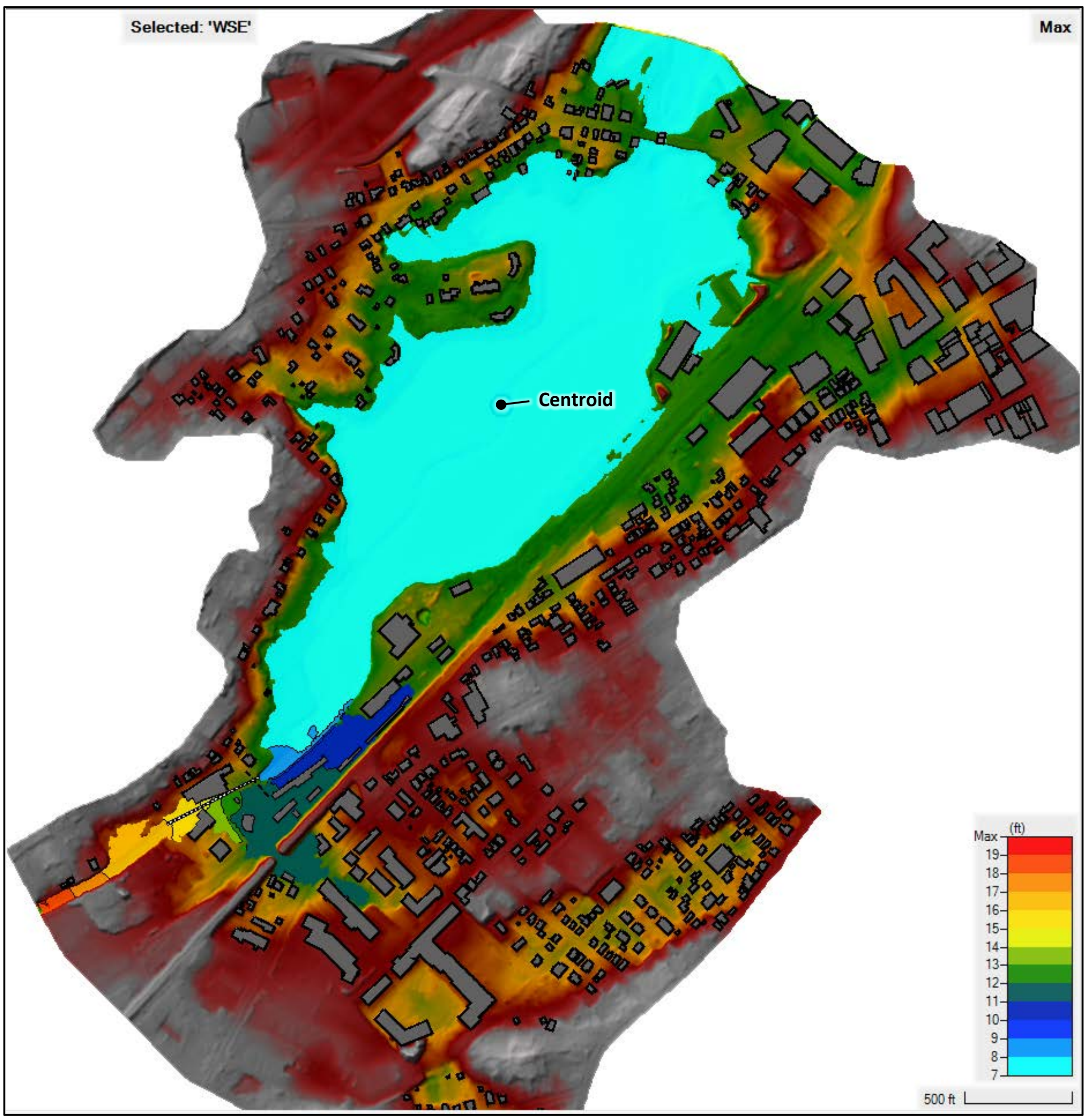


Figure 19 – Existing conditions 50-year inundation map

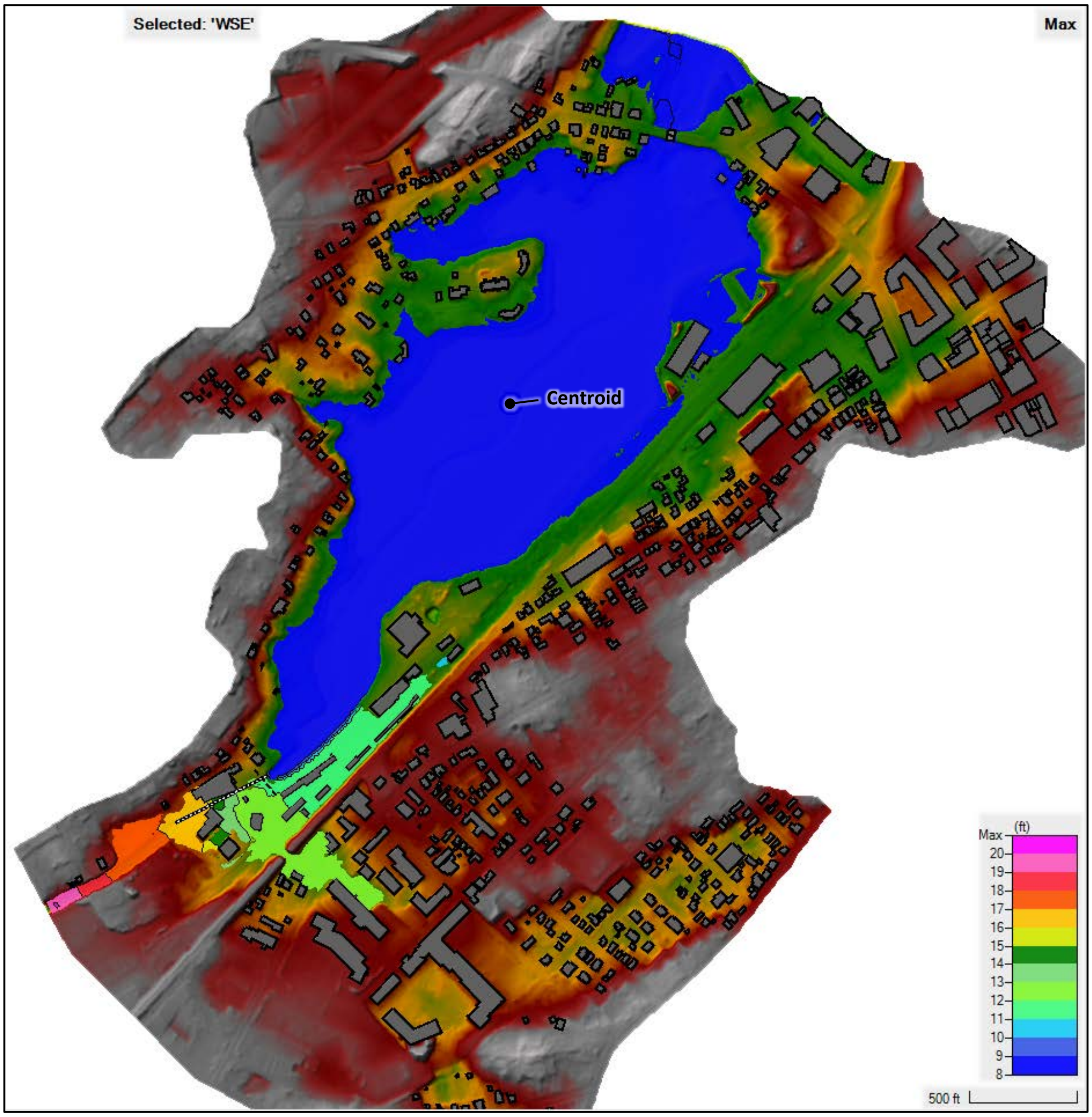


Figure 20 – Existing conditions 100-year inundation map

Figures 21 and 22 show the stage and flow hydrographs at the bridge calculated with the 50- and 100-year existing conditions models. The headwater stage is the water level at the bridge inlet on the south side of Maplewood Avenue and the tailwater stage is the water level at the bridge outlet on the north side of the road. Note that the maximum stage at the bridge inlet at the crest of each tide cycle is more or less equal to the water level at the bridge outlet except at the coincident peak of the freshwater inflow and tidal storm surge when the stage at the inlet is higher due to the freshwater inflow. Also note that due to the flow constriction created by bridge, low water levels in North Mill Pond south of the road at the trough of each tide cycle are higher than, and lag behind, low water levels at the bridge outlet with the greatest differences occurring at the tide cycle trough immediately after the coincident inflow and storm surge peaks.

The maximum flow through the bridge during the 50-year events is 1,874 cfs and occurs about two hours after the coincident inflow and storm surge peaks. The maximum flow through the bridge during the 100-year events is 2,129 cfs and also occurs about two hours after the coincident peaks. Table 8 lists the peak flows through the bridge for existing conditions.

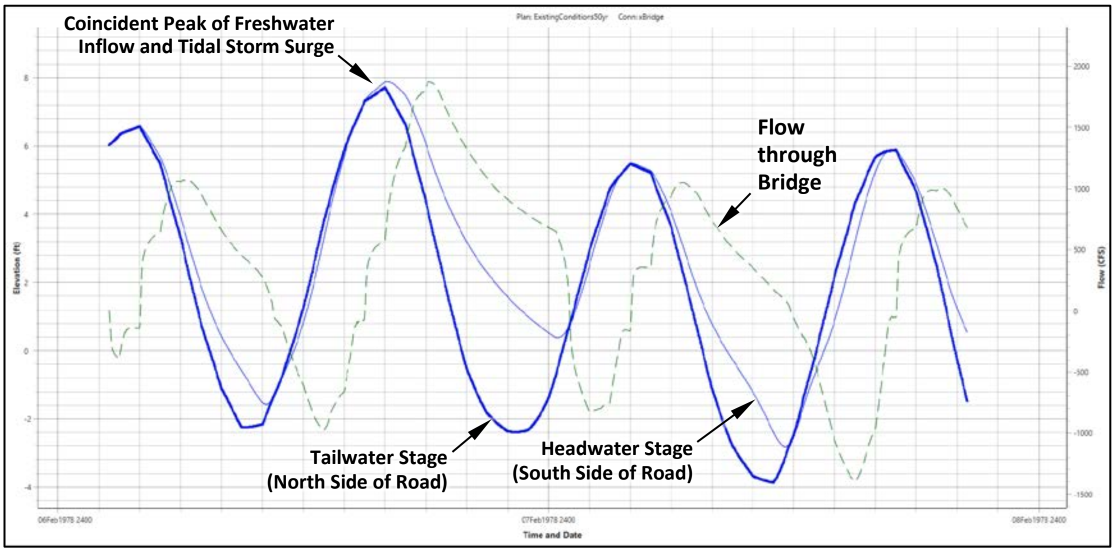


Figure 21 – Existing conditions 50-year stage and flow hydrographs calculated at the bridge

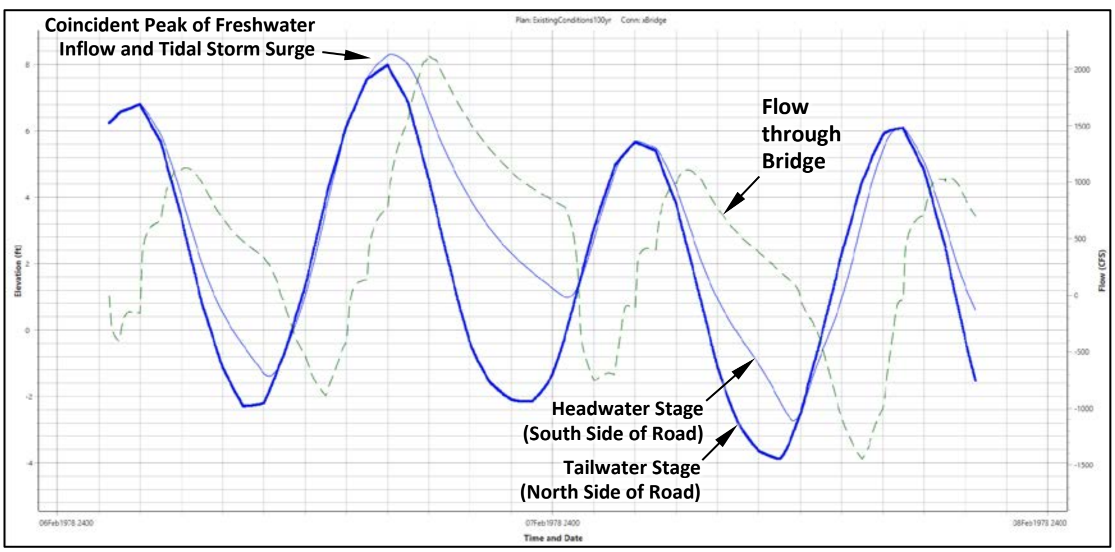


Figure 22 – Existing conditions 100-year stage and flow hydrographs calculated at the bridge

Table 8 – Peak flow through bridge for Existing Conditions

Recurrence Interval (years)	Peak Flow through Bridge – Existing Conditions (cfs)
50	1,874
100	2,129

E. Hydraulic Analyses – Bridge Rehabilitation Alternatives

Three bridge rehabilitation alternatives have been studied as follows:

- *Bridge Rehabilitation Alternative 1: Geopolymer Liner*

  Under this alternative a 4.5-inch thick geopolymer liner would be applied to the underside of the metal arch. No other modifications would be made.

- *Bridge Rehabilitation Alternative 2: Geopolymer Liner and Ancillary Culvert*

  This alternative includes a 4.5-inch thick geopolymer liner on the underside of the metal arch and the addition of one or more culverts through the roadway embankment approximately 60 feet east of the existing bridge. Three culvert configurations were modeled – twin 48-inch and single 60- and 72-inch pipes.

- *Bridge Rehabilitation Alternative 3: Geopolymer Liner and Sewer Main Relocation*

  This alternative includes a 4.5-inch thick geopolymer liner applied to the underside of the metal arch and relocation of the sewer main such that it does not pass through the bridge opening.

Two separate HEC-RAS 2D flow models were developed for each bridge rehabilitation alternative – one simulating a 50-year flood coincident with a 50-year tidal storm surge and one simulating a 100-year flood coincident with a 100-year tidal storm surge. These models are identical to the existing conditions model except for the bridge geometry which was modified to simulate the reduced waterway opening after application of the geopolymer liner and, for Alternatives 2 and 3, installation of a new culvert through the roadway embankment and removal of the sewer main from the waterway opening, respectively.

E.1. Bridge Rehabilitation Alternative 1 – Geopolymer Liner

Figure 23 shows a cross-section of the bridge inlet as modeled under Bridge Rehabilitation Alternative 1. The existing waterway opening area at the inlet is approximately 210 square feet (see Figure 15). The geopolymer liner would occupy approximately 11 square feet, reducing the opening area at the inlet to about 199 square feet.

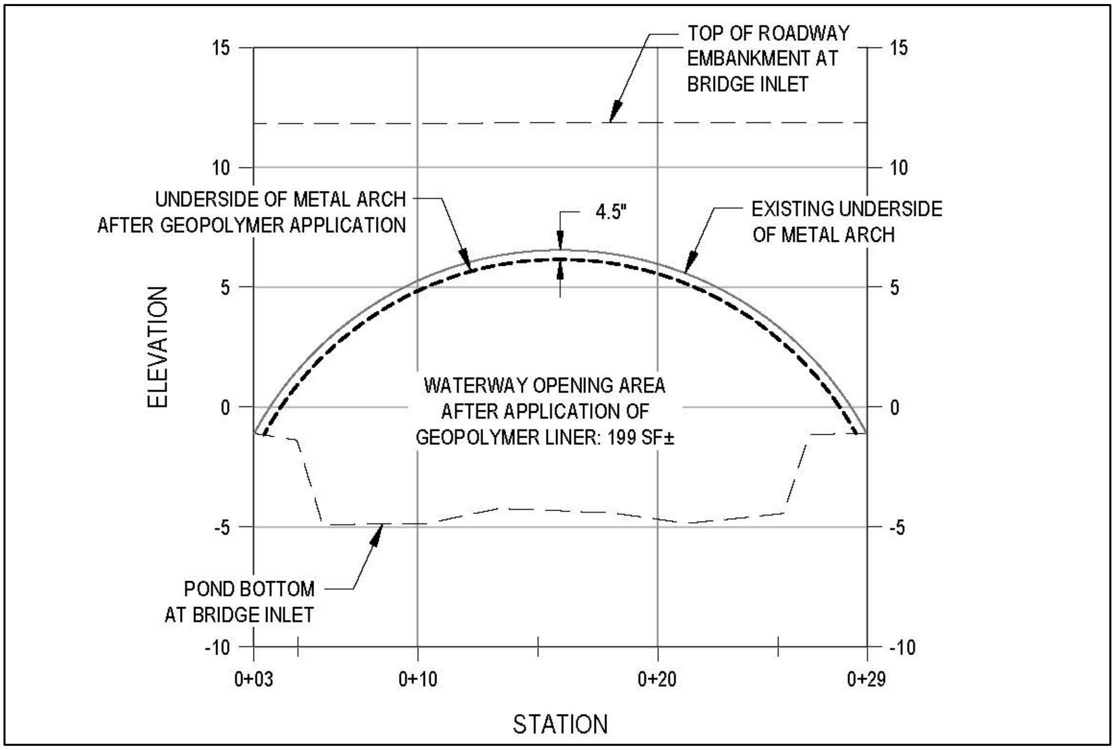


Figure 23 – Bridge Rehabilitation Alternative 1 inlet cross-section

The waterway opening at the bridge outlet was reduced by an area equal to the sum of the areas obstructed by the geopolymer liner and sanitary sewer main. Figure 24 shows the bridge outlet section defined in the Bridge Rehabilitation Alternative 1 hydraulic models to account for these obstructions which have a cumulative area of approximately 45 square feet and reduce the waterway opening area at the bridge outlet from 240 square feet to about 195 square feet.

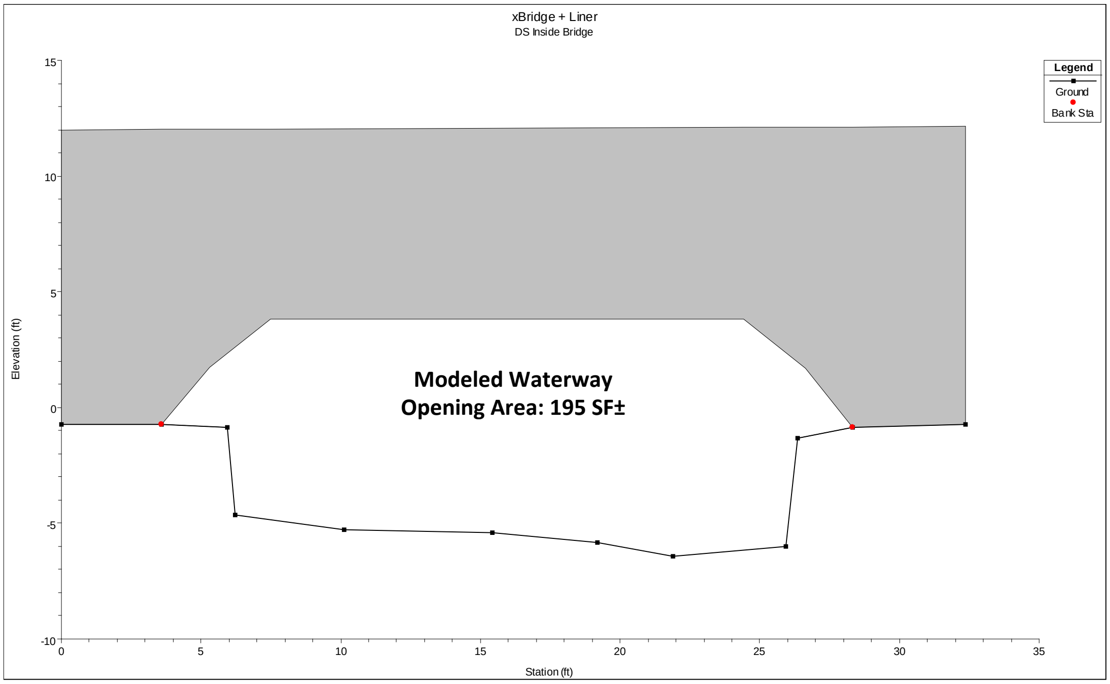


Figure 24 – Bridge Rehabilitation Alternative 1 bridge outlet cross-section as modeled to account for geopolymer liner and sewer main obstructions

Table 9 summarizes the peak water levels in North Mill Pond south of the road calculated with the 50- and 100-year Bridge Rehabilitation Alternative 1 models and the change from existing conditions.

Table 9 – Peak water levels in North Mill Pond on the south side of Maplewood Avenue for Bridge Rehabilitation Alternative 1

Recurrence Interval (years)	Peak Water Level in North Mill Pond on South Side of Maplewood Ave. Bridge Rehabilitation Alternative 1* (feet, NAVD88)	Change from Existing (feet)
50	7.98	0.02
100	8.45	0.04

* *calculated at the centroid of the waterbody on the south side of Maplewood Ave. (N 211315, E 1224317)*

As compared to existing conditions, maximum water levels in the pond on the south side of the road would increase by 0.02 feet for the 50-year events and by 0.04 feet for the 100-year events.

Figures 25 and 26 show the stage and flow hydrographs at the bridge calculated with the 50- and 100-year Bridge Rehabilitation Alternative 1 models. The peak flow through the bridge during the 50-year events is 1,854 cfs and occurs about two hours after the coincident inflow and storm surge peaks. The maximum flow through the bridge during the 100-year events is 2,062 cfs and also occurs about two hours after the coincident peaks. Table 10 lists these peak flows and the change from existing conditions.

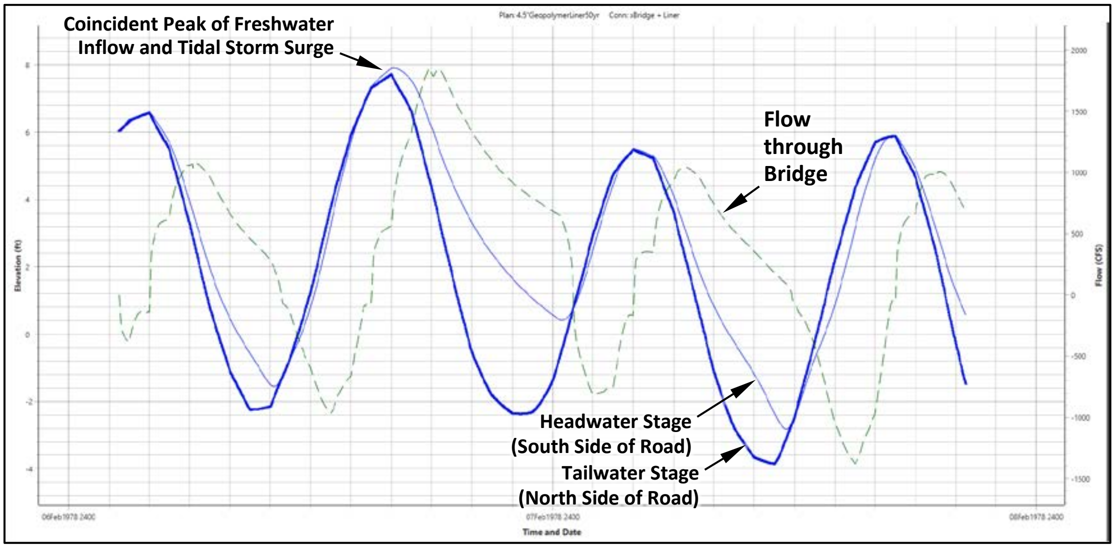


Figure 25 – 50-year stage and flow hydrographs calculated at the bridge for Bridge Rehabilitation Alternative 1

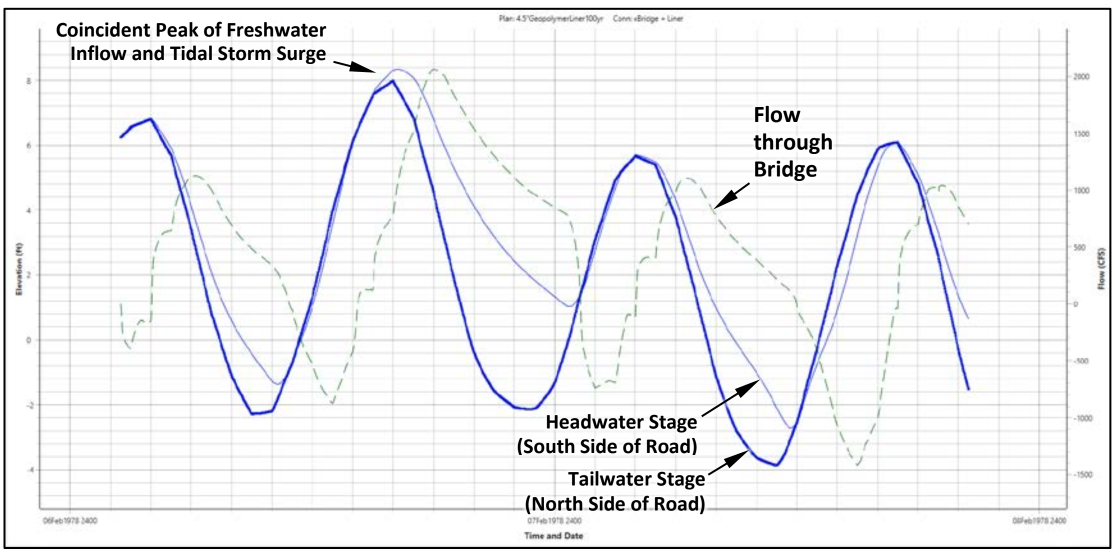


Figure 26 – 100-year stage and flow hydrographs calculated at the bridge for Bridge Rehabilitation Alternative 1

Table 10 – Peak flow through bridge for Bridge Rehabilitation Alternative 1

Recurrence Interval (years)	Peak Flow through Bridge Bridge Rehabilitation Alternative 1 (cfs)	Change from Existing (cfs)
50	1,854	-20
100	2,062	-67

E.2. Bridge Rehabilitation Alternative 2 – Geopolymer Liner and Ancillary Culvert

Three ancillary culvert configurations were evaluated in combination with the bridge, which was modeled with the geopolymer liner applied as described under Bridge Rehabilitation Alternative 1. These included twin 48-inch diameter pipes and single 60- and 72-inch diameter pipes. The west edges of the culvert barrels were assumed to be located sixty feet from the east edge of the existing bridge opening so as to minimize the potential for the culverts to interfere with the future bridge replacement. All culvert invert elevations at both the inlet and outlet ends were modeled at elevation -4.0 (NAVD88). This is about one foot higher than the pond bottom at the bridge inlet and approximately 0.6 foot lower than the bedrock grade control on the pond bottom about eighteen feet south of the bridge inlet. At this elevation the ancillary culverts are expected to be effective at conveying flows at approximately the same tidal range as the bridge. In addition, based on the sewer profile shown on the 2009 Existing Profile Plan by Haight Engineering, it appears that installing the culverts at this elevation would avoid conflicts with the existing sewer main. Approximately 130-foot long culverts would be needed at the assumed installation locations to daylight on the pond bottom at elevation -4.0. All culverts were assumed to be installed so as to be projecting from the embankment without headwalls or other end treatments and modeled with an entrance loss coefficient of 0.9. The addition of end treatments which create smoother flow transition would increase the culvert discharge capacities. All culverts were modeled with a Manning's n roughness coefficient of 0.012 to represent a smooth interior surface typical of dual wall HDPE and precast concrete pipe.

