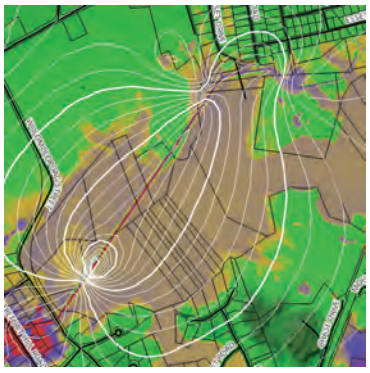
BASIS OF DESIGN: FLOW CONTROL STRUCTURE IN GREAT CEDAR SWAMP

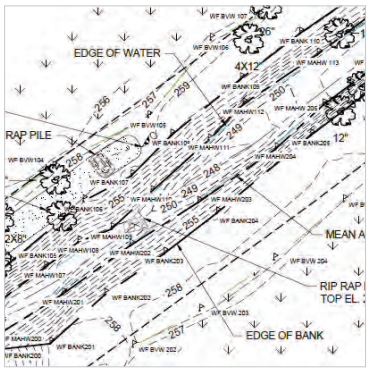
Sharon, Massachusetts

June 2024









Prepared for:
Massachusetts Division of Ecological Restoration, EPA Southeast New England Program, and Town of Sharon, MA

SHARON MASSACHUSETTS

Division of Ecological Restoration
Invested in Nature and Community

SNEP NETWORK

Horsley Witten Group

"Funded in part by the U.S. Environmental Protection Agency's Southeast New England Program"

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Introduction

The Horsley Witten Group (HW) is pleased to provide this report to the Massachusetts Department of Fish and Game, Division of Ecological Restoration (DER), the Town of Sharon (the Town) Conservation Commission, and the Southeast New England Program (SNEP) (collectively the Project Partners) documenting the permit-level (75%) design process for a proposed flow control structure in the drainage ditch within the Great Cedar Swamp (GCS) in Sharon, Massachusetts. The ditch, originally installed to mitigate flooding concerns in a residential development adjacent to the GCS, has significantly lowered groundwater levels causing significant negative impacts on the wetland ecology throughout a large extent of the GCS.

The potential flow control structure described in this report would increase the stage of water in the drainage ditch, thus elevating groundwater levels in the vicinity of the ditch to mitigate these negative ecological impacts to the GCS. The design of the proposed structure balances the objective of increasing water levels enough to create ecological benefits for the GCS, but not so much as to negatively impact nearby infrastructure (e.g., residential basements and septic systems) surrounding the GCS. This project continues a decades-long and ongoing effort to understand and mitigate the detrimental impacts of the drainage ditch on the health and ecosystem function of the GCS.

Project Purpose

The GCS is a 250± acre large wetland complex which includes an Atlantic White Cedar (*Chamaecyparis thyoides*) swamp – a globally rare community type. The site is located within the Town of Sharon in Norfolk County, Massachusetts. Most of the site is owned and managed by the Town, with some privately-owned parcels covering significant portions of the periphery of the swamp. The swamp is currently impaired by a drainage ditch that lies along its western edge. This ditch was installed shortly after the development of an adjacent subdivision to the north of the GCS ("Sharon Heights"), to drain water from the swamp to reduce flooding of the basements therein. Review of Town parcel data indicates that homes in Sharon Heights were constructed between the late 1940s and the late 1970s, with the majority built in the 1950s. Review of available historical aerial photography indicates the absence of the ditch in 1961 and fully completed ditch present in 1978.

In addition to lowering water levels in the Heights, the ditch has drained a large portion of the swamp, leading to die-off of the wetland vegetation, including the regionally-rare cedar trees which are the namesake of the GCS, replacement with more common wetland, upland facultative, and invasive species, subsidence of GCS ground surface levels, and the oxidation of organic soils ("peat"), releasing stored greenhouse gases in the process.