E.2.1. Bridge Rehabilitation Alternative 2.1 – Geopolymer Liner and Twin 48-inch Diameter Culverts

Figure 27 shows a schematic plan view of the ancillary culvert configuration analyzed under Bridge Rehabilitation Alternative 2.1. The twin 48-inch pipes would add approximately 25 square feet of waterway opening area, which is greater than the area displaced by the geopolymer liner (11 square feet).

Table 11 summarizes the peak water levels in North Mill Pond on the south side of Maplewood Avenue calculated with the 50- and 100-year hydraulic models for this alternative and the change from existing conditions.

Table 11 – Peak water levels in North Mill Pond on the south side of Maplewood Avenue for Bridge Rehabilitation Alternative 2.1

Recurrence Interval (years)	Peak Water Level in North Mill Pond on South Side of Maplewood Ave. Bridge Rehabilitation Alternative 2.1* (feet, NAVD88)	Change from Existing (feet)
50	7.94	- 0.02
100	8.38	- 0.03

* *calculated at the centroid of the waterbody on the south side of Maplewood Ave. (N 211315, E 1224317)*

As compared to existing conditions, maximum water levels on the south side of the road would decrease by 0.02 feet for the 50-year events and by 0.03 feet for the 100-year events.

Maximum combined flows through the bridge and culverts during the 50- and 100-year events are 1,940 cfs and 2,228 cfs, respectively, and occur about two hours after the coincident inflow and storm surge peaks. Table 12 lists the individual flows for each hydraulic structure, the peak combined flows, and the change from existing conditions. Note that the peak flows for the 48-inch culverts listed in Table 12 occur when the combined flows are at their maximum, which coincides with peak flows through the bridge. Maximum flows through the culverts occur about 1.5 hours after the peak combined flows and are 246 cfs and 269 cfs for the 50- and 100-year events, respectively.

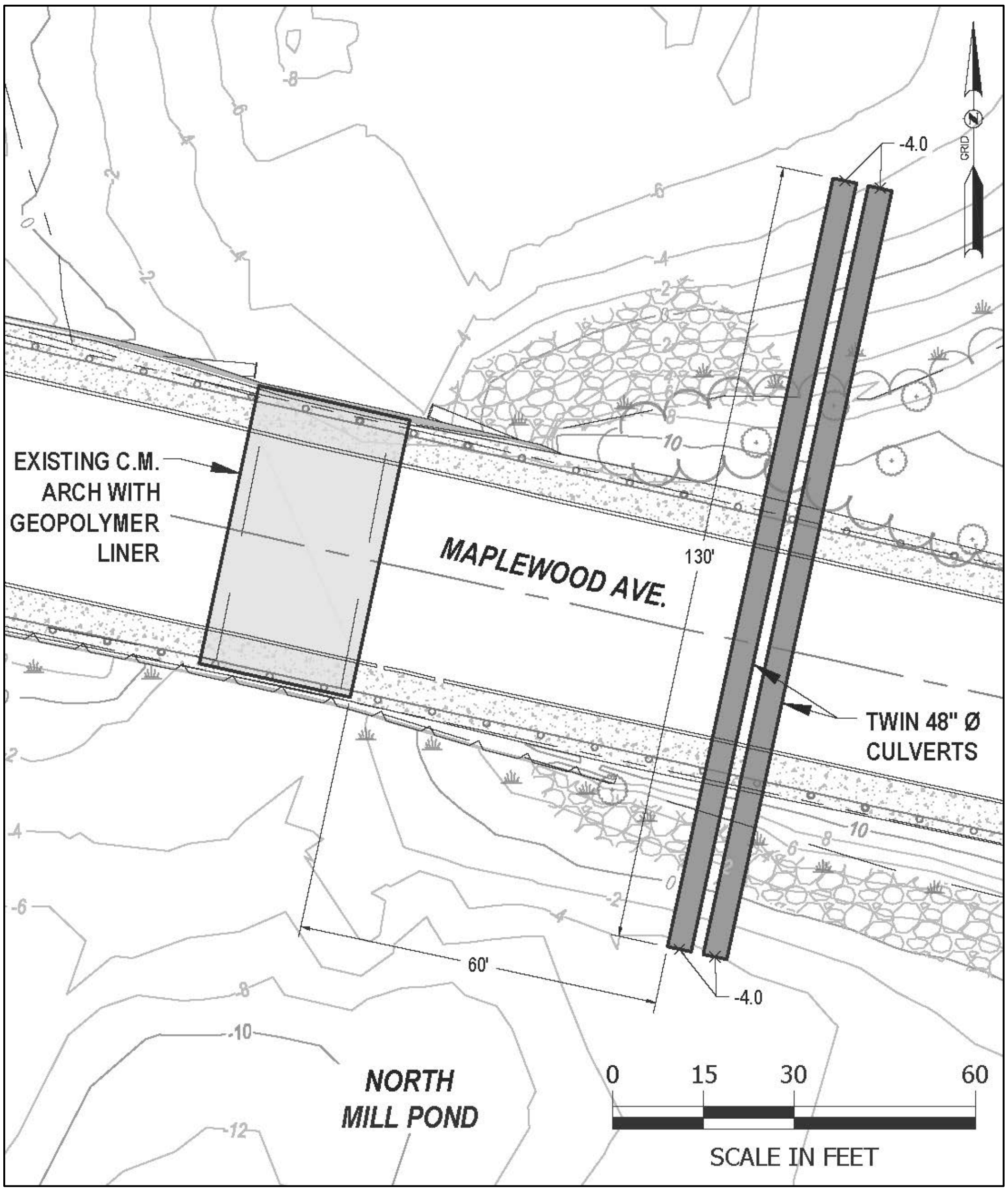


Figure 27 – Bridge Rehabilitation Alternative 2.1 schematic plan

Table 12 – Peak flows through bridge and 48" culverts for Bridge Rehabilitation Alternative 2.1

Recurrence Interval (years)	Peak Flows – Bridge Rehabilitation Alternative 2.1 (cfs)			Change from Existing (cfs)
	Bridge	48" Culverts*	Combined	
50	1,748	192	1,940	66
100	2,022	206	2,228	99

* *culvert flows listed occur when the combined flows are at their maximum*

E.2.2. Bridge Rehabilitation Alternative 2.2 – Geopolymer Liner and Single 60-inch Diameter Culvert

Figure 28 shows a schematic plan view of the ancillary culvert configuration analyzed under Bridge Rehabilitation Alternative 2.2. The 60-inch pipe would add approximately 20 square feet of waterway opening area, which is greater than the area obstructed by the geopolymer liner (11 square feet).

Table 13 summarizes the peak water levels in North Mill Pond on the south side of Maplewood Avenue calculated with the 50- and 100-year hydraulic models for this alternative and the change from existing conditions.

Table 13 – Peak water levels in North Mill Pond on the south side of Maplewood Avenue for Bridge Rehabilitation Alternative 2.2

Recurrence Interval (years)	Peak Water Level in North Mill Pond on South Side of Maplewood Ave. Bridge Rehabilitation Alternative 2.2* (feet, NAVD88)	Change from Existing (feet)
50	7.94	- 0.02
100	8.39	- 0.02

* *calculated at the centroid of the waterbody on the south side of Maplewood Ave. (N 211315, E 1224317)*

As compared to existing conditions, maximum water levels on the south side of the road would decrease by 0.02 feet for both the 50- and 100-year events.

Maximum combined flows through the bridge and culvert during the 50- and 100-year events are 1,924 cfs and 2,217 cfs, respectively, and occur about 2.2 hours after the coincident inflow and storm surge peaks. Table 14 lists the individual flows for each hydraulic structure, the peak combined flows, and the change from existing conditions. Note that the peak flows for the 60-inch culvert listed in Table 14 occur when the combined flows are at their maximum, which coincides with peak flows through the bridge. Maximum flows through the culvert occur about 1.2 hours after the peak combined flows and are 190 cfs and 208 cfs for the 50- and 100-year events, respectively.

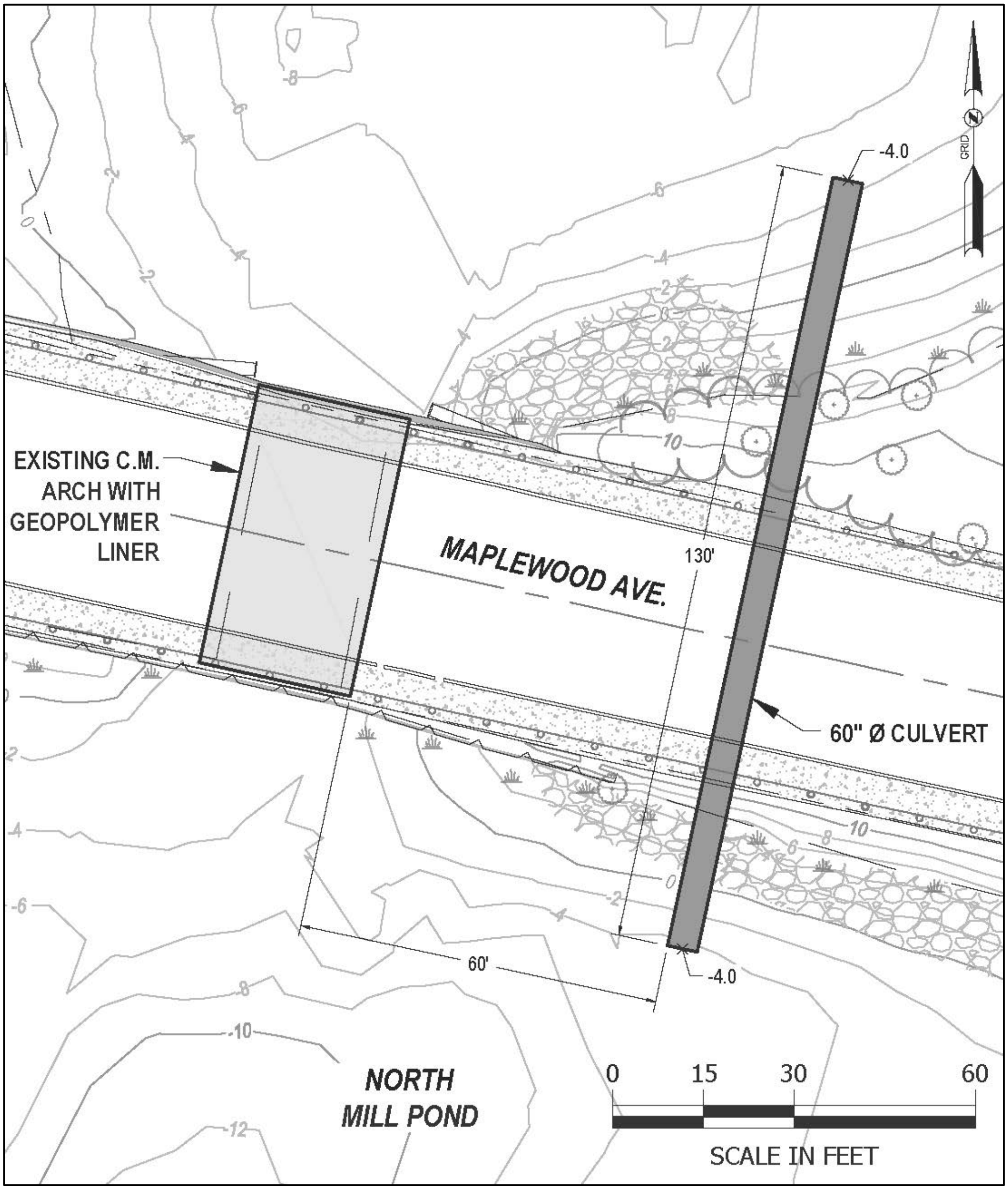


Figure 28 – Bridge Rehabilitation Alternative 2.2 schematic plan

Table 14 – Peak flows through bridge and 60" culvert for Bridge Rehabilitation Alternative 2.2

Recurrence Interval (years)	Peak Flows – Bridge Rehabilitation Alternative 2.2 (cfs)			Change from Existing (cfs)
	Bridge	60" Culvert*	Combined	
50	1,768	156	1,924	50
100	2,049	168	2,217	88

* *culvert flows listed occur when the combined flows are at their maximum*

E.2.3. Bridge Rehabilitation Alternative 2.3 – Geopolymer Liner and Single 72-inch Diameter Culvert

Figure 29 shows a schematic plan view of the ancillary culvert configuration analyzed under Bridge Rehabilitation Alternative 2.3. The 72-inch pipe would add approximately 28 square feet of waterway opening area, which is greater than the area obstructed by the geopolymer liner (11 square feet).

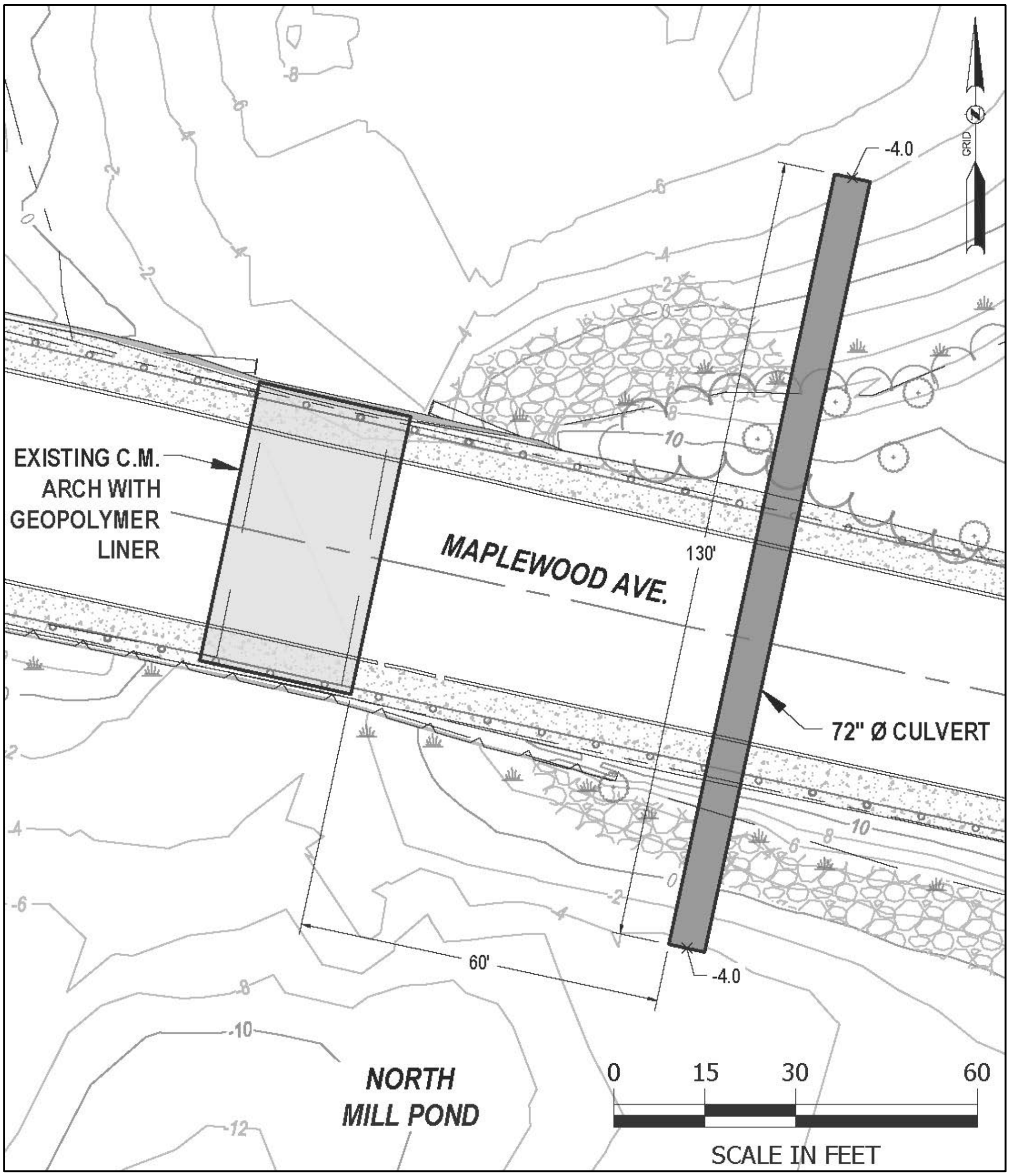


Figure 29 – Bridge Rehabilitation Alternative 2.3 schematic plan

Table 15 summarizes the peak water levels in North Mill Pond on the south side of Maplewood Avenue calculated with the 50- and 100-year hydraulic models for this alternative and the change from existing conditions.

Table 15 – Peak water levels in North Mill Pond on the south side of Maplewood Avenue for Bridge Rehabilitation Alternative 2.3

Recurrence Interval (years)	Peak Water Level in North Mill Pond on South Side of Maplewood Ave. Bridge Rehabilitation Alternative 2.3* (feet, NAVD88)	Change from Existing (feet)
50	7.93	- 0.03
100	8.37	- 0.04

* *calculated at the centroid of the waterbody on the south side of Maplewood Ave. (N 211315, E 1224317)*

As compared to existing conditions, maximum water levels on the south side of the road would decrease by 0.03 feet for the 50-year events and by 0.04 feet for the 100-year events.

Maximum combined flows through the bridge and culvert during the 50- and 100-year events are 1,956 cfs and 2,217 cfs, respectively, and occur about two hours after the coincident inflow and storm surge peaks. Table 16 lists the individual flows for each hydraulic structure, the peak combined flows, and the change from existing conditions. Note that the peak flows for the 60-inch culvert listed in Table 16 occur when the combined flows are at their maximum, which coincides with peak flows through the bridge. Maximum flows through the culvert occur about 0.8 hours after the peak combined flows and are 256 cfs and 283 cfs for the 50- and 100-year events, respectively.

Table 16 – Peak flows through bridge and 72" culvert for Bridge Rehabilitation Alternative 2.3

Recurrence Interval (years)	Peak Flows – Bridge Rehabilitation Alternative 2.3 (cfs)			Change from Existing (cfs)
	Bridge	72" Culvert*	Combined	
50	1,731	225	1,956	82
100	1,976	241	2,217	88

* *culvert flows listed occur when the combined flows are at their maximum*

E.3. <u>Bridge Rehabilitation Alternative 3 – Geopolymer Liner and Sewer Main Relocation</u>

For this alternative the bridge was modeled with the 4.5-inch geopolymer liner applied and the sanitary sewer main removed from the waterway opening. The bridge inlet geometry used in the model was the same as for Bridge Rehabilitation Alternative 1 (see Figure 23). The bridge outlet geometry, however, was coded so as to account for the reduced waterway opening area resulting from the geopolymer liner, but not the sewer main. Figure 30 shows a cross-section of the bridge outlet as modeled under this alternative. As compared to existing conditions, application of the geopolymer liner and removal of the sewer main results in a net increase in the modeled waterway opening area at the bridge outlet from 205 square feet (see Figure 18) to 229 square feet; however, there would still be a net decrease in the waterway opening area at the bridge inlet from 210 square feet to 199 square feet (see Figures 15 and 23).

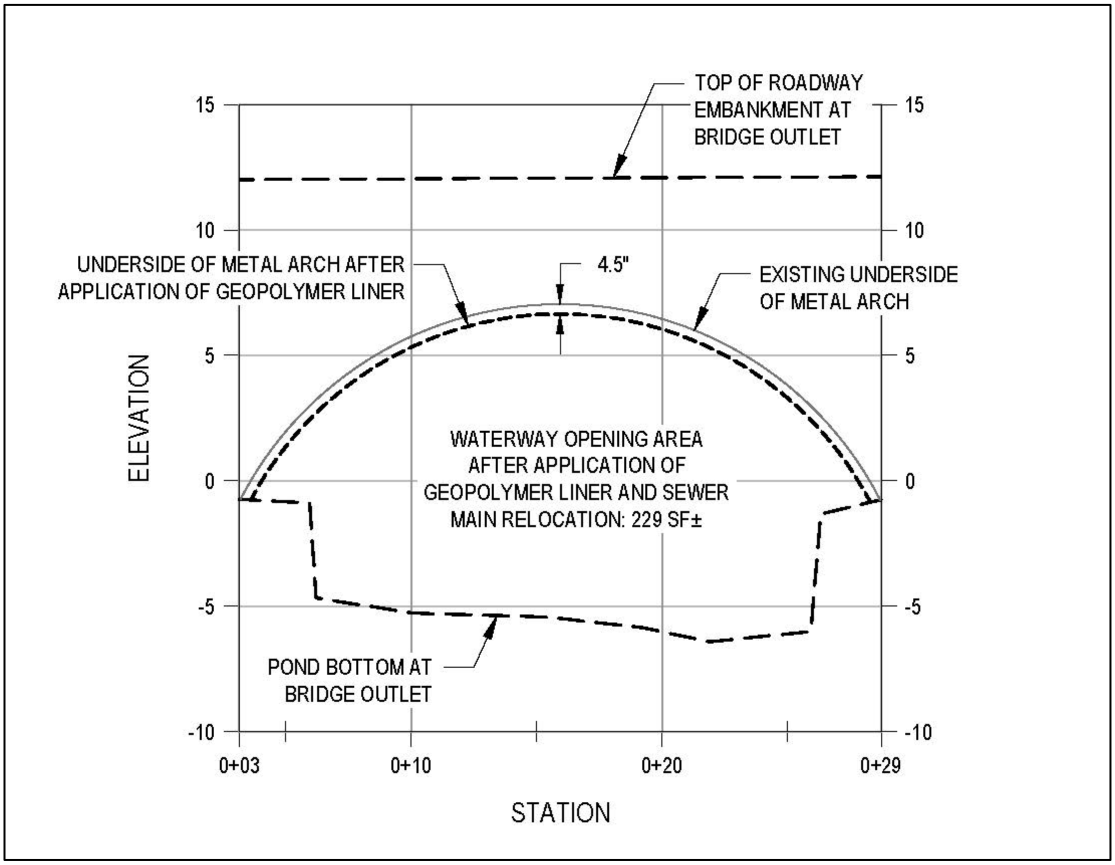


Figure 30 – Bridge Rehabilitation Alternative 3 outlet cross-section

Table 17 summarizes the peak water levels in North Mill Pond on the south side of Maplewood Avenue calculated with the 50- and 100-year hydraulic models for Bridge Rehabilitation Alternative 3 and the change from existing conditions.

Table 17 – Peak water levels in North Mill Pond on the south side of Maplewood Avenue for Bridge Rehabilitation Alternative 3

Recurrence Interval (years)	**Peak Water Level in North Mill Pond on South Side of Maplewood Ave. Bridge Rehabilitation Alternative 3* (feet, NAVD88)**	**Change from Existing (feet)**
50	7.97	0.01
100	8.44	0.03

* *calculated at the centroid of the waterbody on the south side of Maplewood Ave. (N 211315, E 1224317)*

As compared to existing conditions, maximum water levels on the south side of the road would increase by 0.01 feet for the 50-year events and by 0.03 feet for the 100-year events. These water level increases are less than those calculated for Bridge Rehabilitation Alternative 1 (i.e. addition of the geopolymer liner with the sewer main to remain), but are still greater than existing conditions. This indicates that removal of sewer main would not fully compensate for the area obstructed by the geopolymer liner, particularly at the bridge inlet where there would still be a net decrease in the waterway opening area.

The peak flow through the bridge during the 50-year events is 1,878 cfs and occurs about two hours after the coincident inflow and storm surge peaks. The maximum flow through the bridge during the 100-year events is 2,054 cfs and also occurs about two hours after the coincident peaks. Table 18 lists these peak flows and the change from existing conditions.

Table 18 – Peak flow through bridge for Bridge Rehabilitation Alternative 3

Recurrence Interval (years)	Peak Flow through Bridge Bridge Rehabilitation Alternative 1 (cfs)	Change from Existing (cfs)
50	1,878	4
100	2,054	-75

F. Hydraulic Analyses – Water Diversion Alternatives

Four alternatives for dewatering the site during installation of the geopolymer liner have been studied. These are as follows:

- *Water Diversion Alternative 1: Tidal Flow Exclusion and Freshwater Storage*

  Under this alternative cofferdams would be installed on both sides of the crossing but there would be no hydraulic connection between the waterbodies on the north and south sides of the road during construction. The elevation of the cofferdam on the north side of the crossing would be above the anticipated high tide level so as to minimize the potential for tidal flows to overtop it and enter the bridge opening. The cofferdam on the south side of the crossing would be installed at the trough of the low tide wave so as to maximize the Pond volume available to store freshwater inflow from Hodgson Brook and its height would be set so as to maximize storage volume without flooding nearby properties.

- *Water Diversion Alternative 2: Temporary Culverts*

  This alternative entails cofferdams on both sides of the crossing with temporary culverts placed between the cofferdams and through the existing bridge opening to allow tidal and freshwater flows between the waterbodies on the north and south sides of the road during construction. Elevations of the cofferdams would be above anticipated high tide and rainfall-driven flood levels in order to minimize the potential for flows overtopping them and entering the bridge opening. Two temporary culvert configurations were evaluated. Alternative 2.1 includes three 48-inch diameter culverts between the cofferdams and Alternative 2.2 includes two 72-inch culverts.

- *Water Diversion Alternative 3: Phased Water Diversion*

  Under this alternative a temporary cofferdam oriented parallel to the flow would be erected within the existing bridge opening and would connect to cofferdams oriented parallel to the roadway upstream and downstream from the crossing such that slightly more than one-half of the waterway opening would be dewatered and the remaining portion would remain open to convey tidal and freshwater flows. The geopolymer liner would be applied to the dewatered portion of the metal arch and then the cofferdams would be modified so as to dewater the unlined portion of the structure and divert flows through the lined portion so that application of the geopolymer liner can be completed.

- *Water Diversion Alternative 4: Permanent Culvert*

  This alternative involves construction of a permanent 60-inch diameter culvert through the roadway embankment 60 feet east of the existing bridge to carry tidal and freshwater flows during and after the temporary repair project. Cofferdams would be installed on both sides of the crossing during installation of the geopolymer liner to dewater the metal arch. In the future, this culvert could potentially be used to divert flows during construction of the full bridge replacement.

A two-dimensional (2D) HEC-RAS unsteady flow model was developed for each water diversion alternative. With the exception of geometry in the vicinity of the bridge, which was modified to represent the proposed cofferdams and culverts, and boundary conditions, which were altered to reflect the assumed freshwater inflow and tide cycles during construction, the geometry and modeling parameters used in these models were the same as those used in the bridge rehabilitation models.

Assumptions made in evaluating the water diversion alternatives included: cofferdam elevations, culvert sizes and elevations, construction duration and season, tide cycles and elevations, base flow in Hodgson Brook, and the number and magnitude of rainfall events. These assumptions are presented in the detailed descriptions of each water diversion alternative.

Based on the LiDAR DEM, most of the buildings surrounding North Mill Pond on the south side of Maplewood Avenue are above the cofferdam elevation of 6.5 feet assumed in the water diversion alternatives; however, a few structures seem to be very close to this elevation. One of these is the southerly building on Tax Map 124, Lot 7 located about 275 feet southwest of the bridge inlet (see Figure 31). This structure appears to be near or even below elevation 6.5 feet. We have no field survey information for this building, but per the Doucet survey, the top of the retaining wall along the shoreline adjacent to the structure is as low as elevation 5.0 feet.

Figure 31 – View southwest from bridge inlet toward the southerly building on Lot 124-7 (09-23-20)

Another low-lying structure is located on Tax Map 123, Lot 1 about 100 feet west-northwest of the bridge inlet (see Figure 32). The foundation of this structure forms the Pond shoreline and is regularly inundated; however, the three lower window openings may not be intended to be submerged. The Doucet survey included elevations for these window openings, which are all slightly above elevation 6.3 feet.

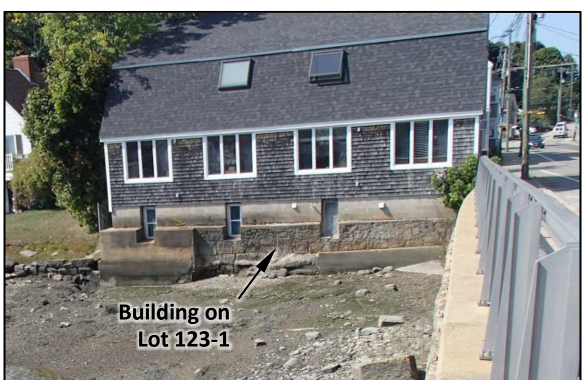


Figure 32 – View west-northwest from bridge inlet toward the building on Lot 123-1 (09-23-20)

If the cofferdams are set higher than structures which are not intended to be submerged, they could potentially backup or trap water to an elevation which floods them. Therefore, the assumed cofferdam elevations used in this study are not necessarily the recommended elevations.

The elevations of the cofferdams which are ultimately installed during construction should be determined once the elevation of the lowest structure on North Mill Pond that is not intended to be flooded has been confirmed and the predicted tide stage hydrograph at the Seavey Island gage for the construction period has been published on the NOAA webpage. NOAA publishes predicted stage hydrographs 31 days into the future. The cofferdam elevations should be the minimum needed to prevent overtopping by tidal and freshwater flows, but in no case should they be so high as to flood structures which are not intended to be submerged.

F.1. Water Diversion Alternative 1 – Tidal Flow Exclusion and Freshwater Storage

Figure 34 shows a schematic plan view of the cofferdams installed under Water Diversion Alternative 1. Assumptions made for this alternative are as follows:

- Top elevation of north cofferdam: 6.5 ft (NAVD88)
- Top elevation of south cofferdam: 6.5 ft (NAVD88)
- Construction duration: 21 days
- Construction month: October
- Highest high tide elevation: 2-year high water level (6.42 ft, NAVD88)
- Water level in North Mill Pond on the south side of Maplewood Avenue at time of south cofferdam installation: -4.19 ft (NAVD88)
- Hodgson Brook base flow: 1 cfs
- Rainfall: Two 1-year, 24-hour rainfall events during construction (5.32” total rainfall depth)

The assumed elevation of the north cofferdam (6.5 ft) was set slightly higher than the assumed highest high tide elevation (6.42 ft) so that it would not be overtopped by water levels on the north side of the road. The assumed elevation of the south cofferdam (6.5 ft) was set to match the north cofferdam elevation.

The 21-day construction duration estimate and October construction timeframe were based on discussions with Hoyle, Tanner & Associates (HTA).

The 2-year high water level is approximately 0.67 meters above mean higher high water (MHHW) per the NOAA high water annual exceedance probability curve for the Seavey Island tide gage (see Figure 33). This high water level converts to 6.42 feet relative to the North American Vertical Datum of 1988 (NAVD88) and would have an approximately 4% chance of being equaled or exceeded during any 21-day period.

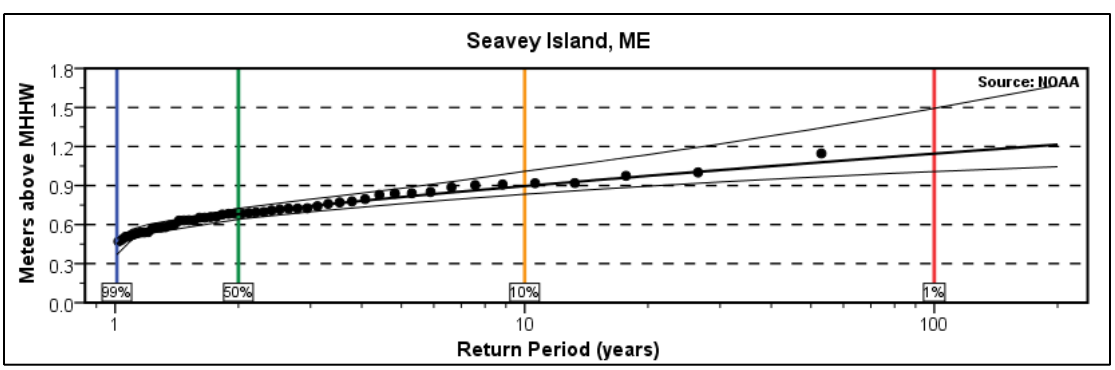


Figure 33 – NOAA high water annual exceedance probability curve for tide gage #8419870

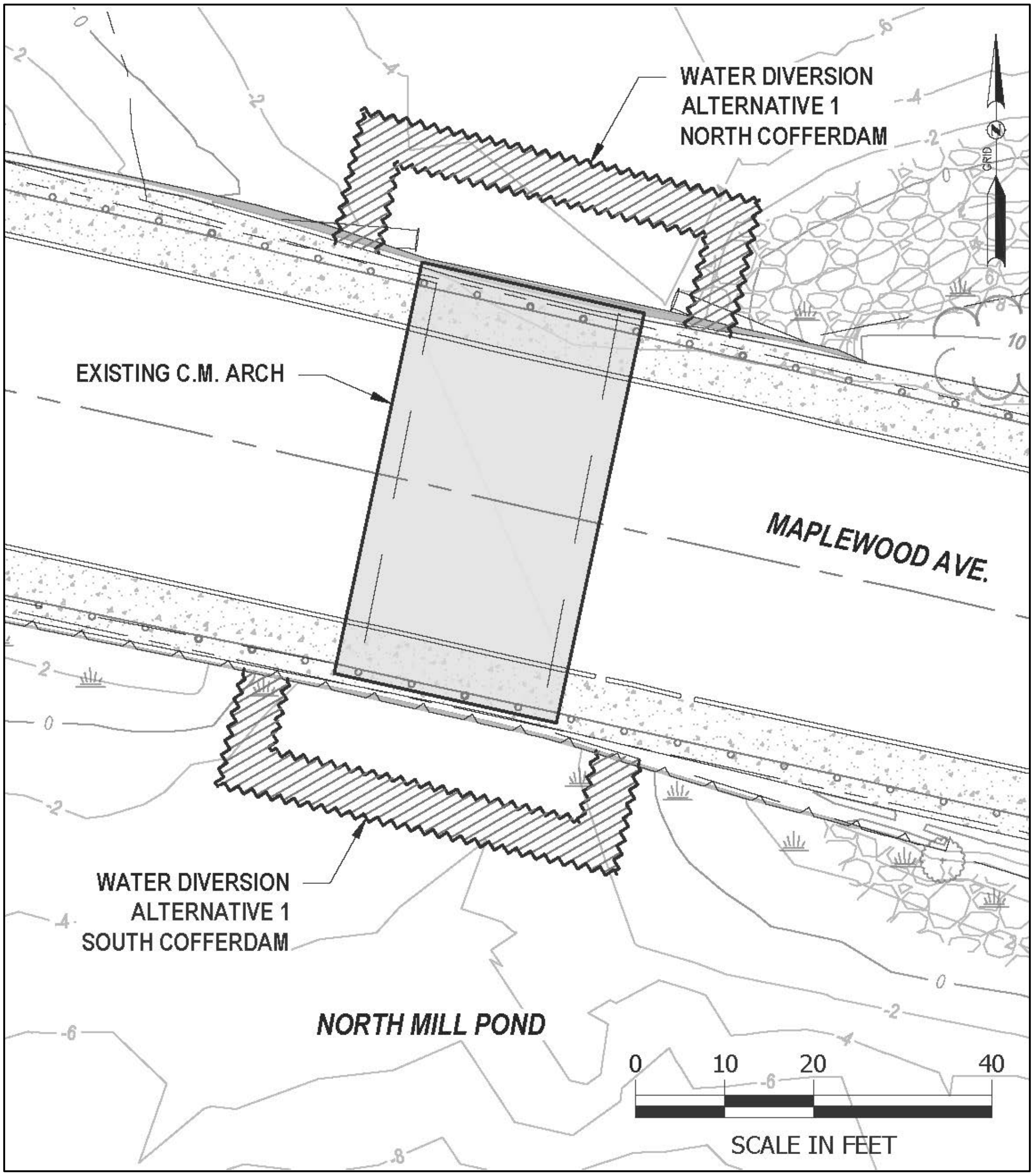


Figure 34 – Water Diversion Alternative 1 Schematic Plan

Water level records for the Seavey Island tide gage were searched to find a high water level equal to the 2-year recurrence interval. The most recent record occurred on December 17, 2020. 6-minute interval water level data for the 21-day period centered on December 17, 2020 (i.e. from December 7, 2020 to December 27, 2020) was obtained from the NOAA webpage (https://tidesandcurrents.noaa.gov/stationhome.html?id=8419870) and used to simulate water levels on the north side of the bridge during the construction window. The low water elevation measured on December 7, 2020 (-4.19 ft, NAVD88) was used as the water level on the south side of the road at the start of the simulation period. The analysis assumed this water level at the time the cofferdam on the south side of the bridge was installed such that only portions of the pond on the south side of the road above this elevation were available to store freshwater inflow during construction. This is considered a reasonable assumption for the low water level in that it is close to both the average (-4.12 ft) and median (-4.02 ft) low tide elevations for the simulation period and is about 2.2 feet higher than the 1.01-year low water level per the NOAA low water annual exceedance probability curve for tide gage. Figure 35 shows the measured stage hydrograph (green line) at the Seavey Island tide gage between December 7, 2020 and December 27, 2020.

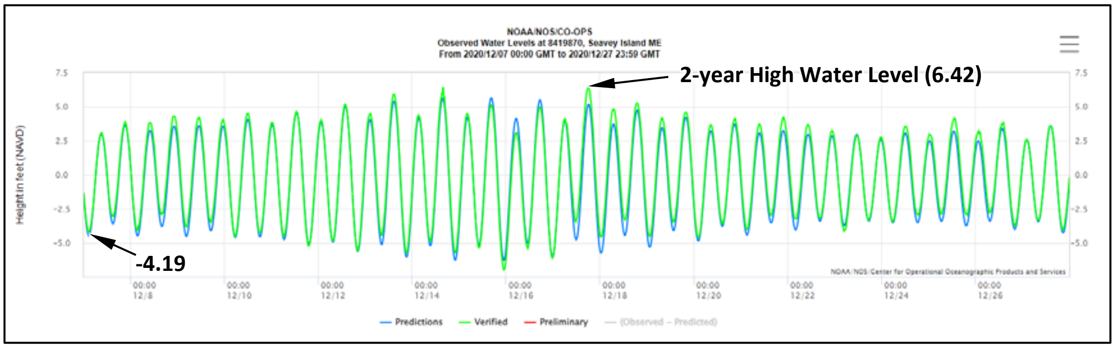


Figure 35 – Measured stage hydrograph (green line) at tide gage #8419870 for December 7 to December 27, 2020

The assumed base flow in Hodgson Brook (1 cfs) is about twice the flow that is equaled or exceeded 60% of the time between June 1 and October 31 as estimated with the seasonal flow regression equations in the web-based USGS StreamStats program[2] (see Appendix 2).

Based on NOAA precitation records for the years 1991 through 2020, the average October precipitation in Portland, ME is 5.25 inches (see Appendix 2). We have assumed the same average October precipitation in Portsmouth. If this rainfall is distributed evenly throughout the month, the average daily rainfall would be approximately 0.17 inches and the average cumulative rainfall for a 21-day period would be be 3.56 inches.

Per the Northeast Regional Climate Center (NRCC) "Extreme Precipitation" web tool (http://precip.eas.cornell.edu), the 1-year, 24-hour rainfall for the centroid of the watershed draining to the bridge is 2.66 inches (see Appendix 1). We have assumed that two storms of this magnitude would occur during the 21-day constuction period, one on day 7 and the other on day 14. The cumulative precipitation depth from these two storms is 5.32 inches, or approximately 150% of the estimated average precipitation for a 21-day period in October.

The SCS unit hydrograph method was used with the HydroCAD computer program to estimate runoff hydrographs resulting from the two 1-year, 24-hour storms (refer to Section B for a detailed description of the rainfall-runoff model). These hydrographs were combined with the base flow estimate in Hodgson Brook to develop the overall freshwater inflow hydrograph for the 21-day construction period shown as Figure 36.

[2] Flynn, R.H. and Tasker, G.D.,2002, Development of Regression Equations to Estimate Flow Durations and Low-Flow-Frequency Statistics in New Hampshire Streams: U.S. Geological Survey Scientific Investigations Report 02-4298, 66 p. (http://pubs.water.usgs.gov/wrir02-4298)

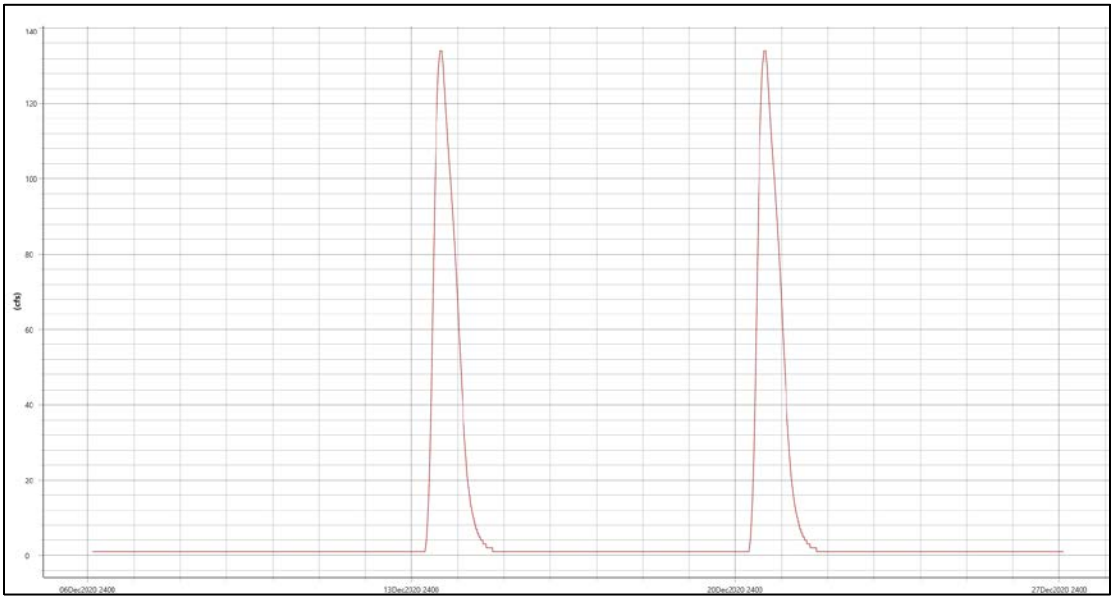


Figure 36 – Assumed freshwater inflow hydrograph for Water Diversion Alternative 1

The assumed cofferdam geometry, tide stage hydrograph, and freshwater inflow hydrograph were used to create a HEC-RAS 2D flow model which simulates the 21-day construction period. Table 19 summarizes the peak water levels calculated with the model.

Table 19 – Peak water levels calculated for Water Diversion Alternative 1

Location	Peak Water Level – Water Diversion Alternative 1 (feet, NAVD88)
North Cofferdam	6.42
South Cofferdam	5.75

The total assumed inflow volume to the portion of North Mill Pond south of the road during the 21-day construction period is 329 acre-feet.

F.2. <u>Water Diversion Alternative 2 – Temporary Culverts</u>

Two temporary culvert scenarios were evaluated – three 48-inch diameter pipes (Water Diversion Alternative 2.1) and two 72-inch diameter pipes (Water Diversion Alternative 2.2). Assumptions made for these two alternatives are as follows:

- Top elevation of north cofferdam: 6.5 ft (NAVD88)
- Top elevation of south cofferdam: 6.5 ft (NAVD88)
- Culvert inlet invert elevations: -3.5 ft (NAVD88)
- Culvert outlet invert elevations: -4.5 ft (NAVD88)
- Construction month: October
- Highest high tide elevation: 2-year high water level (6.42 ft, NAVD88)
- Hodgson Brook base flow: 1 cfs
- Rainfall: One 1-year, 24-hour rainfall event (2.66” rainfall depth) with peak freshwater inflow coincident with highest high tide

Under these two water diversion alternatives the temporary culverts would allow the portion of North Mill Pond south of Maplewood Avenue to drain during tide cycle troughs such that, unlike Water Diversion Alternative 1, there would not be a continuous accumulation of freshwater on the south side of the road. Therefore, modeling the entire construction period was unnecessary and the models for these two alternatives only simulate a 36-hour period which includes an assumed worst case scenario where peak freshwater inflow coincides with the highest high tide.

The 2-year high water level was used as the assumed highest high tide elevation (6.42 ft) in the models. This elevation was determined from the NOAA high water annual exceedance probability curve for the Seavey Island tide gage as described under Section F.1. Water level records for the Seavey Island tide gage for the 36-hour period between 9:00 AM on December 17, 2020 and 9:00 PM on December 18, 2020 were downloaded from the NOAA webpage and used to develop a tide stage hydrograph simulating water levels on the north side of the bridge (see Figure 37). This data includes a stage equal to the 2-year high water level which was measured at 5:48 PM on December 17, 2020.

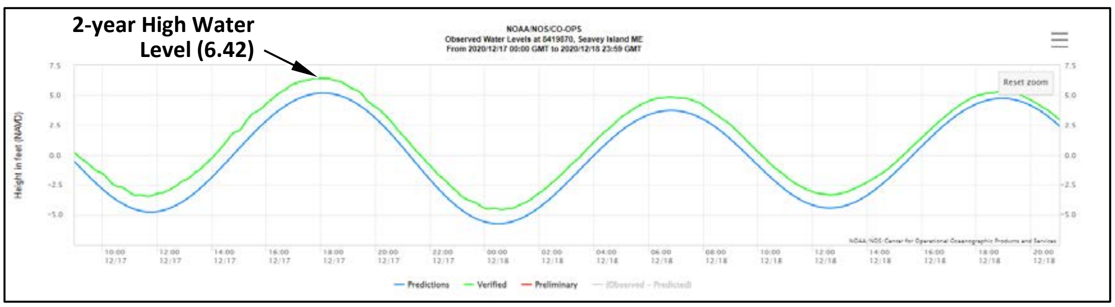


Figure 37 – Measured stage hydrograph (green line) at tide gage #8419870 for 9:00 AM December 17 to 9:00 PM December 18, 2020

As with Water Diversion Alternative 1, the assumed elevation of the north cofferdam (6.5 ft) was set slightly higher than the assumed highest high tide elevation and the assumed elevation of the south cofferdam (6.5 ft) was set to match the north cofferdam elevation.

The October construction timeframe and assumed base flow in Hodgson Brook (1 cfs) were estimated as described under Section F.1.

The freshwater inflow hydrograph to North Mill Pond resulting from the 1-year, 24-hour storm was estimated with the rainfall-runoff model described in Section B. This hydrograph was combined with the base flow estimate in Hodgson Brook to develop the overall freshwater inflow hydrograph for the 36-hour simulation period shown as Figure 38. The estimated inflow hydrograph was generated to peak concurrent with the highest high tide so as to simulate a near worst-case scenario wherein the peak freshwater runoff enters North Mill Pond at the same time the tide reaches its maximum level.

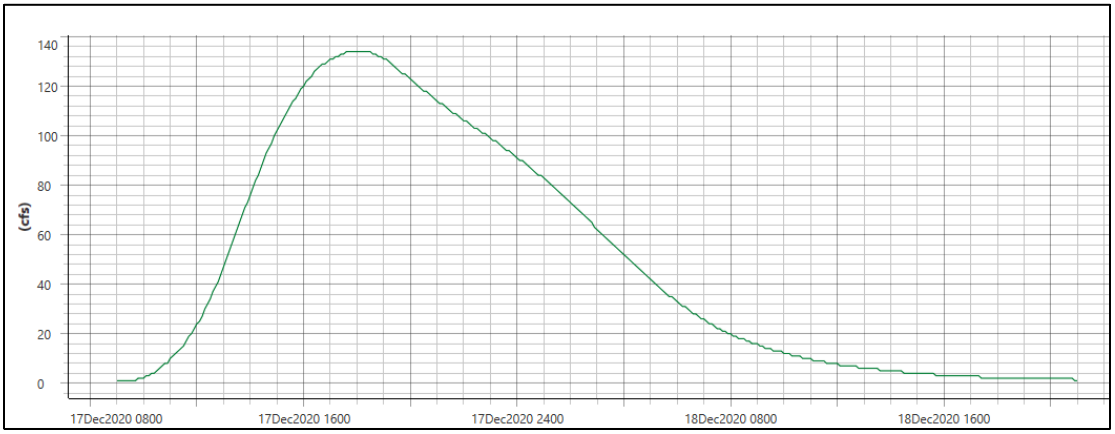


Figure 38 – Assumed freshwater inflow hydrograph for Water Diversion Alternatives 2 through 4

The assumed cofferdam and culvert geometries, tide stage hydrograph, and freshwater inflow hydrograph were used to create a HEC-RAS 2D flow model for each alternative which simulates the 36-hour analysis period. Results of each model are summarized in Sections F.2.1 and F.2.2.