The drainage ditch is approximately 1.25 miles in length. At its downstream extent, where the ditch crosses under Wolomolopoag Street, the ditch is approximately 30 feet wide at the crest of its banks and 10 feet deep. The top of the ditch is lined with earthen berms composed of spoils from the original excavation of the ditch. In sections of its course the depth of the ditch extends below that of the overlying swamp deposit materials (poorly drained peat); deep enough to intersect the more readily drained sand and gravel aquifer stratum beneath the peat and thereby allow for more significant aquifer drainage for a large area of the swamp than would have been the case were the ditch confined to within the peat deposits.

The purpose of this project is to design a potential flow control structure that would be located near the downstream extent of the ditch within the GCS. The potential flow control structure would retain water behind it, elevating the stage of water throughout a portion of the ditch and surrounding aquifer, and reducing the amount of groundwater draining from the swamp and the aquifer into the ditch. The control elevation of the structure would be variable, allowing for seasonal adjustment during periods of high and low groundwater throughout the year, as well as for adaptive management to adjust water levels to mitigate any potential impacts to nearby infrastructure.

Setting

The GCS is bounded to the north and south by residential properties. To the north, the Sharon Heights neighborhood (Sunset Dr., Lee Rd., Essex Rd., Suffolk Place, and Clark Ct.) abuts the upstream extent of the drainage ditch, with developed parcels located as close as 50 feet from the ditch. To the west, the swamp is bounded by Ward's Berry Farm. Wolomolopoag St. forms the southern boundary of the swamp, with residences also present in the vicinity of the ditch culvert crossing under Wolomolopoag St., on Chippewa Ln., and across Wolomolopoag St. on Foxfire Dr., Barefoot Hill Rd., and Furnace St. The southeastern side of the GCS has nearby residences on Niantic and Manomet Rds., as well as property currently occupied by the Islamic Academy of New England School and Mosque. The northeastern portion of the GCS is crossed by the Northeast Corridor rail right-of-way which hydraulically isolates the northeastern lobe of the GCS from its larger body (Figure 1). Beyond the railroad, the GCS extends to East Foxboro Street. The section of the GCS east of the railroad has not experienced the detrimental impacts of drainage observed in the main body of the GCS west of the railroad (Fletcher, et al., 2012), likely due to the distance from the drainage ditch and the hydraulic separation provided by fill material underlying the rail bed.