F.2.1. Water Diversion Alternative 2.1 – Temporary 48-inch Culverts

Figure 39 shows a schematic plan view of the cofferdams and temporary culverts evaluated under Water Diversion Alternative 2.1, Figure 40 shows a cross-section at the existing bridge inlet with the temporary culverts installed, and Table 20 summarizes the peak water levels calculated with the model.

Table 20 – Peak water levels calculated for Water Diversion Alternative 2.1

Location	Peak Water Level – Water Diversion Alternative 2.1 (feet, NAVD88)
North Cofferdam	6.42
South Cofferdam	4.22

Figure 41 shows the stage and flow hydrographs calculated at the bridge with the HEC-RAS 2D model for Water Diversion Alternative 2.1. The headwater stage hydrograph represents water levels at the south cofferdam, the tailwater stage hydrograph represents water levels at the north cofferdam, and the flow hydrograph shows the cumulative flow through the three temporary 48-inch culverts. The maximum combined flow through the culverts from the south to north is approximately 368 cfs and from north to south the maximum combined flow is about 406 cfs.

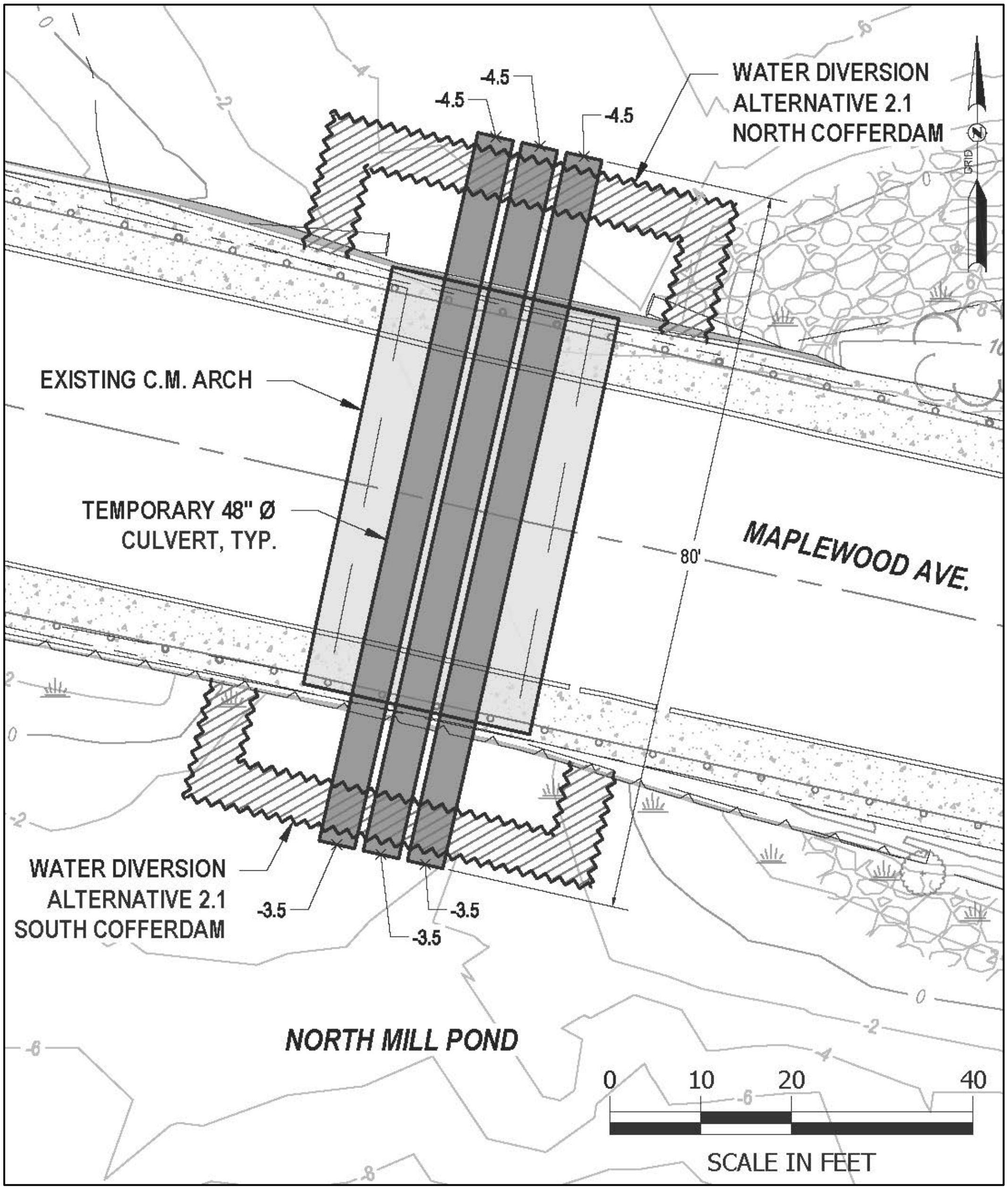


Figure 39 – Water Diversion Alternative 2.1 Schematic Plan

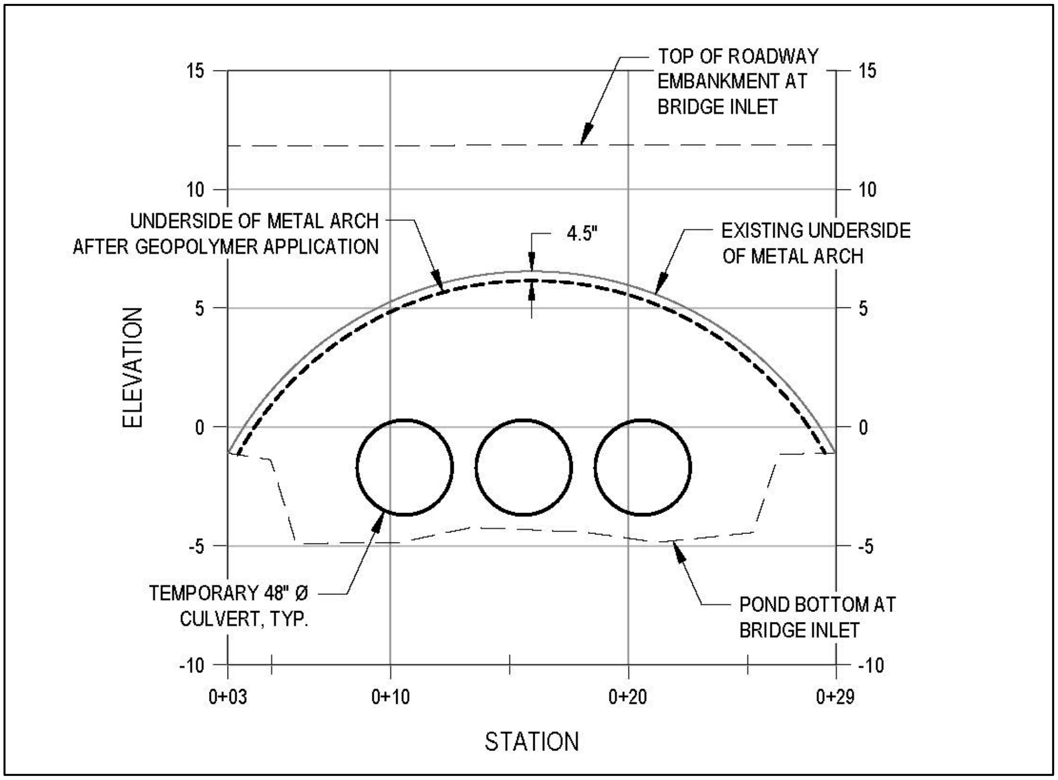


Figure 40 – Water Diversion Alternative 2.1 Bridge Inlet Cross-Section

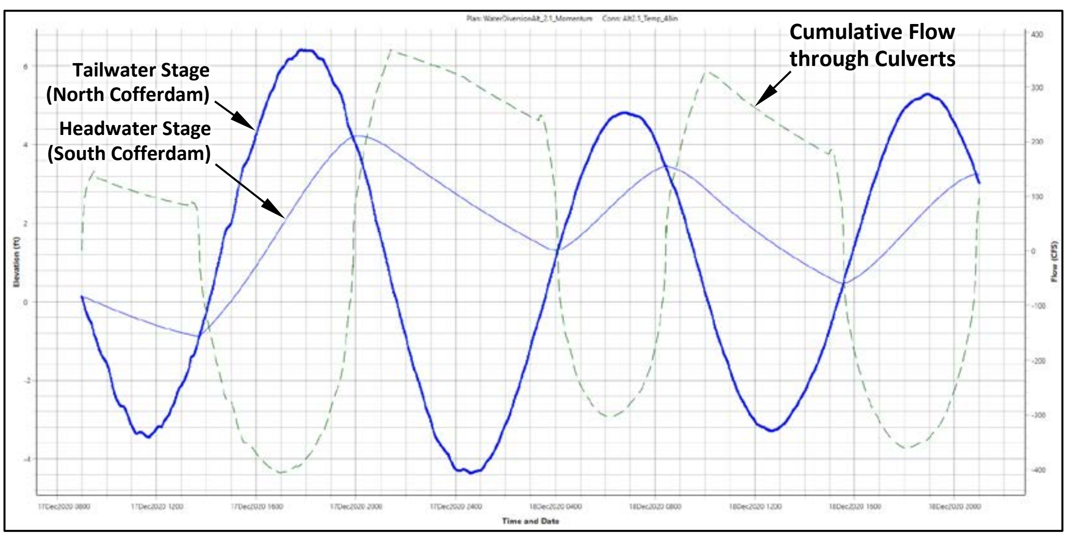


Figure 41 – Stage and flow hydrographs calculated at the bridge for Water Diversion Alternative 2.1

As shown in Figure 41, water levels on the south side of Maplewood Avenue never get as high or as low as the water levels on the north side of the road. This is due to the limited discharge capacity of the temporary culverts and the relatively short time between the crests and troughs of the tide cycles, both of which combine to prevent the portion of the pond on the south side of the road from filling or draining to the same water levels experienced on the north side.

F.2.2. Water Diversion Alternative 2.2 – Temporary 72-inch Culverts

Figure 43 shows a schematic plan view of the cofferdams and temporary culverts evaluated under Water Diversion Alternative 2.2, Figure 44 shows a cross-section at the existing bridge inlet with the temporary culverts installed, and Table 21 summarizes the peak water levels calculated with the model.

Table 21 – Peak water levels calculated for Water Diversion Alternative 2.2

Location	Peak Water Level – Water Diversion Alternative 2.2 (feet, NAVD88)
North Cofferdam	6.42
South Cofferdam	4.98

As compared to Water Diversion Alternative 2.1, maximum water levels on the south side of the road are 0.66 feet higher. This is because the twin 72-inch pipes have a greater capacity than the three 48-inch pipes which allows more tidal inflow from north to south. The maximum cumulative flow through the culverts from north to south is approximately 583 cfs (as compared to 406 cfs for Water Diversion Alternative 2.1) and from the south to north the maximum combined flow is about 500 cfs (as compared to 368 cfs for Water Diversion Alternative 2.1). Figure 42 shows the stage and flow hydrographs calculated at the bridge with the HEC-RAS 2D model for Water Diversion Alternative 2.2.

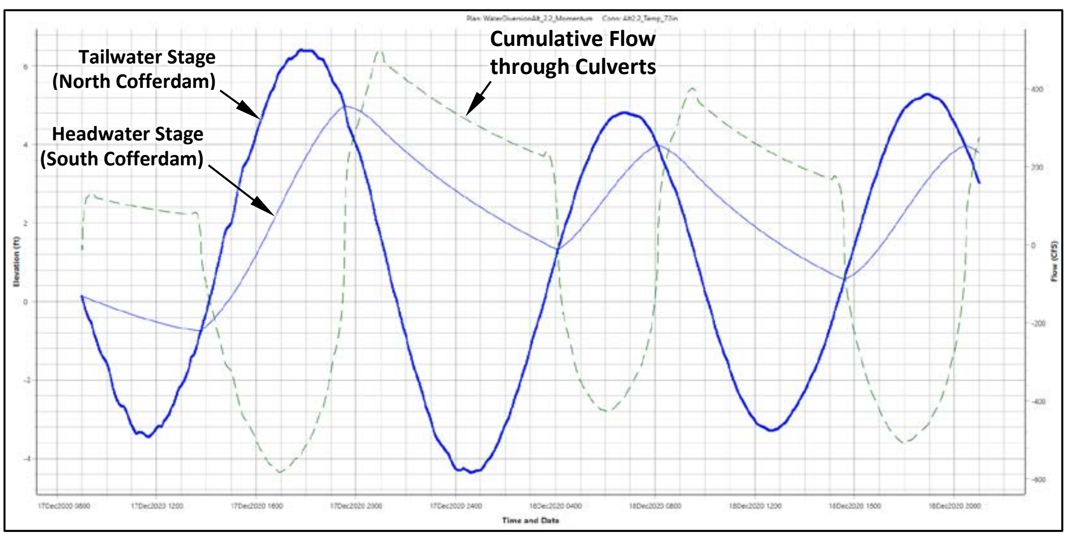


Figure 42 – Stage and flow hydrographs calculated at the bridge for Water Diversion Alternative 2.2

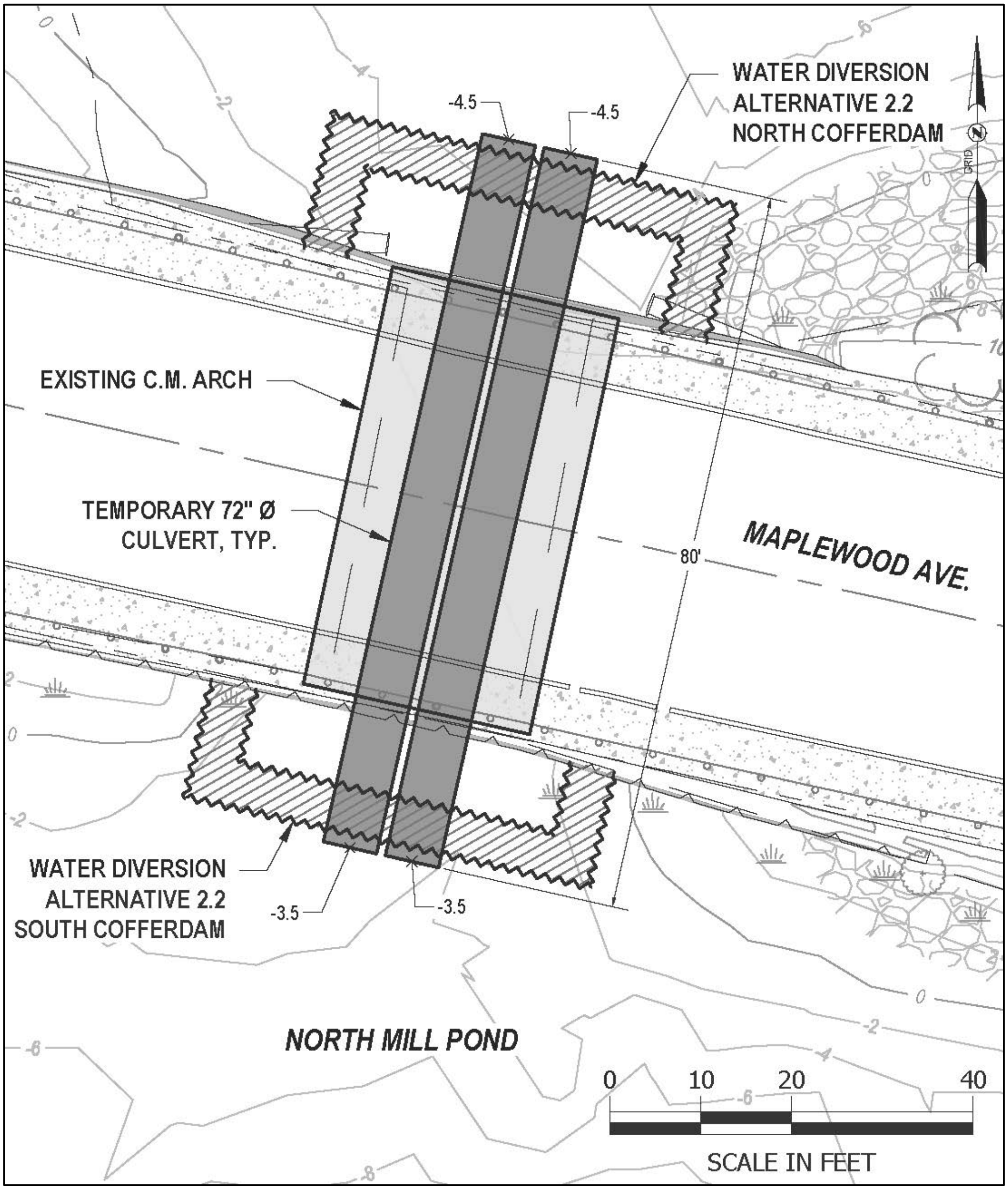


Figure 43 – Water Diversion Alternative 2.2 Schematic Plan

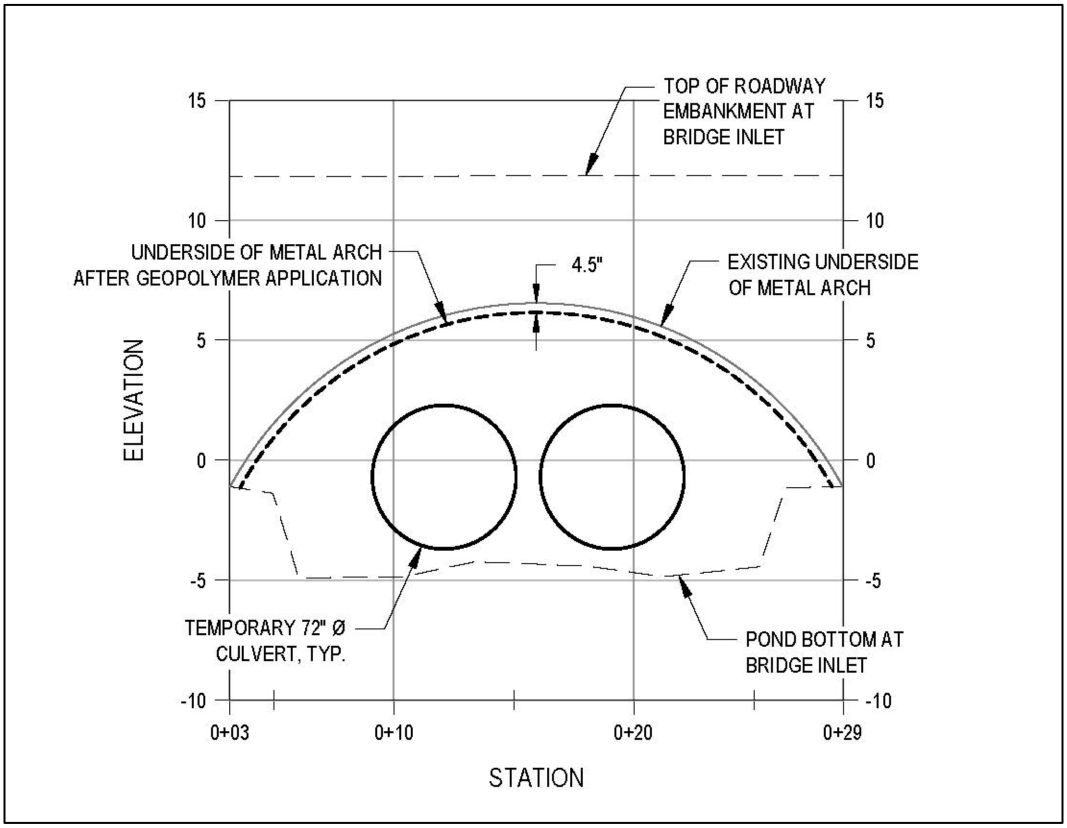


Figure 44 – Water Diversion Alternative 2.2 Bridge Inlet Cross-Section

F.3. Water Diversion Alternative 3 – Phased Water Diversion

Figure 45 shows a schematic plan view of the cofferdam configuration analyzed under Water Diversion Alternative 3. This configuration has the east side of the existing waterway opening dewatered and flow diverted through the west side of the bridge. The deepest portion of the channel between the footings is on the east side of the waterway opening; therefore, the smallest active flow area under this alternative would occur when flow is diverted through the west side of the bridge as modeled. Application of the geopolymer liner to the west side of the metal arch would require mirroring the cofferdam configuration shown in Figure 45 about the structure centerline such that flow is diverted through the east side of the bridge. This cofferdam arrangement was not modeled, but since the waterway opening would be slightly larger, flow conveyance would be slightly greater and model results would likely be similar.

The analysis assumed that the exterior face of the cofferdam would be offset two feet from the structure centerline such that slightly more than half of the waterway opening would be dewatered and slightly less than half of the opening would be available for flow conveyance. We have assumed that slightly more than half of the structure will need to be dewatered so that the geopolymer liner can be applied to half of the structure. In addition, the model geometry assumes that: (1) the geopolymer liner has been applied to the underside of the metal arch on the west side of the structure prior to diverting flow and (2) the sewer main obstructs a portion of the waterway opening at the bridge outlet.

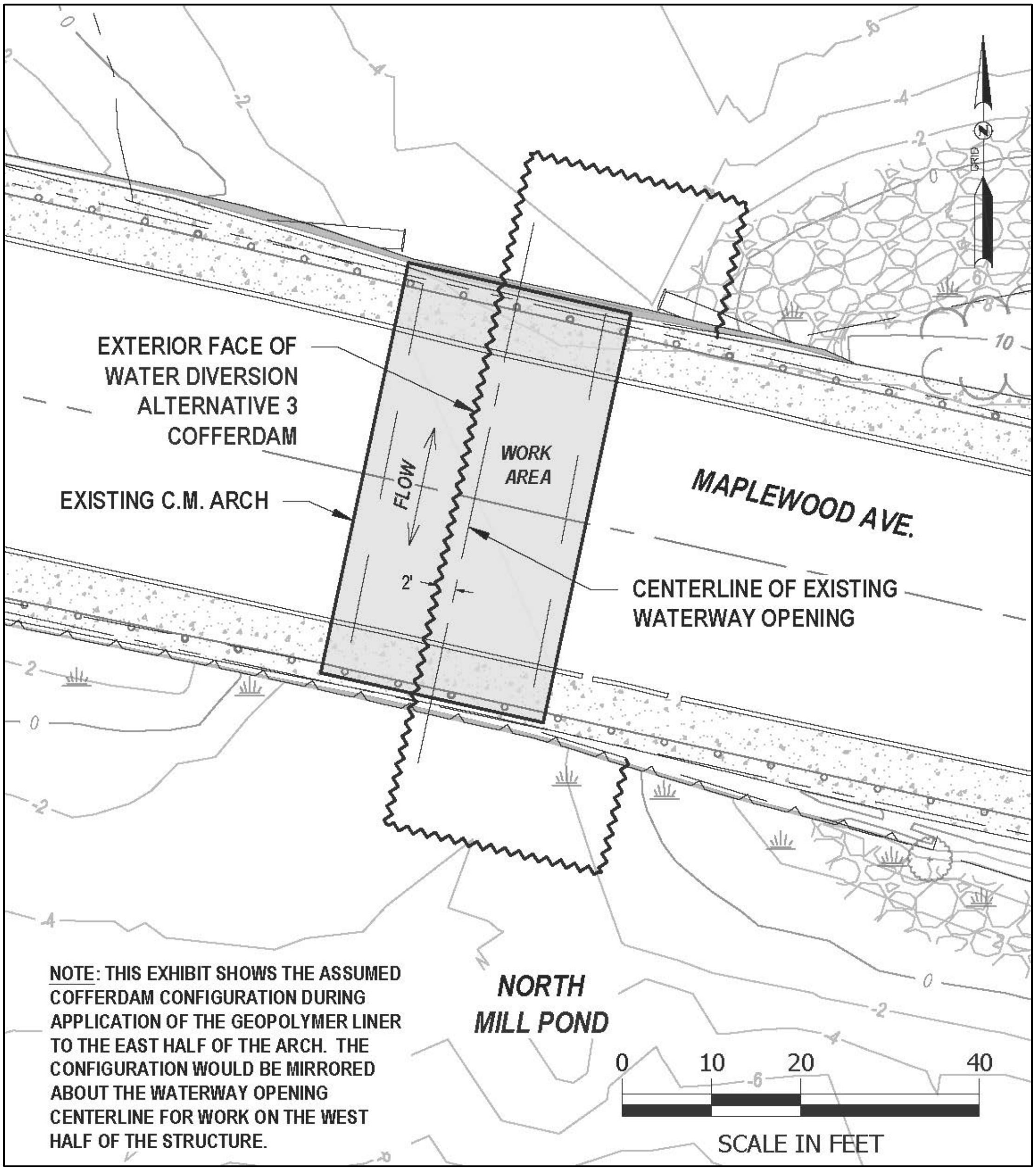


Figure 45 – Water Diversion Alternative 3 Schematic Plan

Figure 46 shows a cross-section at the existing bridge inlet under Water Diversion Alternative 3 with the temporary cofferdam installed and the geopolymer liner applied.

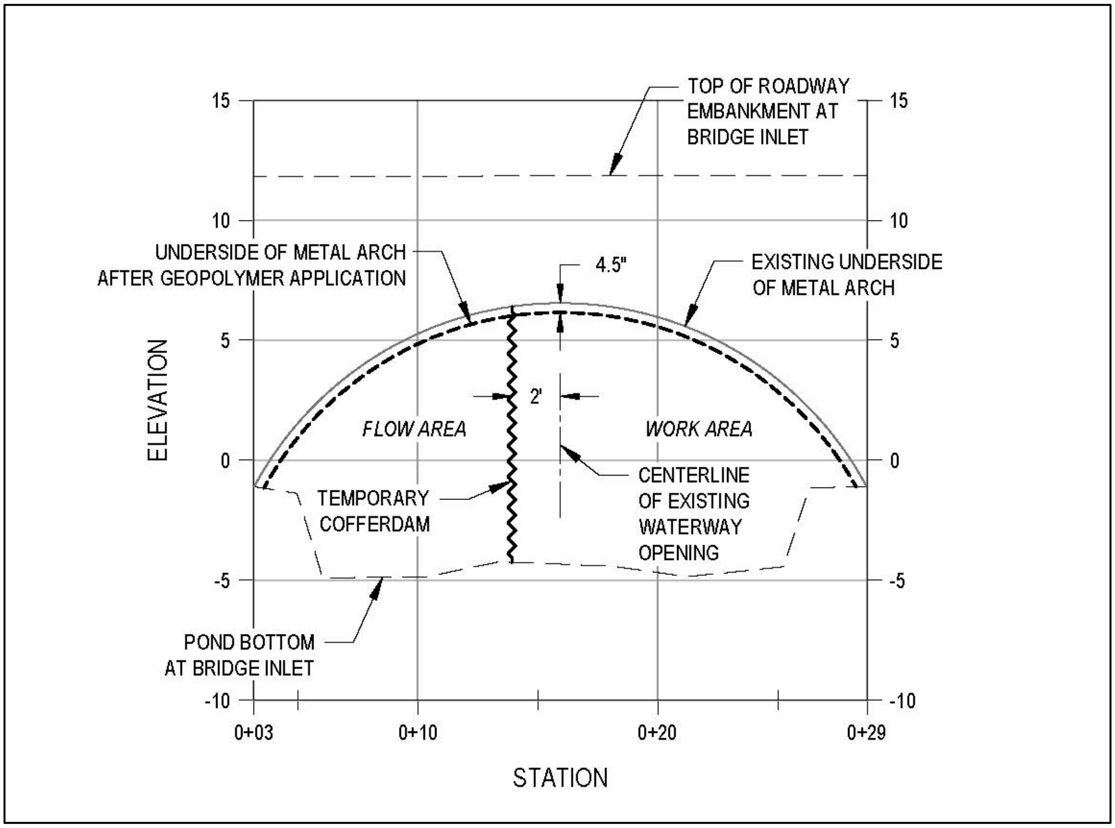


Figure 46 – Water Diversion Alternative 3 Bridge Inlet Cross-Section

The hydraulic model for this water diversion alternative was developed by modifying the existing bridge geometry to reflect the reduced waterway opening area resulting from the cofferdam and geopolymer liner. Figure 47 shows a cross-section of the bridge inlet as coded in the model.

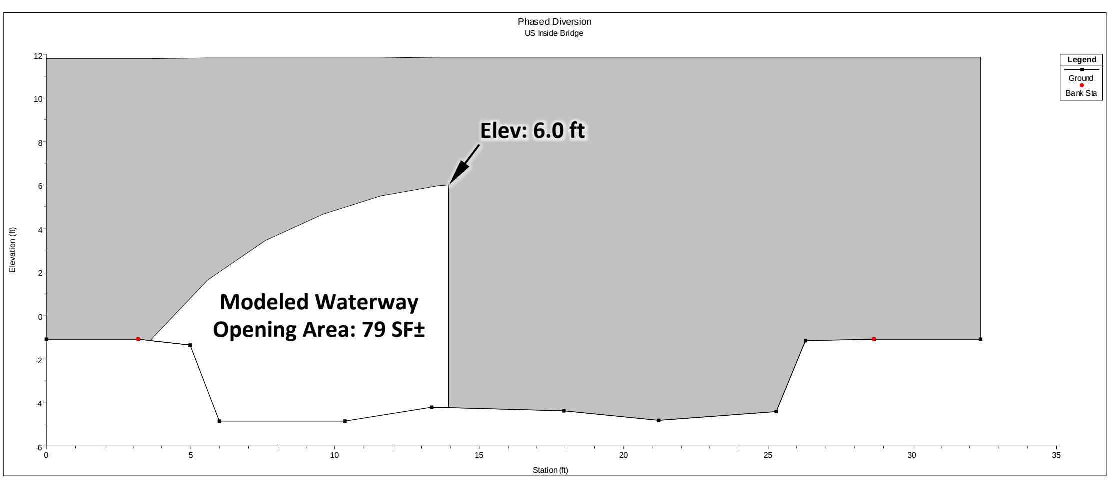


Figure 47 - Bridge inlet cross-section as coded in the hydraulic model for Water Diversion Alternative 3

The hydraulic connection afforded by the unobstructed portion of the bridge opening would allow the portion of North Mill Pond on the south side of Maplewood Avenue to partially drain as the tide falls. Consequently, freshwater inflow to the Pond would not be stored throughout the construction period and the model for this alternative only simulates a 36-hour period which includes an assumed worst case scenario where peak freshwater inflow coincides with the highest high tide. The flow and tide conditions assumed for Water Diversion Alternatives 2.1 and 2.2 were also assumed for this alternative. These and other assumptions are as follows:

- Construction month: October
- Highest high tide elevation: 2-year high water level (6.42 ft, NAVD88)
- Hodgson Brook base flow: 1 cfs
- Rainfall: One 1-year, 24-hour rainfall event (2.66" rainfall depth) with peak freshwater inflow coincident with highest high tide

The HEC-RAS 2D flow model for Water Diversion Alternative 3 used the same 2-year high water level, tide stage hydrograph, and freshwater inflow hydrograph assumed for Water Diversion Alternatives 2.1 and 2.2 (see Section F.2.).

Table 22 summarizes the peak water levels calculated with the model. These water levels are very close to the low chord elevations of the metal arch directly above the assumed cofferdam locations within the structure, especially after application of the geopolymer liner, and are well above the elevation of the sewer main. Therefore, if water levels similar to those assumed in the model are expected during construction, water-tight seals between the cofferdam and underside of the metal arch and between the cofferdam and sewer main would be needed to prevent water intrusion into the work area.

Table 22 – Peak water levels calculated for Water Diversion Alternative 3

Location	**Peak Water Level – Water Diversion Alternative 3 (feet, NAVD88)**
Bridge Outlet (North)	6.42
Bridge Inlet (South)	5.36

Figure 48 shows the stage and flow hydrographs calculated at the bridge with the HEC-RAS 2D model for Water Diversion Alternative 3. The maximum flow from south to north through the open portion of the bridge is approximately 580 cfs and from north to south the maximum flow is about 700 cfs.

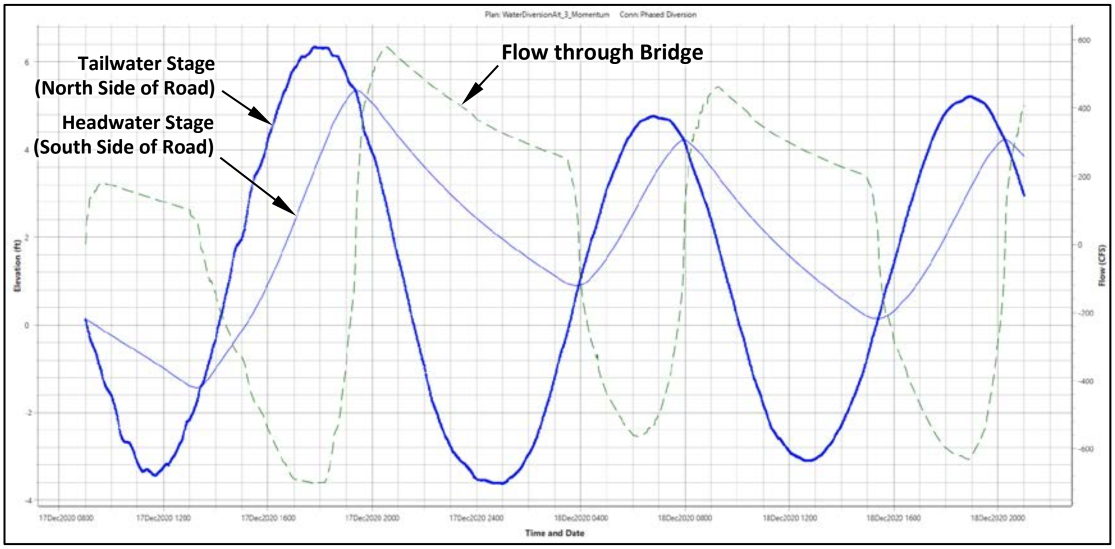


Figure 48 – Stage and flow hydrographs calculated at the bridge for Water Diversion Alternative 3

F.4. Water Diversion Alternative 4 – Permanent Culvert

Figure 49 shows a schematic plan view of the cofferdam and permanent culvert configuration analyzed under Water Diversion Alternative 4. As shown on the schematic plan, during construction the north and south cofferdams would prevent any flow through the bridge and all water would pass through the culvert. The culvert geometry assumed for this alternative is the same assumed for Bridge Rehabilitation Alternative 2.2 (see Section E.2.2) and includes a single 130-foot long 60-inch diameter pipe installed with the west edge of the barrel 60 feet east from the edge of the existing bridge opening and invert elevations of -4.0 feet (NAVD88) at both ends. The culvert was modeled with an entrance loss coefficient of 0.9, which reflects our assumption that it will be installed such that both ends project from the embankment without headwalls or other end treatments, and a Manning's n roughness coefficient of 0.012 to represent a smooth interior surface typical of dual wall HDPE and precast concrete pipe.

The culvert would allow the portion of North Mill Pond south of Maplewood Avenue to partially drain during tide cycle troughs such that freshwater would not continuously accumulate on the south side of the road when the cofferdams are in place. Therefore, modeling the entire construction period was unnecessary and the model for this alternative only simulates a 36-hour period which includes a scenario where the peak freshwater inflow coincides with the highest high tide. The same flow and tide conditions assumed for Water Diversion Alternatives 2.1, 2.2, and 3 were also assumed for this alternative. These and other assumptions are as follows:

- Top elevation of north cofferdam: 6.5 ft (NAVD88)
- Top elevation of south cofferdam: 6.5 ft (NAVD88)
- Culvert inlet invert elevation: -4.0 ft (NAVD88)
- Culvert outlet invert elevation: -4.0 ft (NAVD88)
- Construction month: October
- Highest high tide elevation: 2-year high water level (6.42 ft, NAVD88)
- Hodgson Brook base flow: 1 cfs

- Rainfall: One 1-year, 24-hour rainfall event (2.66" rainfall depth) with peak freshwater inflow coincident with highest high tide

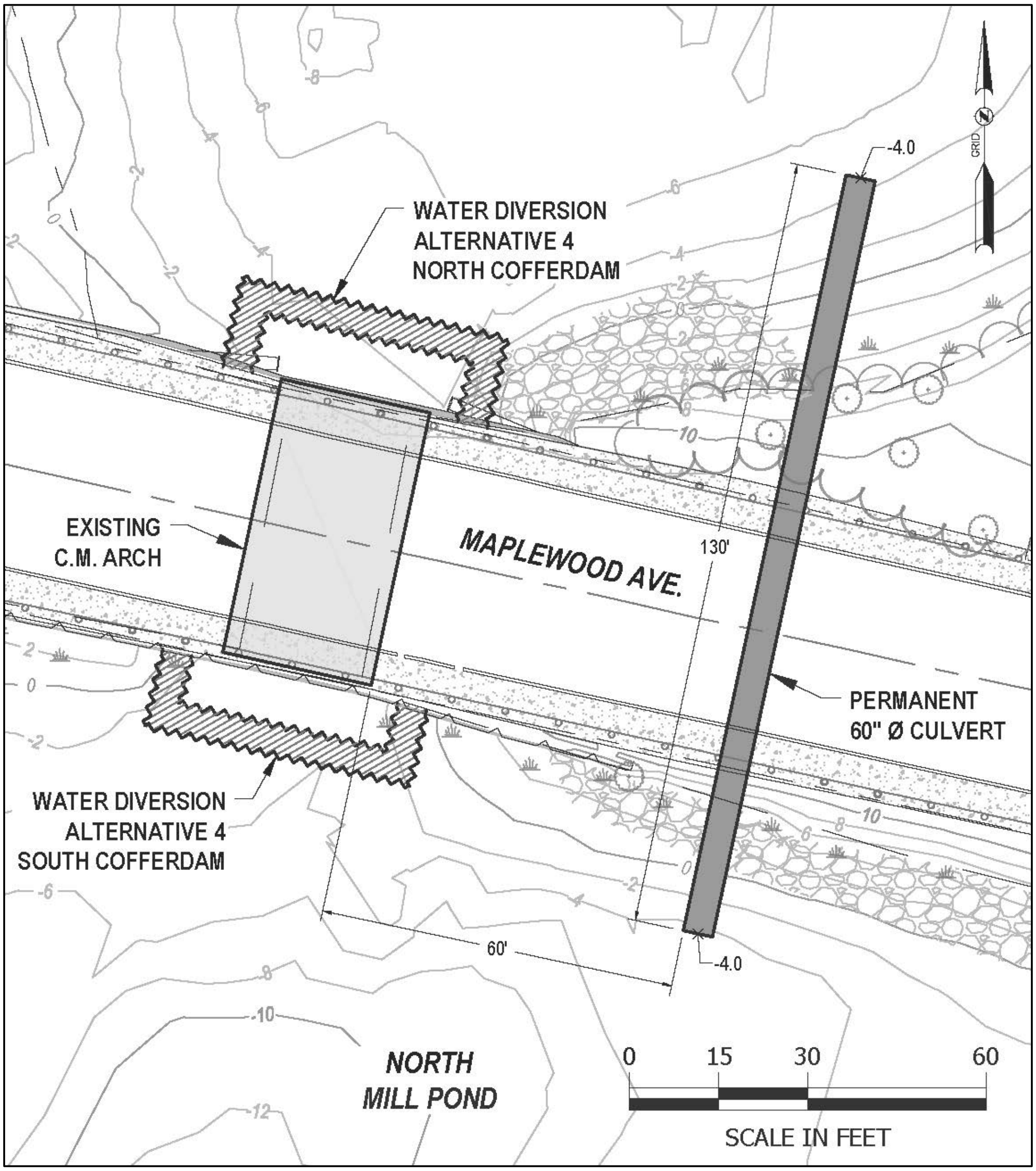


Figure 49 – Water Diversion Alternative 4 Schematic Plan

The HEC-RAS 2D flow model for Water Diversion Alternative 4 used the same 2-year high water level, tide stage hydrograph, and freshwater inflow hydrograph assumed for Water Diversion Alternatives 2.1, 2.2, and 3 (see Section F.2. and F.3.). Table 23 summarizes the calculated peak water levels.

Table 23 – Peak water levels calculated for Water Diversion Alternative 4

Location	Peak Water Level – Water Diversion Alternative 4 (feet, NAVD88)
North Cofferdam	6.42
South Cofferdam	3.32

Figure 50 shows the stage and flow hydrographs calculated at the bridge with the HEC-RAS 2D model for Water Diversion Alternative 4. The maximum flow from south to north through the 60-inch culvert is approximately 174 cfs and from north to south the maximum flow through the culvert is about 224 cfs.

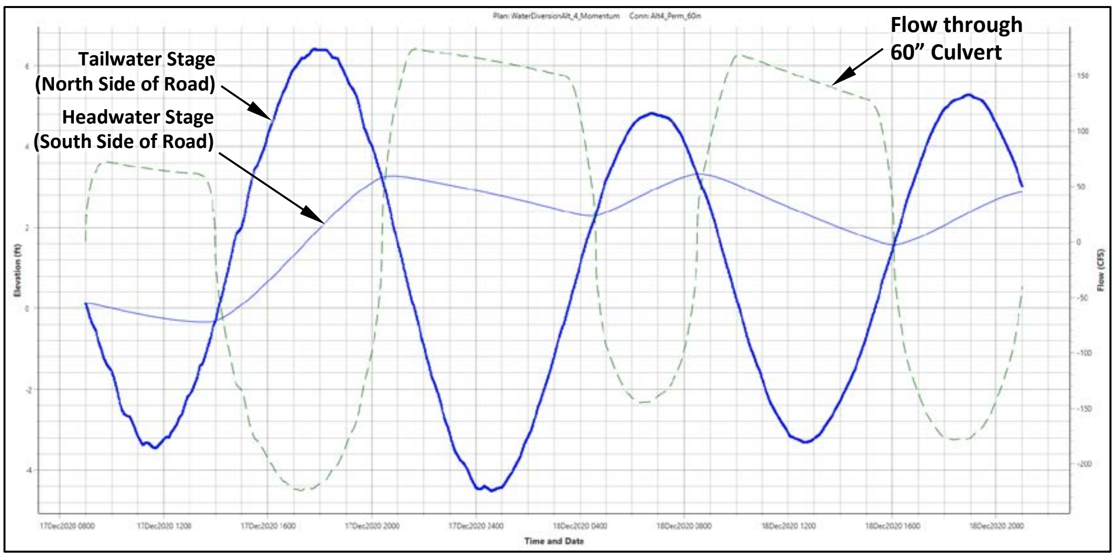


Figure 50 – Stage and flow hydrographs calculated for Water Diversion Alternative 4

As shown in Figure 50, water levels on the south side of the road do not get as high or low as the water levels on the north side. This is due to the limited flow capacity of the culvert and the relatively short time between the crests and troughs of the tide cycles, both of which combine to prevent the waterbody on the south side of the road from filling or draining to the same levels in the waterbody on the north side.

APPENDIX 1

SUPPORTING DOCUMENTATION FOR RAINFALL-RUNOFF MODEL

Watershed Relief Map

Drainage Plan

NRCC Precipitation Estimates

Land Cover Table

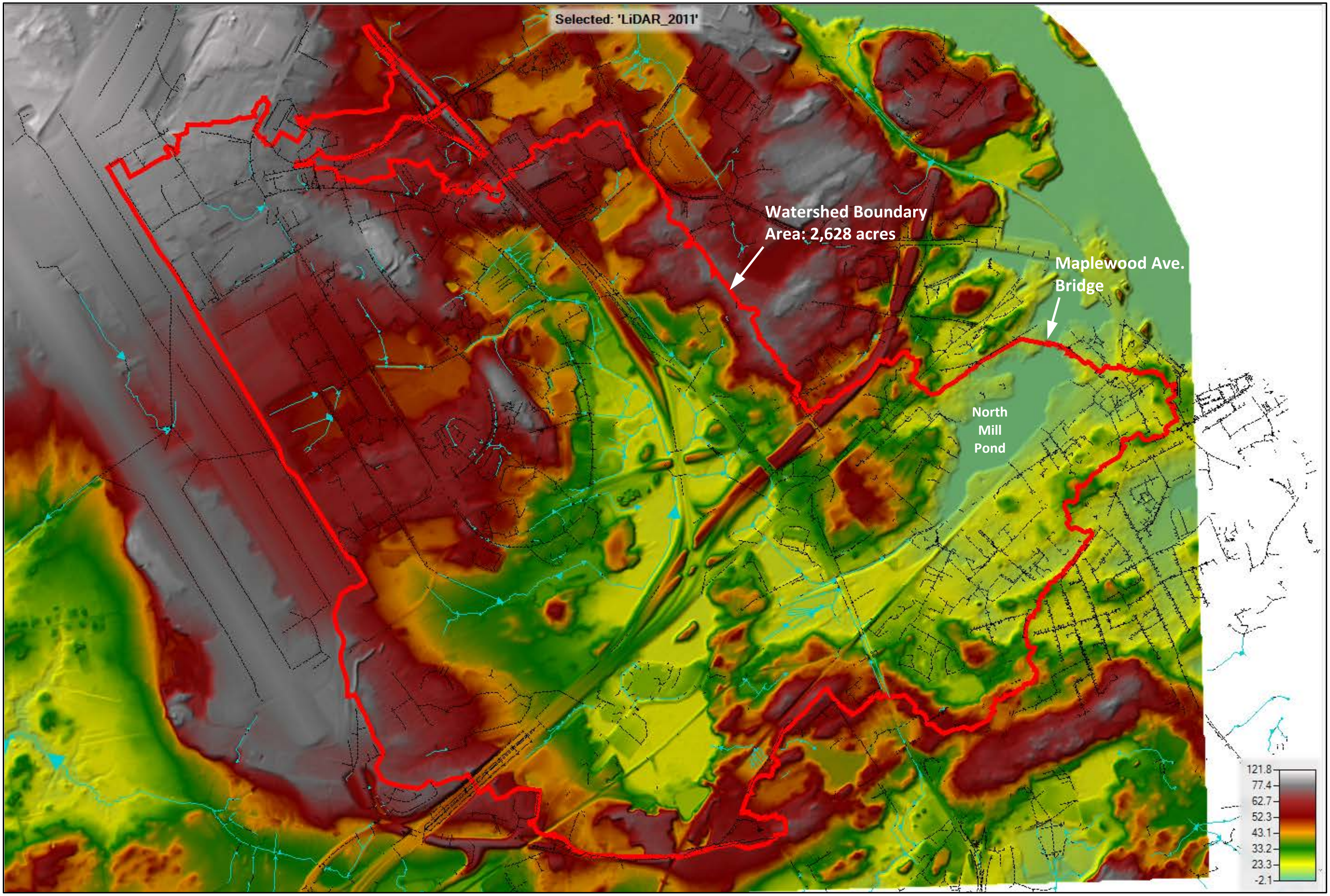
Soil – Land Cover Map

Soil – Land Cover Polygons Table

Time of Concentration Table

HydroCAD Report

Maplewood Avenue over North Mill Pond
Watershed Relief Map



NOTES:

1. TOPOGRAPHY WAS GENERATED FROM 2011 LiDAR DATA DOWNLOADED FROM NH GRANIT (COASTAL NEW HAMPSHIRE - 2011 DATASET). CONTOUR INTERVAL IS 2 FEET. VERTICAL DATUM IS NAVD88.
2. BACKGROUND IMAGE IS 2017 ORTHOPHOTOGRAPHY DISPLAYED VIA LINK TO NH GRANIT WEB MAP SERVICES (2017 COASTAL HIGH RESOLUTION (1 FT.) AERIAL PHOTOGRAPHY LAYER).
3. CHANNEL AND PIPE MAPPING IS FROM GIS DATA PROVIDED BY THE CITY OF PORTSMOUTH.

DATE OF PRINT: 02/01/21

HEADWATERS
Consulting, LLC
P.O. Box 744 Littleton, NH 03561 (603) 616-6850

HOYLE, TANNER & ASSOCIATES, INC.
MAPLEWOOD AVE. OVER NORTH MILL POND BRIDGE REPLACEMENT
PORTSMOUTH, NH

DRAINAGE PLAN

NO.	REVISION	DATE

DATE: FEB. 2021
PROJECT #: 1920
ENGINEERED BY: ---
DRAWN BY: SPS

SHEET 1 OF 1

Extreme Precipitation Tables

Northeast Regional Climate Center

Data represents point estimates calculated from partial duration series. All precipitation amounts are displayed in inches.

Smoothing	No
State	New Hampshire
Location	
Longitude	70.792 degrees West
Latitude	43.074 degrees North
Elevation	0 feet
Date/Time	Mon, 01 Feb 2021 08:12:03 -0500

Extreme Precipitation Estimates

	5min	10min	15min	30min	60min	120min		1hr	2hr	3hr	6hr	12hr	24hr	48hr		1day	2day	4day	7day	10day	
1yr	0.26	0.40	0.49	0.66	0.82	1.00	**1yr**	0.70	0.98	1.14	1.58	2.02	2.66	2.92	**1yr**	2.35	2.81	3.22	3.94	4.55	**1yr**
2yr	0.32	0.50	0.61	0.83	1.02	1.21	**2yr**	0.88	1.18	1.40	1.86	2.41	3.21	3.57	**2yr**	2.84	3.43	3.93	4.68	5.32	**2yr**
5yr	0.37	0.57	0.71	0.98	1.24	1.50	**5yr**	1.07	1.46	1.73	2.32	2.96	4.07	4.57	**5yr**	3.60	4.40	5.04	5.93	6.70	**5yr**
10yr	0.42	0.65	0.80	1.12	1.44	1.76	**10yr**	1.25	1.72	2.04	2.73	3.47	4.87	5.53	**10yr**	4.31	5.32	6.08	7.10	7.98	**10yr**
25yr	0.50	0.75	0.94	1.34	1.76	2.18	**25yr**	1.52	2.13	2.53	3.39	4.27	6.17	7.10	**25yr**	5.46	6.83	7.79	9.02	10.06	**25yr**
50yr	0.56	0.85	1.06	1.53	2.06	2.57	**50yr**	1.78	2.51	2.98	3.99	5.01	7.39	8.58	**50yr**	6.54	8.25	9.41	10.81	11.99	**50yr**
100yr	0.64	0.97	1.21	1.75	2.40	3.03	**100yr**	2.07	2.96	3.51	4.71	5.88	8.86	10.38	**100yr**	7.84	9.98	11.36	12.96	14.29	**100yr**
200yr	0.73	1.09	1.38	2.01	2.80	3.57	**200yr**	2.41	3.49	4.13	5.56	6.89	10.62	12.55	**200yr**	9.40	12.07	13.72	15.54	17.05	**200yr**
500yr	0.87	1.29	1.66	2.42	3.44	4.45	**500yr**	2.97	4.35	5.14	6.92	8.52	13.50	16.15	**500yr**	11.95	15.53	17.62	19.78	21.54	**500yr**

Lower Confidence Limits

	5min	10min	15min	30min	60min	120min		1hr	2hr	3hr	6hr	12hr	24hr	48hr		1day	2day	4day	7day	10day	
1yr	0.23	0.36	0.44	0.59	0.73	0.89	**1yr**	0.63	0.87	0.92	1.32	1.66	2.22	2.52	**1yr**	1.97	2.42	2.85	3.15	3.88	**1yr**
2yr	0.31	0.49	0.60	0.81	1.00	1.19	**2yr**	0.86	1.16	1.37	1.82	2.34	3.05	3.46	**2yr**	2.70	3.32	3.82	4.55	5.07	**2yr**
5yr	0.35	0.54	0.67	0.92	1.17	1.40	**5yr**	1.01	1.37	1.61	2.12	2.74	3.79	4.20	**5yr**	3.36	4.04	4.72	5.54	6.25	**5yr**
10yr	0.39	0.59	0.73	1.03	1.32	1.60	**10yr**	1.14	1.56	1.81	2.40	3.07	4.38	4.89	**10yr**	3.88	4.70	5.46	6.43	7.22	**10yr**
25yr	0.44	0.67	0.83	1.19	1.56	1.90	**25yr**	1.35	1.86	2.10	2.77	3.55	4.69	5.93	**25yr**	4.15	5.71	6.68	7.83	8.72	**25yr**
50yr	0.48	0.73	0.91	1.31	1.77	2.17	**50yr**	1.53	2.12	2.35	3.09	3.96	5.30	6.86	**50yr**	4.69	6.60	7.78	9.10	10.07	**50yr**
100yr	0.54	0.81	1.02	1.47	2.01	2.47	**100yr**	1.74	2.42	2.63	3.44	4.39	5.95	7.94	**100yr**	5.26	7.63	9.07	10.58	11.62	**100yr**
200yr	0.59	0.89	1.13	1.64	2.29	2.82	**200yr**	1.97	2.76	2.94	3.82	4.85	6.65	9.18	**200yr**	5.89	8.83	10.56	12.32	13.44	**200yr**
500yr	0.69	1.03	1.32	1.92	2.73	3.37	**500yr**	2.35	3.30	3.41	4.37	5.54	7.73	11.12	**500yr**	6.84	10.69	12.92	15.09	16.27	**500yr**

Upper Confidence Limits

	5min	10min	15min	30min	60min	120min		1hr	2hr	3hr	6hr	12hr	24hr	48hr		1day	2day	4day	7day	10day	
1yr	0.28	0.44	0.54	0.72	0.89	1.08	**1yr**	0.77	1.06	1.26	1.75	2.21	3.00	3.14	**1yr**	2.65	3.02	3.58	4.38	5.05	**1yr**
2yr	0.33	0.52	0.64	0.86	1.06	1.26	**2yr**	0.92	1.24	1.48	1.96	2.51	3.43	3.69	**2yr**	3.04	3.55	4.07	4.83	5.64	**2yr**
5yr	0.40	0.61	0.76	1.05	1.33	1.61	**5yr**	1.15	1.58	1.88	2.53	3.24	4.34	4.94	**5yr**	3.84	4.75	5.37	6.35	7.13	**5yr**
10yr	0.47	0.72	0.89	1.24	1.60	1.97	**10yr**	1.38	1.92	2.27	3.10	3.93	5.34	6.17	**10yr**	4.72	5.93	6.77	7.81	8.72	**10yr**
25yr	0.57	0.87	1.08	1.54	2.03	2.55	**25yr**	1.75	2.50	2.94	4.05	5.11	7.81	8.28	**25yr**	6.92	7.96	9.05	10.28	11.36	**25yr**
50yr	0.66	1.01	1.26	1.81	2.44	3.10	**50yr**	2.10	3.03	3.58	4.97	6.25	9.79	10.37	**50yr**	8.66	9.97	11.29	12.65	13.90	**50yr**
100yr	0.78	1.18	1.48	2.13	2.93	3.77	**100yr**	2.53	3.69	4.34	6.11	7.67	12.25	12.97	**100yr**	10.85	12.48	14.08	15.59	17.01	**100yr**
200yr	0.91	1.37	1.74	2.52	3.51	4.60	**200yr**	3.03	4.50	5.30	7.52	9.40	15.38	16.26	**200yr**	13.61	15.63	17.58	19.21	20.82	**200yr**
500yr	1.13	1.68	2.16	3.14	4.46	5.96	**500yr**	3.85	5.83	6.87	9.93	12.33	20.80	21.91	**500yr**	18.41	21.07	23.59	25.31	27.22	**500yr**

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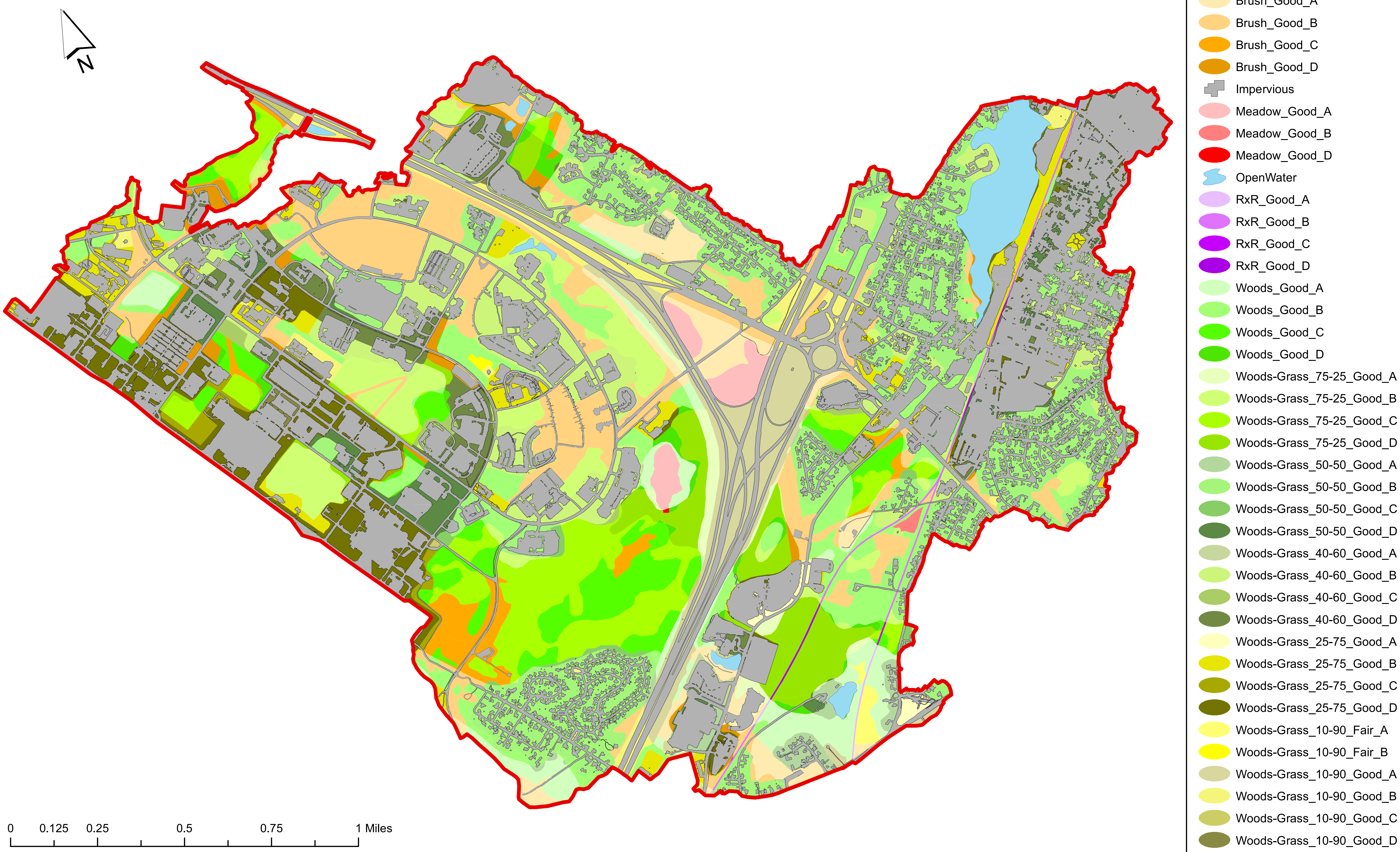
North Mill Pond Watershed Land Cover

categories from NHGRANIT "Land Use 2015 - Southeastern New Hampshire" layer

Note: Impervious areas have been removed from Land Use Category polygons such that the Cover Type applies to the land cover of the remaining polygons outside of impervious areas as estimated from 2017 orthophotography.

NHGRANIT Land Use Category	Cover Type	Condition
Brush or transitional between open & forested	Brush	Good
Electric, gas, and other utilities	Brush	Good
Limited & controlled highway right-of-way	Impervious	n/a
Park & ride lot	Impervious	n/a
Road right-of-way	Impervious	n/a
Agricultural land	Meadow	Good
Water	Open Water	n/a
Rail transportation	Railroad Tracks	n/a
Forest land	Woods	Good
Other transportation, communications, and utilities	Woods	Good
Auxilliary transportation	Woods/Grass 10/90	Good
Cemetaries	Woods/Grass 10/90	Good
Communication	Woods/Grass 10/90	Good
Disturbed land	Woods/Grass 10/90	Fair
Other commercial, services, and institutional	Woods/Grass 10/90	Good
Water and wastewater utilities	Woods/Grass 10/90	Good
Air transportation	Woods/Grass 25/75	Good
Commercial wholesale	Woods/Grass 25/75	Good
Government	Woods/Grass 25/75	Good
Institutional	Woods/Grass 25/75	Good
Lodging	Woods/Grass 25/75	Good
Multi-family (4 or more stories)	Woods/Grass 25/75	Good
Other commercial complexes	Woods/Grass 25/75	Good
Outdoor recreation	Woods/Grass 25/75	Good
Parking structure/lot	Woods/Grass 25/75	Good
Commercial retail	Woods/Grass 40/60	Good
Educational	Woods/Grass 40/60	Good
Multi-family (1-3 stories)	Woods/Grass 40/60	Good
Office park	Woods/Grass 40/60	Good
Other agricultural land	Woods/Grass 40/60	Good
Other industrial complexes	Woods/Grass 40/60	Good
Services	Woods/Grass 40/60	Good
Indoor cultural/ public assembly	Woods/Grass 50/50	Good
Industrial	Woods/Grass 50/50	Good
Other residential	Woods/Grass 50/50	Good
Single family/duplex	Woods/Grass 50/50	Good
Vacant land	Woods/Grass 50/50	Good
Wetlands	Woods/Grass 75/25	Good

North Mill Pond Watershed
Soil - Land Cover Map



North Mill Pond Watershed Soil-Land Cover Polygons

Land Cover	Hydrologic Condition	HSG	Area (AC)	CN
Brush	Good	A	58.81	30
Brush	Good	B	179.13	48
Brush	Good	C	32.85	65
Brush	Good	D	20.82	73
Impervious	n/a		930.36	98
Impervious2	n/a		5.67	98
Meadow	Good	A	23.27	30
Meadow	Good	B	1.73	58
Meadow	Good	C	0.00	71
Meadow	Good	D	0.12	78
Open Water	n/a		54.48	100
RxR	Good	A	1.28	76
RxR	Good	B	5.93	85
RxR	Good	C	0.20	89
RxR	Good	D	1.60	91
Woods	Good	A	60.28	30
Woods	Good	B	120.30	55
Woods	Good	C	80.53	70
Woods	Good	D	17.09	77
Woods-Grass 10-90	Fair	A	5.94	48
Woods-Grass 10-90	Fair	B	1.08	68
Woods-Grass 10-90	Fair	C	0.00	78
Woods-Grass 10-90	Fair	D	0.00	84
Woods-Grass 10-90	Good	A	69.10	38
Woods-Grass 10-90	Good	B	33.81	60
Woods-Grass 10-90	Good	C	2.13	74
Woods-Grass 10-90	Good	D	3.07	80
Woods-Grass 25-75	Good	A	5.89	36
Woods-Grass 25-75	Good	B	55.58	60
Woods-Grass 25-75	Good	C	10.22	73
Woods-Grass 25-75	Good	D	70.08	79
Woods-Grass 40-60	Good	A	5.06	33
Woods-Grass 40-60	Good	B	120.91	59
Woods-Grass 40-60	Good	C	7.04	72
Woods-Grass 40-60	Good	D	38.94	79
Woods-Grass 50-50	Good	A	16.68	32
Woods-Grass 50-50	Good	B	250.09	58
Woods-Grass 50-50	Good	C	7.28	72
Woods-Grass 50-50	Good	D	24.38	79
Woods-Grass 75-25	Good	A	16.01	30
Woods-Grass 75-25	Good	B	94.23	57
Woods-Grass 75-25	Good	C	120.21	71
Woods-Grass 75-25	Good	D	76.21	78
			2628.4	

Surface Description	Curve Number - Good Condition			
	A	B	C	D
Open Water	100	100	100	100
Impervious	98	98	98	98
Railroad Tracks	76	85	89	91
Grass	39	61	74	80
Meadow	30	58	71	78
Brush	30	48	65	73
Woods/Grass 10/90	38	60	74	80
Woods/Grass 25/75	36	60	73	79
Woods/Grass 40/60	33	59	72	79
Woods/Grass 50/50	32	58	72	79
Woods/Grass 60/40	31	57	72	78
Woods/Grass 75/25	30	57	71	78
Woods	30	55	70	77

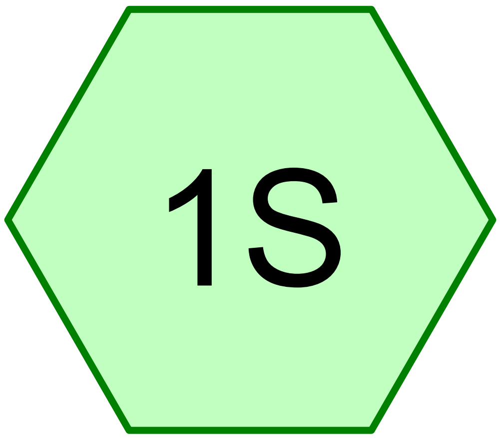
Note: CN values are for "good" hydrologic condition (>75% ground cover)

Surface Description	Curve Number - Fair Condition			
	A	B	C	D
Open Water	100	100	100	100
Impervious	98	98	98	98
Railroad Tracks	76	85	89	91
Grass	49	69	79	84
Meadow	30	58	71	78
Brush	35	56	70	77
Woods/Grass 10/90	48	68	78	84
Woods/Grass 25/75	46	67	78	83
Woods/Grass 40/60	44	65	77	82
Woods/Grass 50/50	43	65	76	82
Woods/Grass 60/40	41	64	75	81
Woods/Grass 75/25	39	62	75	80
Woods	36	60	73	79

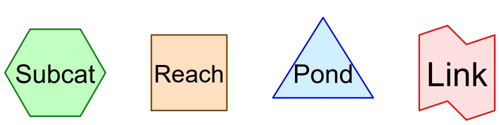
Note: CN values are for "fair" hydrologic condition (50-75% ground cover)

North Mill Pond Watershed Time of Concentration

Flow Path Segment	Type	Start Sta	Inv In	End Sta	Inv Out	Dia	A	P	Length	Slope	Surface	Notes
1	sheet	0	97.28	73	96.31	-	-	-	73	0.01329	Pavement	
2	shallow	73	96.31	478	92.55	-	-	-	405	0.00928	Grass	
3	pipe	478	88.55	2389	81.02	15	-	-	1911	0.00394	RCP	pipe size & material estimated and inv in estimated at 4' below ground elevation at grate
4	channel	2389	81.02	3584	75.09	-	41	74	1195	0.00496	Forest	A & P measured at typical section at max depth of 1' (estimated bankfull stage)
5	pipe	3584	75.09	3991	71.71	26	-	-	407	0.00831	RCP	pipe slope estimated as average slope between inlet segment 5 and outlet segment 7
6	pipe	3991	71.71	5936	55.54	36	-	-	1945	0.00831	RCP	pipe slope estimated as average slope between inlet segment 5 and outlet segment 7
7	pipe	5936	55.54	7933	38.95	48	-	-	1997	0.00831	RCP	pipe slope estimated as average slope between inlet segment 5 and outlet segment 7
8	channel	7933	38.95	8243	37.04	-	57	123	310	0.00616	Brush	A & P measured at typical section at max depth of 0.87' (elev. Difference between thalwet & height of land in right overbank)
9	pipe	8243	37.04	8344	37.00	60	-	-	101	0.00040	RCP	pipe size & material estimated
10	channel	8344	37.00	9090	34.40	-	148	210	746	0.00349	Brush	A & P measured at typical section at max depth of 1' (estimated bankfull stage)
11	pipe	9090	34.40	9189	33.76	60	-	-	99	0.00646	RCP	pipe size & material estimated
12	channel	9189	33.76	13125	19.25	-	15	27	3936	0.00369	Brush/Forest	A & P measured at typical section at max depth of 1' (estimated bankfull stage)
13	pipe	13125	19.25	13346	18.58	72	-	-	221	0.00303	RCP	pipe size & material estimated
14	channel	13346	18.58	13858	18.14	-	17	26	512	0.00086	Brush/Forest	A & P measured at typical section at max depth of 1' (estimated bankfull stage)
15	pipe	13858	18.14	14194	17.39	72	-	-	336	0.00223	RCP	pipe size & material estimated
16	channel	14194	17.39	14550	17.04	-	18	29	356	0.00098	Brush/Forest	A & P measured at typical section at max depth of 1' (estimated bankfull stage)
17	pipe	14550	17.04	15234	16.40	96	-	-	684	0.00094	CMP	pipe size & material estimated
18	channel	15234	16.40	15909	15.47	-	17	26	675	0.00138	Brush/Forest	A & P measured at typical section at max depth of 1' (estimated bankfull stage)
19	pipe	15909	15.47	16084	15.41	96	-	-	175	0.00034	CMP	pipe size & material estimated
20	channel	16084	15.41	16960	15.35	-	21	32	876	0.00007	Brush/Forest	A & P measured at typical section at max depth of 1' (estimated bankfull stage)
21	pipe	16960	15.35	17041	15.32	96	-	-	81	0.00037	CMP	pipe size & material estimated
22	channel	17041	15.32	17622	15.31	-	13	22	581	0.00002	Forest	A & P measured at typical section at max depth of 1' (estimated bankfull stage)
23	pipe	17622	15.31	17712	13.54	96	-	-	90	0.01967	CMP	pipe size & material estimated
24	channel	17712	13.54	18977	5.58	-	16	23	1265	0.00629	Forest	A & P measured at typical section at max depth of 1' (estimated bankfull stage)
25	pipe	18977	5.58	19479	3.54	72Hx144W	-	-	502	0.00406	Concrete Box	pipe size & material from field measurements
26	channel	19479	1.05	23320	-3.40	-	32	34	3841	0.00116	Cobble/Gravel	channel inverts from field measurments, channel geometry estimated from aerial photography and are based on a channel bottom width of 30', 2:1 side slopes, and flow depth of 1'
									23320			



North Mill Pond Watershed



Routing Diagram for NorthMillPond
Prepared by Headwaters Consulting, LLC, Printed 12/2/2022

Rainfall Events Listing (selected events)

Event#	Event Name	Storm Type	Curve	Mode	Duration (hours)	B/B	Depth (inches)	AMC
1	1-yr	NH-NorthMillPond_NRCC 24-hr S1	1-yr	Default	24.00	1	2.66	2
2	50-yr	NH-NorthMillPond_NRCC 24-hr S1	50-yr	Default	24.00	1	7.39	2
3	100-yr	NH-NorthMillPond_NRCC 24-hr S1	100-yr	Default	24.00	1	8.86	2

Area Listing (all nodes)

Area (acres)	CN	Description (subcatchment-numbers)
1	76	Ballasted RxR Tracks, HSG A (1S)
6	85	Ballasted RxR Tracks, HSG B (1S)
0	89	Ballasted RxR Tracks, HSG C (1S)
2	91	Ballasted RxR Tracks, HSG D (1S)
59	30	Brush, Good, HSG A (1S)
179	48	Brush, Good, HSG B (1S)
33	65	Brush, Good, HSG C (1S)
21	73	Brush, Good, HSG D (1S)
936	98	Impervious (1S)
23	30	Meadow, non-grazed, HSG A (1S)
2	58	Meadow, non-grazed, HSG B (1S)
0	78	Meadow, non-grazed, HSG D (1S)
54	100	Open Water (1S)
60	30	Woods, Good, HSG A (1S)
120	55	Woods, Good, HSG B (1S)
80	70	Woods, Good, HSG C (1S)
17	77	Woods, Good, HSG D (1S)
6	48	Woods/grass 10/90, Fair, HSG A (1S)
1	68	Woods/grass 10/90, Fair, HSG B (1S)
69	38	Woods/grass 10/90, Good, HSG A (1S)
34	60	Woods/grass 10/90, Good, HSG B (1S)
2	74	Woods/grass 10/90, Good, HSG C (1S)
3	80	Woods/grass 10/90, Good, HSG D (1S)
6	36	Woods/grass 25/75, Good, HSG A (1S)
56	60	Woods/grass 25/75, Good, HSG B (1S)
10	73	Woods/grass 25/75, Good, HSG C (1S)
70	79	Woods/grass 25/75, Good, HSG D (1S)
5	33	Woods/grass 40/60, Good, HSG A (1S)
121	59	Woods/grass 40/60, Good, HSG B (1S)
7	72	Woods/grass 40/60, Good, HSG C (1S)
39	79	Woods/grass 40/60, Good, HSG D (1S)
17	32	Woods/grass 50/50, Good, HSG A (1S)
250	58	Woods/grass 50/50, Good, HSG B (1S)
7	72	Woods/grass 50/50, Good, HSG C (1S)
24	79	Woods/grass 50/50, Good, HSG D (1S)
16	30	Woods/grass 75/25, Good, HSG A (1S)
94	57	Woods/grass 75/25, Good, HSG B (1S)
120	71	Woods/grass 75/25, Good, HSG C (1S)
76	78	Woods/grass 75/25, Good, HSG D (1S)
2,628	**73**	**TOTAL AREA**

Soil Listing (all nodes)

Area (acres)	Soil Group	Subcatchment Numbers
262	HSG A	1S
863	HSG B	1S
260	HSG C	1S
252	HSG D	1S
991	Other	1S
2,628		**TOTAL AREA**

Ground Covers (all nodes)

HSG-A (acres)	HSG-B (acres)	HSG-C (acres)	HSG-D (acres)	Other (acres)	Total (acres)	Ground Cover	Subcatchment Numbers
1	6	0	2	0	9	Ballasted RxR Tracks	1S
59	179	33	21	0	291	Brush, Good	1S
0	0	0	0	936	936	Impervious	1S
23	2	0	0	0	25	Meadow, non-grazed	1S
0	0	0	0	54	54	Open Water	1S
60	120	80	17	0	278	Woods, Good	1S
6	1	0	0	0	7	Woods/grass 10/90, Fair	1S
69	34	2	3	0	108	Woods/grass 10/90, Good	1S
6	56	10	70	0	142	Woods/grass 25/75, Good	1S
5	121	7	39	0	172	Woods/grass 40/60, Good	1S
17	250	7	24	0	298	Woods/grass 50/50, Good	1S
16	94	120	76	0	307	Woods/grass 75/25, Good	1S
262	**863**	**260**	**252**	**991**	**2,628**	**TOTAL AREA**	

Pipe Listing (all nodes)

Line#	Node Number	In-Invert (feet)	Out-Invert (feet)	Length (feet)	Slope (ft/ft)	n	Width (inches)	Diam/Height (inches)	Inside-Fill (inches)
1	1S	0.00	0.00	1,911.0	0.0039	0.015	0.0	15.0	0.0
2	1S	0.00	0.00	407.0	0.0083	0.015	0.0	26.0	0.0
3	1S	0.00	0.00	1,945.0	0.0083	0.015	0.0	36.0	0.0
4	1S	0.00	0.00	1,997.0	0.0083	0.015	0.0	48.0	0.0
5	1S	0.00	0.00	101.0	0.0004	0.015	0.0	60.0	0.0
6	1S	0.00	0.00	99.0	0.0065	0.015	0.0	60.0	0.0
7	1S	0.00	0.00	221.0	0.0030	0.015	0.0	72.0	0.0
8	1S	0.00	0.00	336.0	0.0022	0.015	0.0	72.0	0.0
9	1S	0.00	0.00	684.0	0.0009	0.025	0.0	96.0	0.0
10	1S	0.00	0.00	175.0	0.0003	0.025	0.0	96.0	0.0
11	1S	0.00	0.00	81.0	0.0004	0.025	0.0	96.0	0.0
12	1S	0.00	0.00	90.0	0.0197	0.025	0.0	96.0	0.0
13	1S	0.00	0.00	502.0	0.0041	0.015	144.0	72.0	0.0

Summary for Subcatchment 1S: North Mill Pond Watershed

Runoff = 133 cfs @ 20.74 hrs, Volume= 144 af, Depth> 0.66"

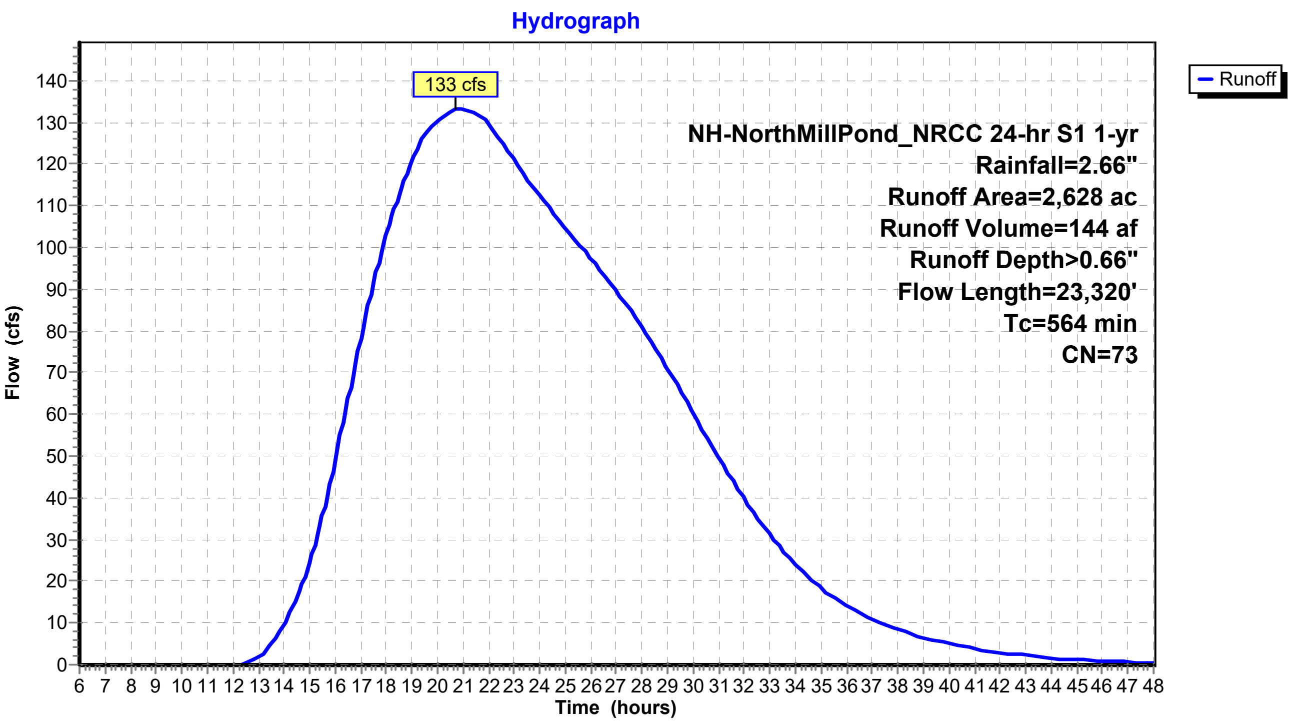
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 6.00-48.00 hrs, dt= 0.10 hrs
NH-NorthMillPond_NRCC 24-hr S1 1-yr Rainfall=2.66"

	Area (ac)	CN	Description
	59	30	Brush, Good, HSG A
	179	48	Brush, Good, HSG B
	33	65	Brush, Good, HSG C
	21	73	Brush, Good, HSG D
*	930	98	Impervious
*	6	98	Impervious
	23	30	Meadow, non-grazed, HSG A
	2	58	Meadow, non-grazed, HSG B
	0	78	Meadow, non-grazed, HSG D
*	54	100	Open Water
*	1	76	Ballasted RxR Tracks, HSG A
*	6	85	Ballasted RxR Tracks, HSG B
*	0	89	Ballasted RxR Tracks, HSG C
*	2	91	Ballasted RxR Tracks, HSG D
	60	30	Woods, Good, HSG A
	120	55	Woods, Good, HSG B
	80	70	Woods, Good, HSG C
	17	77	Woods, Good, HSG D
*	6	48	Woods/grass 10/90, Fair, HSG A
*	1	68	Woods/grass 10/90, Fair, HSG B
*	69	38	Woods/grass 10/90, Good, HSG A
*	34	60	Woods/grass 10/90, Good, HSG B
*	2	74	Woods/grass 10/90, Good, HSG C
*	3	80	Woods/grass 10/90, Good, HSG D
*	6	36	Woods/grass 25/75, Good, HSG A
*	56	60	Woods/grass 25/75, Good, HSG B
*	10	73	Woods/grass 25/75, Good, HSG C
*	70	79	Woods/grass 25/75, Good, HSG D
*	5	33	Woods/grass 40/60, Good, HSG A
*	121	59	Woods/grass 40/60, Good, HSG B
*	7	72	Woods/grass 40/60, Good, HSG C
*	39	79	Woods/grass 40/60, Good, HSG D
*	17	32	Woods/grass 50/50, Good, HSG A
*	250	58	Woods/grass 50/50, Good, HSG B
*	7	72	Woods/grass 50/50, Good, HSG C
*	24	79	Woods/grass 50/50, Good, HSG D
*	16	30	Woods/grass 75/25, Good, HSG A
*	94	57	Woods/grass 75/25, Good, HSG B
*	120	71	Woods/grass 75/25, Good, HSG C
*	76	78	Woods/grass 75/25, Good, HSG D
	2,628	73	Weighted Average
	1,638		62.31% Pervious Area
	991		37.69% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1	73	0.0133	1.12		**Sheet Flow, Segment 1** Smooth surfaces n= 0.011 P2= 3.33"
5	405	0.0093	1.45		**Shallow Concentrated Flow, Segment 2** Grassed Waterway Kv= 15.0 fps
11	1,911	0.0039	2.85	3.50	**Pipe Channel, Segment 3** 15.0" Round Area= 1.2 sf Perim= 3.9' r= 0.31' n= 0.015 Concrete sewer w/manholes & inlets
28	1,195	0.0050	0.71	29.06	**Channel Flow, Segment 4** Area= 41.0 sf Perim= 74.0' r= 0.55' n= 0.100 Earth, dense brush, high stage
1	407	0.0083	6.00	22.11	**Pipe Channel, Segment 5** 26.0" Round Area= 3.7 sf Perim= 6.8' r= 0.54' n= 0.015 Concrete sewer w/manholes & inlets
4	1,945	0.0083	7.45	52.66	**Pipe Channel, Segment 6** 36.0" Round Area= 7.1 sf Perim= 9.4' r= 0.75' n= 0.015 Concrete sewer w/manholes & inlets
4	1,997	0.0083	9.03	113.42	**Pipe Channel, Segment 7** 48.0" Round Area= 12.6 sf Perim= 12.6' r= 1.00' n= 0.015 Concrete sewer w/manholes & inlets
7	310	0.0062	0.70	39.94	**Channel Flow, Segment 8** Area= 57.0 sf Perim= 123.0' r= 0.46' n= 0.100 Earth, dense brush, high stage
1	101	0.0004	2.30	45.14	**Pipe Channel, Segment 9** 60.0" Round Area= 19.6 sf Perim= 15.7' r= 1.25' n= 0.015 Concrete sewer w/manholes & inlets
18	746	0.0035	0.70	103.04	**Channel Flow, Segment 10** Area= 148.0 sf Perim= 210.0' r= 0.70' n= 0.100 Earth, dense brush, high stage
0	99	0.0065	9.27	181.98	**Pipe Channel, Segment 11** 60.0" Round Area= 19.6 sf Perim= 15.7' r= 1.25' n= 0.015 Concrete sewer w/manholes & inlets
107	3,936	0.0037	0.61	9.16	**Channel Flow, Segment 12** Area= 15.0 sf Perim= 27.0' r= 0.56' n= 0.100 Earth, dense brush, high stage
1	221	0.0030	7.11	201.04	**Pipe Channel, Segment 13** 72.0" Round Area= 28.3 sf Perim= 18.8' r= 1.50' n= 0.015 Concrete sewer w/manholes & inlets
25	512	0.0009	0.34	5.71	**Channel Flow, Segment 14** Area= 17.0 sf Perim= 26.0' r= 0.65' n= 0.100 Earth, dense brush, high stage
1	336	0.0022	6.09	172.16	**Pipe Channel, Segment 15** 72.0" Round Area= 28.3 sf Perim= 18.8' r= 1.50' n= 0.015 Concrete sewer w/manholes & inlets
17	356	0.0010	0.34	6.15	**Channel Flow, Segment 16** Area= 18.0 sf Perim= 29.0' r= 0.62' n= 0.100 Earth, dense brush, high stage
4	684	0.0009	2.83	142.28	**Pipe Channel, Segment 17** 96.0" Round Area= 50.3 sf Perim= 25.1' r= 2.00' n= 0.025 Corrugated metal
27	675	0.0014	0.42	7.12	**Channel Flow, Segment 18** Area= 17.0 sf Perim= 26.0' r= 0.65'

					n= 0.100 Earth, dense brush, high stage
2	175	0.0003	1.63	82.15	**Pipe Channel, Segment 19** 96.0" Round Area= 50.3 sf Perim= 25.1' r= 2.00' n= 0.025 Corrugated metal
130	876	0.0001	0.11	2.36	**Channel Flow, Segment 20** Area= 21.0 sf Perim= 32.0' r= 0.66' n= 0.100
1	81	0.0004	1.89	94.86	**Pipe Channel, Segment 21** 96.0" Round Area= 50.3 sf Perim= 25.1' r= 2.00' n= 0.025 Corrugated metal
93	581	0.0001	0.10	1.36	**Channel Flow, Segment 22** Area= 13.0 sf Perim= 22.0' r= 0.59' n= 0.100 Earth, dense brush, high stage
0	90	0.0197	13.24	665.68	**Pipe Channel, Segment 23** 96.0" Round Area= 50.3 sf Perim= 25.1' r= 2.00' n= 0.025 Corrugated metal
23	1,265	0.0063	0.93	14.82	**Channel Flow, Segment 24** Area= 16.0 sf Perim= 23.0' r= 0.70' n= 0.100
1	502	0.0041	10.07	725.00	**Pipe Channel, Segment 25** 144.0" x 72.0" Box Area= 72.0 sf Perim= 36.0' r= 2.00' n= 0.015 Concrete sewer w/manholes & inlets
52	3,841	0.0012	1.24	39.55	**Channel Flow, Segment 26** Area= 32.0 sf Perim= 34.0' r= 0.94' n= 0.040 Earth, cobble bottom, clean sides
564	23,320	Total			

Subcatchment 1S: North Mill Pond Watershed



Summary for Subcatchment 1S: North Mill Pond Watershed

Runoff = 908 cfs @ 19.52 hrs, Volume= 936 af, Depth> 4.27"

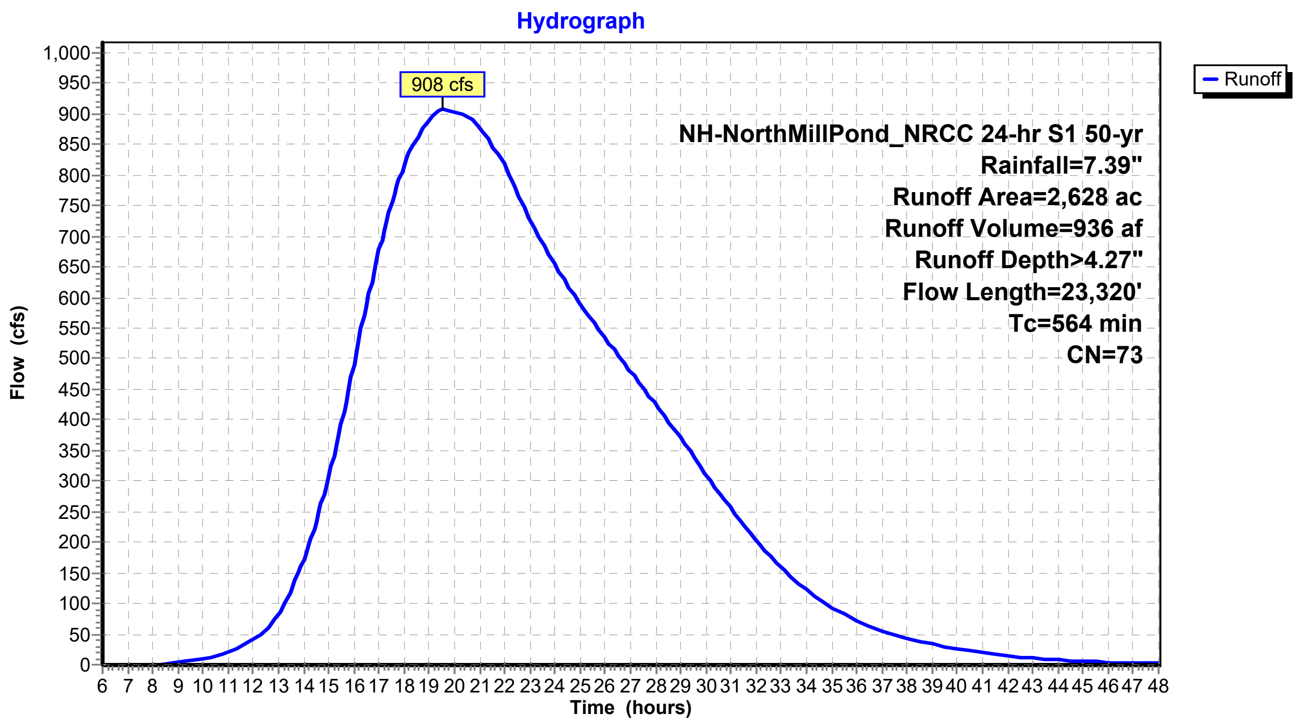
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 6.00-48.00 hrs, dt= 0.10 hrs
NH-NorthMillPond_NRCC 24-hr S1 50-yr Rainfall=7.39"

	Area (ac)	CN	Description
	59	30	Brush, Good, HSG A
	179	48	Brush, Good, HSG B
	33	65	Brush, Good, HSG C
	21	73	Brush, Good, HSG D
*	930	98	Impervious
*	6	98	Impervious
	23	30	Meadow, non-grazed, HSG A
	2	58	Meadow, non-grazed, HSG B
	0	78	Meadow, non-grazed, HSG D
*	54	100	Open Water
*	1	76	Ballasted RxR Tracks, HSG A
*	6	85	Ballasted RxR Tracks, HSG B
*	0	89	Ballasted RxR Tracks, HSG C
*	2	91	Ballasted RxR Tracks, HSG D
	60	30	Woods, Good, HSG A
	120	55	Woods, Good, HSG B
	80	70	Woods, Good, HSG C
	17	77	Woods, Good, HSG D
*	6	48	Woods/grass 10/90, Fair, HSG A
*	1	68	Woods/grass 10/90, Fair, HSG B
*	69	38	Woods/grass 10/90, Good, HSG A
*	34	60	Woods/grass 10/90, Good, HSG B
*	2	74	Woods/grass 10/90, Good, HSG C
*	3	80	Woods/grass 10/90, Good, HSG D
*	6	36	Woods/grass 25/75, Good, HSG A
*	56	60	Woods/grass 25/75, Good, HSG B
*	10	73	Woods/grass 25/75, Good, HSG C
*	70	79	Woods/grass 25/75, Good, HSG D
*	5	33	Woods/grass 40/60, Good, HSG A
*	121	59	Woods/grass 40/60, Good, HSG B
*	7	72	Woods/grass 40/60, Good, HSG C
*	39	79	Woods/grass 40/60, Good, HSG D
*	17	32	Woods/grass 50/50, Good, HSG A
*	250	58	Woods/grass 50/50, Good, HSG B
*	7	72	Woods/grass 50/50, Good, HSG C
*	24	79	Woods/grass 50/50, Good, HSG D
*	16	30	Woods/grass 75/25, Good, HSG A
*	94	57	Woods/grass 75/25, Good, HSG B
*	120	71	Woods/grass 75/25, Good, HSG C
*	76	78	Woods/grass 75/25, Good, HSG D
	2,628	73	Weighted Average
	1,638		62.31% Pervious Area
	991		37.69% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1	73	0.0133	1.12		**Sheet Flow, Segment 1** Smooth surfaces n= 0.011 P2= 3.33"
5	405	0.0093	1.45		**Shallow Concentrated Flow, Segment 2** Grassed Waterway Kv= 15.0 fps
11	1,911	0.0039	2.85	3.50	**Pipe Channel, Segment 3** 15.0" Round Area= 1.2 sf Perim= 3.9' r= 0.31' n= 0.015 Concrete sewer w/manholes & inlets
28	1,195	0.0050	0.71	29.06	**Channel Flow, Segment 4** Area= 41.0 sf Perim= 74.0' r= 0.55' n= 0.100 Earth, dense brush, high stage
1	407	0.0083	6.00	22.11	**Pipe Channel, Segment 5** 26.0" Round Area= 3.7 sf Perim= 6.8' r= 0.54' n= 0.015 Concrete sewer w/manholes & inlets
4	1,945	0.0083	7.45	52.66	**Pipe Channel, Segment 6** 36.0" Round Area= 7.1 sf Perim= 9.4' r= 0.75' n= 0.015 Concrete sewer w/manholes & inlets
4	1,997	0.0083	9.03	113.42	**Pipe Channel, Segment 7** 48.0" Round Area= 12.6 sf Perim= 12.6' r= 1.00' n= 0.015 Concrete sewer w/manholes & inlets
7	310	0.0062	0.70	39.94	**Channel Flow, Segment 8** Area= 57.0 sf Perim= 123.0' r= 0.46' n= 0.100 Earth, dense brush, high stage
1	101	0.0004	2.30	45.14	**Pipe Channel, Segment 9** 60.0" Round Area= 19.6 sf Perim= 15.7' r= 1.25' n= 0.015 Concrete sewer w/manholes & inlets
18	746	0.0035	0.70	103.04	**Channel Flow, Segment 10** Area= 148.0 sf Perim= 210.0' r= 0.70' n= 0.100 Earth, dense brush, high stage
0	99	0.0065	9.27	181.98	**Pipe Channel, Segment 11** 60.0" Round Area= 19.6 sf Perim= 15.7' r= 1.25' n= 0.015 Concrete sewer w/manholes & inlets
107	3,936	0.0037	0.61	9.16	**Channel Flow, Segment 12** Area= 15.0 sf Perim= 27.0' r= 0.56' n= 0.100 Earth, dense brush, high stage
1	221	0.0030	7.11	201.04	**Pipe Channel, Segment 13** 72.0" Round Area= 28.3 sf Perim= 18.8' r= 1.50' n= 0.015 Concrete sewer w/manholes & inlets
25	512	0.0009	0.34	5.71	**Channel Flow, Segment 14** Area= 17.0 sf Perim= 26.0' r= 0.65' n= 0.100 Earth, dense brush, high stage
1	336	0.0022	6.09	172.16	**Pipe Channel, Segment 15** 72.0" Round Area= 28.3 sf Perim= 18.8' r= 1.50' n= 0.015 Concrete sewer w/manholes & inlets
17	356	0.0010	0.34	6.15	**Channel Flow, Segment 16** Area= 18.0 sf Perim= 29.0' r= 0.62' n= 0.100 Earth, dense brush, high stage
4	684	0.0009	2.83	142.28	**Pipe Channel, Segment 17** 96.0" Round Area= 50.3 sf Perim= 25.1' r= 2.00' n= 0.025 Corrugated metal
27	675	0.0014	0.42	7.12	**Channel Flow, Segment 18** Area= 17.0 sf Perim= 26.0' r= 0.65'

n= 0.100 Earth, dense brush, high stage					
2	175	0.0003	1.63	82.15	**Pipe Channel, Segment 19** 96.0" Round Area= 50.3 sf Perim= 25.1' r= 2.00' n= 0.025 Corrugated metal
130	876	0.0001	0.11	2.36	**Channel Flow, Segment 20** Area= 21.0 sf Perim= 32.0' r= 0.66' n= 0.100
1	81	0.0004	1.89	94.86	**Pipe Channel, Segment 21** 96.0" Round Area= 50.3 sf Perim= 25.1' r= 2.00' n= 0.025 Corrugated metal
93	581	0.0001	0.10	1.36	**Channel Flow, Segment 22** Area= 13.0 sf Perim= 22.0' r= 0.59' n= 0.100 Earth, dense brush, high stage
0	90	0.0197	13.24	665.68	**Pipe Channel, Segment 23** 96.0" Round Area= 50.3 sf Perim= 25.1' r= 2.00' n= 0.025 Corrugated metal
23	1,265	0.0063	0.93	14.82	**Channel Flow, Segment 24** Area= 16.0 sf Perim= 23.0' r= 0.70' n= 0.100
1	502	0.0041	10.07	725.00	**Pipe Channel, Segment 25** 144.0" x 72.0" Box Area= 72.0 sf Perim= 36.0' r= 2.00' n= 0.015 Concrete sewer w/manholes & inlets
52	3,841	0.0012	1.24	39.55	**Channel Flow, Segment 26** Area= 32.0 sf Perim= 34.0' r= 0.94' n= 0.040 Earth, cobble bottom, clean sides
564	23,320	Total			

Subcatchment 1S: North Mill Pond Watershed



Summary for Subcatchment 1S: North Mill Pond Watershed

Runoff = 1,179 cfs @ 19.49 hrs, Volume= 1,221 af, Depth> 5.58"

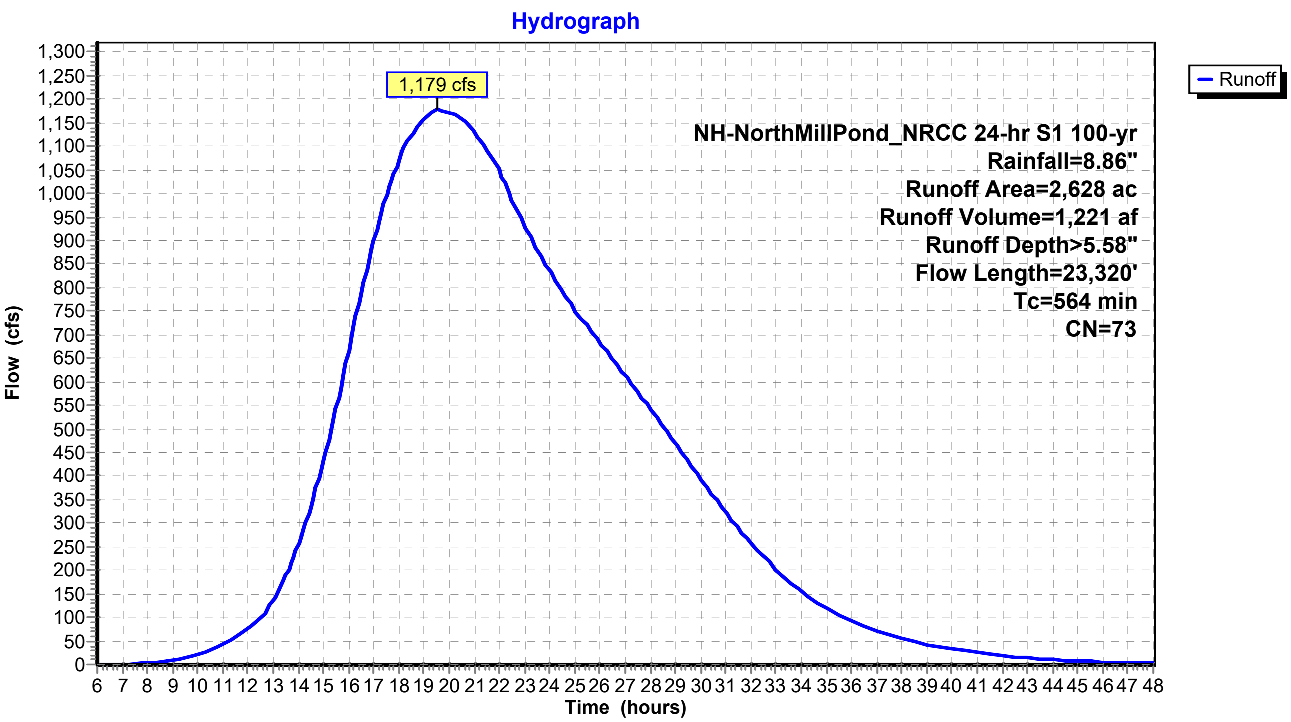
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 6.00-48.00 hrs, dt= 0.10 hrs
NH-NorthMillPond_NRCC 24-hr S1 100-yr Rainfall=8.86"

	Area (ac)	CN	Description
	59	30	Brush, Good, HSG A
	179	48	Brush, Good, HSG B
	33	65	Brush, Good, HSG C
	21	73	Brush, Good, HSG D
*	930	98	Impervious
*	6	98	Impervious
	23	30	Meadow, non-grazed, HSG A
	2	58	Meadow, non-grazed, HSG B
	0	78	Meadow, non-grazed, HSG D
*	54	100	Open Water
*	1	76	Ballasted RxR Tracks, HSG A
*	6	85	Ballasted RxR Tracks, HSG B
*	0	89	Ballasted RxR Tracks, HSG C
*	2	91	Ballasted RxR Tracks, HSG D
	60	30	Woods, Good, HSG A
	120	55	Woods, Good, HSG B
	80	70	Woods, Good, HSG C
	17	77	Woods, Good, HSG D
*	6	48	Woods/grass 10/90, Fair, HSG A
*	1	68	Woods/grass 10/90, Fair, HSG B
*	69	38	Woods/grass 10/90, Good, HSG A
*	34	60	Woods/grass 10/90, Good, HSG B
*	2	74	Woods/grass 10/90, Good, HSG C
*	3	80	Woods/grass 10/90, Good, HSG D
*	6	36	Woods/grass 25/75, Good, HSG A
*	56	60	Woods/grass 25/75, Good, HSG B
*	10	73	Woods/grass 25/75, Good, HSG C
*	70	79	Woods/grass 25/75, Good, HSG D
*	5	33	Woods/grass 40/60, Good, HSG A
*	121	59	Woods/grass 40/60, Good, HSG B
*	7	72	Woods/grass 40/60, Good, HSG C
*	39	79	Woods/grass 40/60, Good, HSG D
*	17	32	Woods/grass 50/50, Good, HSG A
*	250	58	Woods/grass 50/50, Good, HSG B
*	7	72	Woods/grass 50/50, Good, HSG C
*	24	79	Woods/grass 50/50, Good, HSG D
*	16	30	Woods/grass 75/25, Good, HSG A
*	94	57	Woods/grass 75/25, Good, HSG B
*	120	71	Woods/grass 75/25, Good, HSG C
*	76	78	Woods/grass 75/25, Good, HSG D
	2,628	73	Weighted Average
	1,638		62.31% Pervious Area
	991		37.69% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
1	73	0.0133	1.12		**Sheet Flow, Segment 1** Smooth surfaces n= 0.011 P2= 3.33"
5	405	0.0093	1.45		**Shallow Concentrated Flow, Segment 2** Grassed Waterway Kv= 15.0 fps
11	1,911	0.0039	2.85	3.50	**Pipe Channel, Segment 3** 15.0" Round Area= 1.2 sf Perim= 3.9' r= 0.31' n= 0.015 Concrete sewer w/manholes & inlets
28	1,195	0.0050	0.71	29.06	**Channel Flow, Segment 4** Area= 41.0 sf Perim= 74.0' r= 0.55' n= 0.100 Earth, dense brush, high stage
1	407	0.0083	6.00	22.11	**Pipe Channel, Segment 5** 26.0" Round Area= 3.7 sf Perim= 6.8' r= 0.54' n= 0.015 Concrete sewer w/manholes & inlets
4	1,945	0.0083	7.45	52.66	**Pipe Channel, Segment 6** 36.0" Round Area= 7.1 sf Perim= 9.4' r= 0.75' n= 0.015 Concrete sewer w/manholes & inlets
4	1,997	0.0083	9.03	113.42	**Pipe Channel, Segment 7** 48.0" Round Area= 12.6 sf Perim= 12.6' r= 1.00' n= 0.015 Concrete sewer w/manholes & inlets
7	310	0.0062	0.70	39.94	**Channel Flow, Segment 8** Area= 57.0 sf Perim= 123.0' r= 0.46' n= 0.100 Earth, dense brush, high stage
1	101	0.0004	2.30	45.14	**Pipe Channel, Segment 9** 60.0" Round Area= 19.6 sf Perim= 15.7' r= 1.25' n= 0.015 Concrete sewer w/manholes & inlets
18	746	0.0035	0.70	103.04	**Channel Flow, Segment 10** Area= 148.0 sf Perim= 210.0' r= 0.70' n= 0.100 Earth, dense brush, high stage
0	99	0.0065	9.27	181.98	**Pipe Channel, Segment 11** 60.0" Round Area= 19.6 sf Perim= 15.7' r= 1.25' n= 0.015 Concrete sewer w/manholes & inlets
107	3,936	0.0037	0.61	9.16	**Channel Flow, Segment 12** Area= 15.0 sf Perim= 27.0' r= 0.56' n= 0.100 Earth, dense brush, high stage
1	221	0.0030	7.11	201.04	**Pipe Channel, Segment 13** 72.0" Round Area= 28.3 sf Perim= 18.8' r= 1.50' n= 0.015 Concrete sewer w/manholes & inlets
25	512	0.0009	0.34	5.71	**Channel Flow, Segment 14** Area= 17.0 sf Perim= 26.0' r= 0.65' n= 0.100 Earth, dense brush, high stage
1	336	0.0022	6.09	172.16	**Pipe Channel, Segment 15** 72.0" Round Area= 28.3 sf Perim= 18.8' r= 1.50' n= 0.015 Concrete sewer w/manholes & inlets
17	356	0.0010	0.34	6.15	**Channel Flow, Segment 16** Area= 18.0 sf Perim= 29.0' r= 0.62' n= 0.100 Earth, dense brush, high stage
4	684	0.0009	2.83	142.28	**Pipe Channel, Segment 17** 96.0" Round Area= 50.3 sf Perim= 25.1' r= 2.00' n= 0.025 Corrugated metal
27	675	0.0014	0.42	7.12	**Channel Flow, Segment 18** Area= 17.0 sf Perim= 26.0' r= 0.65'

n= 0.100 Earth, dense brush, high stage					
2	175	0.0003	1.63	82.15	**Pipe Channel, Segment 19** 96.0" Round Area= 50.3 sf Perim= 25.1' r= 2.00' n= 0.025 Corrugated metal
130	876	0.0001	0.11	2.36	**Channel Flow, Segment 20** Area= 21.0 sf Perim= 32.0' r= 0.66' n= 0.100
1	81	0.0004	1.89	94.86	**Pipe Channel, Segment 21** 96.0" Round Area= 50.3 sf Perim= 25.1' r= 2.00' n= 0.025 Corrugated metal
93	581	0.0001	0.10	1.36	**Channel Flow, Segment 22** Area= 13.0 sf Perim= 22.0' r= 0.59' n= 0.100 Earth, dense brush, high stage
0	90	0.0197	13.24	665.68	**Pipe Channel, Segment 23** 96.0" Round Area= 50.3 sf Perim= 25.1' r= 2.00' n= 0.025 Corrugated metal
23	1,265	0.0063	0.93	14.82	**Channel Flow, Segment 24** Area= 16.0 sf Perim= 23.0' r= 0.70' n= 0.100
1	502	0.0041	10.07	725.00	**Pipe Channel, Segment 25** 144.0" x 72.0" Box Area= 72.0 sf Perim= 36.0' r= 2.00' n= 0.015 Concrete sewer w/manholes & inlets
52	3,841	0.0012	1.24	39.55	**Channel Flow, Segment 26** Area= 32.0 sf Perim= 34.0' r= 0.94' n= 0.040 Earth, cobble bottom, clean sides
564	23,320	Total			

Subcatchment 1S: North Mill Pond Watershed



APPENDIX 2

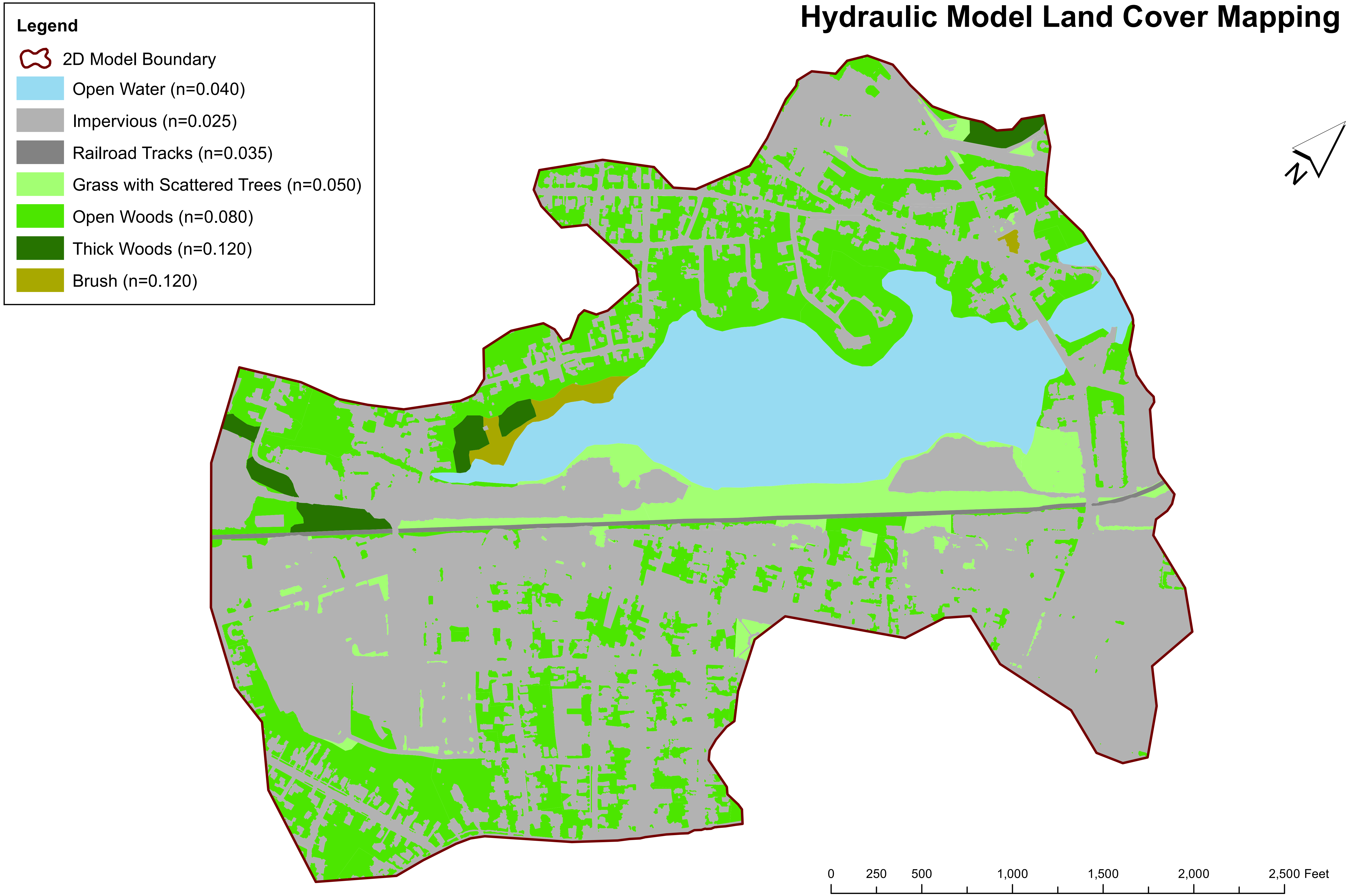
SUPPORTING DOCUMENTATION FOR HYDRAULIC MODELS

Hydraulic Model Land Cover Map

StreamStats Output

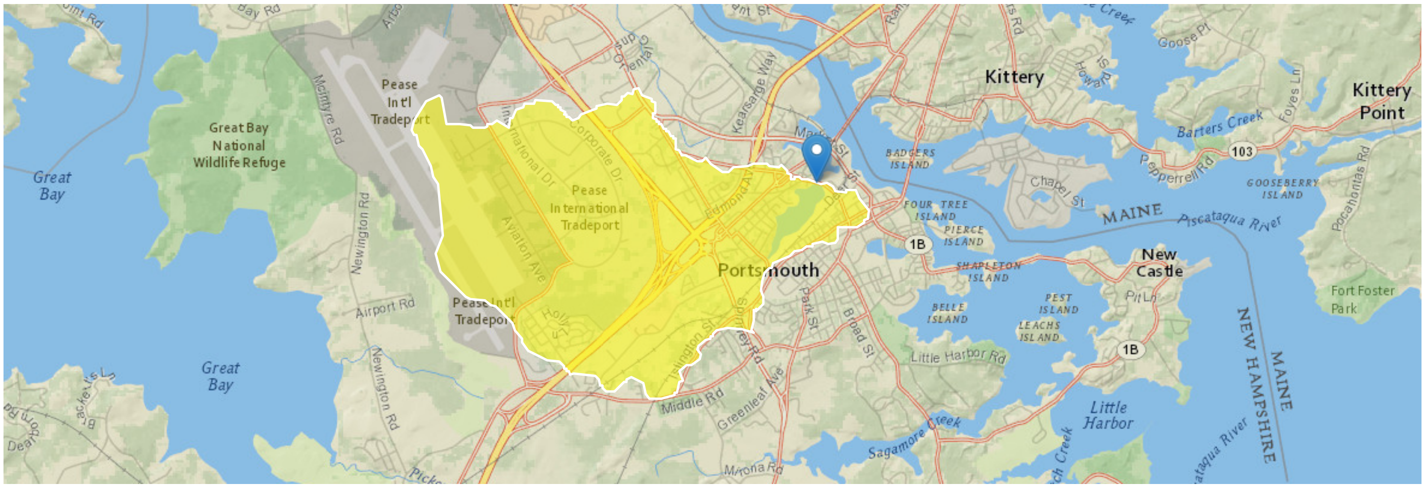
NOAA Average October Precipitation for Portland, ME

Maplewood Avenue Bridge Replacement
Hydraulic Model Land Cover Mapping



StreamStats Report - North Mill Pond at Maplewood Ave.

Region ID: NH
Workspace ID: NH20221003123325873000
Clicked Point (Latitude, Longitude): 43.07969, -70.76530
Time: 2022-10-03 08:33:51 -0400



Collapse All

Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
APRAVPRE	Mean April Precipitation	4.429	inches
BSLDEM30M	Mean basin slope computed from 30 m DEM	1.47	percent
CONIF	Percentage of land surface covered by coniferous forest	6.3785	percent
CSL10_85	Change in elevation divided by length between points 10 and 85 percent of distance along main channel to basin divide - main channel method not known	19	feet per mi
DRNAREA	Area that drains to a point on a stream	4.16	square miles
ELEVMAX	Maximum basin elevation	101.072	feet
MIXFOR	Percentage of land area covered by mixed deciduous and coniferous forest	2.2681	percent
PREBC0103	Mean annual precipitation of basin centroid for January 1 to March 15 winter period	9.25	inches
PREG_03_05	Mean precipitation at gaging station location for March 16 to May 31 spring period	9.6	inches
PREG_06_10	Mean precipitation at gaging station location for June to October summer period	17.2	inches
TEMP	Mean Annual Temperature	46.223	degrees F
TEMP_06_10	Basinwide average temperature for June to October summer period	62.036	degrees F
WETLAND	Percentage of Wetlands	7.3067	percent

Peak-Flow Statistics

Peak-Flow Statistics Parameters [Peak Flow Statewide SIR2008 5206]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	4.16	square miles	0.7	1290
APRAVPRE	Mean April Precipitation	4.429	inches	2.79	6.23
WETLAND	Percent Wetlands	7.3067	percent	0	21.8
CSL10_85	Stream Slope 10 and 85 Method	19	feet per mi	5.43	543

Peak-Flow Statistics Flow Report [Peak Flow Statewide SIR2008 5206]

PIl: Prediction Interval-Lower, Plu: Prediction Interval-Upper, ASEp: Average Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	PIl	Plu	ASEp	Equiv. Yrs.
50-percent AEP flood	115	ft^3/s	69.6	190	30.1	3.2
20-percent AEP flood	196	ft^3/s	117	329	31.1	4.7
10-percent AEP flood	266	ft^3/s	155	455	32.3	6.2
4-percent AEP flood	363	ft^3/s	204	644	34.3	8
2-percent AEP flood	445	ft^3/s	243	815	36.4	9
1-percent AEP flood	546	ft^3/s	287	1040	38.6	9.8
0.2-percent AEP flood	799	ft^3/s	386	1650	44.1	11

Peak-Flow Statistics Citations

Olson, S.A.,2009, Estimation of flood discharges at selected recurrence intervals for streams in New Hampshire: U.S.Geological Survey Scientific Investigations Report 2008-5206, 57 p. (http://pubs.usgs.gov/sir/2008/5206/)

Flow-Duration Statistics

Flow-Duration Statistics Parameters [Low Flow Statewide]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	4.16	square miles	3.26	689
PREG_06_10	Jun to Oct Gage Precipitation	17.2	inches	16.5	23.1
TEMP	Mean Annual Temperature	46.223	degrees F	36	48.7

Flow-Duration Statistics Flow Report [Low Flow Statewide]

PIl: Prediction Interval-Lower, Plu: Prediction Interval-Upper, ASEp: Average Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	PIl	Plu	SE	ASEp
60 Percent Duration	1.94	ft^3/s	1.41	2.6	18	18
70 Percent Duration	1.21	ft^3/s	0.84	1.68	20.6	20.6
80 Percent Duration	0.64	ft^3/s	0.388	0.991	28	28
90 Percent Duration	0.289	ft^3/s	0.147	0.509	37.5	37.5
95 Percent Duration	0.164	ft^3/s	0.0741	0.313	44.1	44.1
98 Percent Duration	0.0948	ft^3/s	0.0356	0.203	54.3	54.3

Flow-Duration Statistics Citations

Flynn, R.H. and Tasker, G.D.,2002, Development of Regression Equations to Estimate Flow Durations and Low-Flow-Frequency Statistics in New Hampshire Streams: U.S.Geological Survey Scientific Investigations Report 02-4298, 66 p. (http://pubs.water.usgs.gov/wrir02-4298)

Seasonal Flow Statistics

Seasonal Flow Statistics Parameters [Low Flow Statewide]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	4.16	square miles	3.26	689
CONIF	Percent Coniferous Forest	6.3785	percent	3.07	56.2
PREBC0103	Jan to Mar Basin Centroid Precip	9.25	inches	5.79	15.1
BSLDEM30M	Mean Basin Slope from 30m DEM	1.47	percent	3.19	38.1
MIXFOR	Percent Mixed Forest	2.2681	percent	6.21	46.1
PREG_03_05	Mar to May Gage Precipitation	9.6	inches	6.83	11.5
TEMP	Mean Annual Temperature	46.223	degrees F	36	48.7
TEMP_06_10	Jun to Oct Mean Basinwide Temp	62.036	degrees F	52.9	64.4
PREG_06_10	Jun to Oct Gage Precipitation	17.2	inches	16.5	23.1
ELEVMAX	Maximum Basin Elevation	101.072	feet	260	6290

Seasonal Flow Statistics Disclaimers [Low Flow Statewide]

One or more of the parameters is outside the suggested range. Estimates were extrapolated with unknown errors.

Seasonal Flow Statistics Flow Report [Low Flow Statewide]

Statistic	Value	Unit
Jan to Mar15 60 Percent Flow	4.67	ft^3/s
Jan to Mar15 70 Percent Flow	3.99	ft^3/s
Jan to Mar15 80 Percent Flow	3.25	ft^3/s
Jan to Mar15 90 Percent Flow	2.3	ft^3/s
Jan to Mar15 95 Percent Flow	1.77	ft^3/s
Jan to Mar15 98 Percent Flow	1.32	ft^3/s
Jan to Mar15 7 Day 2 Year Low Flow	2.95	ft^3/s
Jan to Mar15 7 Day 10 Year Low Flow	1.63	ft^3/s
Mar16 to May 60 Percent Flow	4.82	ft^3/s
Mar16 to May 70 Percent Flow	4.02	ft^3/s
Mar16 to May 80 Percent Flow	4.18	ft^3/s
Mar16 to May 90 Percent Flow	3.76	ft^3/s
Mar16 to May 95 Percent Flow	3.4	ft^3/s
Mar16 to May 98 Percent Flow	2.92	ft^3/s
Mar16 to May 7 Day 2 Year Low Flow	3.39	ft^3/s
Mar16 to May 7 Day 10 Year Low Flow	1.87	ft^3/s
Jun to Oct 60 Percent Flow	0.536	ft^3/s
Jun to Oct 70 Percent Flow	0.381	ft^3/s
Jun to Oct 80 Percent Flow	0.225	ft^3/s
Jun to Oct 90 Percent Flow	0.134	ft^3/s
Jun to Oct 95 Percent Flow	0.0875	ft^3/s
Jun to Oct 98 Percent Flow	0.0703	ft^3/s
Jun to Oct 7 Day 2 Year Low Flow	0.157	ft^3/s
Jun to Oct 7 Day 10 Year Low Flow	0.0492	ft^3/s
Nov to Dec 60 Percent Flow	2.14	ft^3/s
Nov to Dec 70 Percent Flow	1.37	ft^3/s
Nov to Dec 80 Percent Flow	0.814	ft^3/s
Nov to Dec 90 Percent Flow	0.42	ft^3/s
Nov to Dec 95 Percent Flow	0.227	ft^3/s
Nov to Dec 98 Percent Flow	0.107	ft^3/s
Oct to Nov 7 Day 2 Year Low Flow	0.848	ft^3/s
Oct to Nov 7 Day 10 Year Low Flow	0.182	ft^3/s

Seasonal Flow Statistics Citations

Flynn, R.H. and Tasker, G.D.,2002, Development of Regression Equations to Estimate Flow Durations and Low-Flow-Frequency Statistics in New Hampshire Streams: U.S.Geological Survey Scientific Investigations Report 02-4298, 66 p. (http://pubs.water.usgs.gov/wrir02-4298)

Low-Flow Statistics

Low-Flow Statistics Parameters [Low Flow Statewide]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	4.16	square miles	3.26	689
TEMP	Mean Annual Temperature	46.223	degrees F	36	48.7
PREG_06_10	Jun to Oct Gage Precipitation	17.2	inches	16.5	23.1

Low-Flow Statistics Flow Report [Low Flow Statewide]

PII: Prediction Interval-Lower, PIu: Prediction Interval-Upper, ASEp: Average Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	PII	PIu	SE	ASEp
7 Day 2 Year Low Flow	0.154	ft^3/s	0.0553	0.327	55.7	55.7
7 Day 10 Year Low Flow	0.0477	ft^3/s	0.0111	0.125	79.4	79.4

Low-Flow Statistics Citations

Flynn, R.H. and Tasker, G.D.,2002, Development of Regression Equations to Estimate Flow Durations and Low-Flow-Frequency Statistics in New Hampshire Streams: U.S.Geological Survey Scientific Investigations Report 02-4298, 66 p. (http://pubs.water.usgs.gov/wrir02-4298)

Application Version: 4.10.1
StreamStats Services Version: 1.2.22
NSS Services Version: 2.2.1

PORTLAND, ME

OCTOBER NORMAL (1991-2020)						
DATE	HIGH	LOW	AVE	PRECIP	SNOW	DEPTH
10/1	65	46	56	0.15	0.0	0
10/2	65	46	55	0.15	0.0	0
10/3	64	46	55	0.15	0.0	0
10/4	64	45	55	0.15	0.0	0
10/5	64	45	54	0.16	0.0	0
10/6	63	44	54	0.15	0.0	0
10/7	63	44	53	0.18	0.0	0
10/8	62	44	53	0.17	0.0	0
10/9	62	43	53	0.18	0.0	0
10/10	62	43	52	0.18	0.0	0
10/11	61	43	52	0.17	0.0	0
10/12	61	42	52	0.17	0.0	0
10/13	61	42	51	0.17	0.0	0
10/14	60	42	51	0.18	0.0	0
10/15	60	41	51	0.18	0.0	0
10/16	59	41	50	0.18	0.0	0
10/17	59	41	50	0.18	0.0	0
10/18	59	40	50	0.18	0.0	0
10/19	58	40	49	0.17	0.0	0
10/20	58	40	49	0.19	0.1	0
10/21	58	39	49	0.18	0.0	0
10/22	57	39	48	0.18	0.0	0
10/23	57	39	48	0.18	0.0	0
10/24	57	38	48	0.17	0.0	0
10/25	56	38	47	0.17	0.0	0
10/26	56	38	47	0.17	0.0	0
10/27	56	38	47	0.17	0.0	0
10/28	55	37	46	0.16	0.0	0
10/29	55	37	46	0.17	0.1	0
10/30	54	37	46	0.15	0.0	T
10/31	54	36	45	0.16	0.0	T

MONTH	59.5	41.0	50.3	5.25	0.2

https://www.weather.gov/media/gyx/climo/PWMNormal10.pdf