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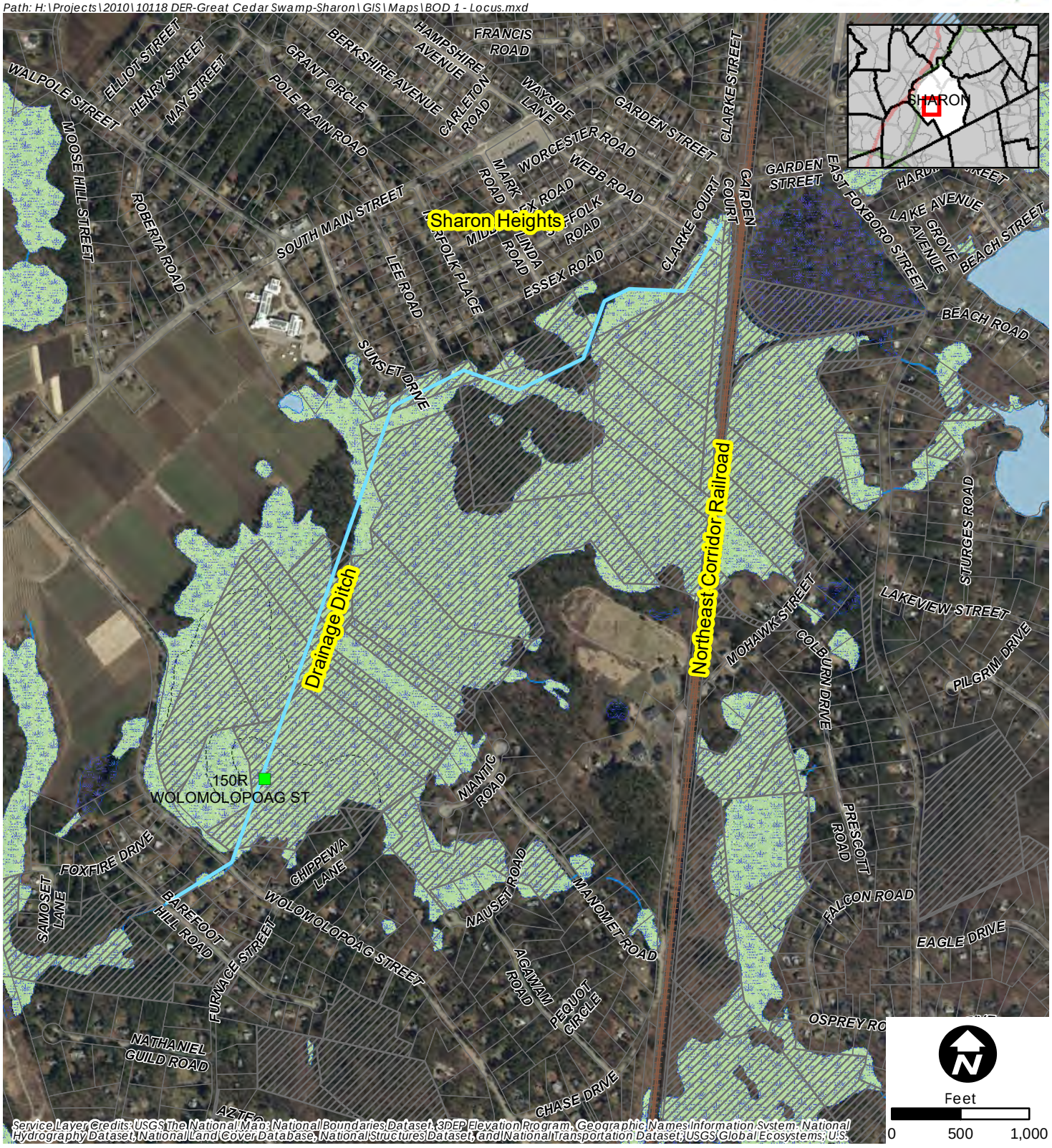

Date: 6/28/2024
Data Sources: Bureau of Geographic Information (MassGIS), ESRI

This map is for informational purposes and may not be suitable for legal, engineering, or surveying purposes.

- Railroads
- Drainage Ditch
- Proposed Structure Location
- Privately-owned Parcels
- Town-owned Parcels
- Shoreline
- Hydrologic Connection
- Mean Low Water Line
- Wetland Limit
- Closure Line
- Marsh/Bog
- Wooded marsh
- Open Water

Flow Control Structure Basis of Design
Great Cedar Swamp - Sharon, MA

Figure 1
Locus

Background

A large body of previous research for the GCS area in general, some dating back nearly 40 years, precedes and undergirds this report. Extensive study of the ditch itself and its impacts has been previously conducted under multiple project iterations extending back approximately 15 years. The sum of the information learned from those earlier studies informs the design process for the current project described in this report. Relevant portions of key studies related to the design task are summarized here:

Aquifer Protection Study (IEP Inc., 1987)

A study was conducted in 1987 to develop a comprehensive groundwater protection program for the Town, including the delineation of aquifer recharge areas, assessment of water quality threats, and potential mitigation measures to address those threats. Data collected during this study, specifically boring logs in the vicinity of the GCS, informed the development of most of the later, more refined GCS studies described in this report and included as Appendices.

Restoration and Revitalization of the Sharon Great Cedar Swamp Progress Report (Fletcher et al., 2012)

A concentrated period of investigation, spurred by the Sharon Conservation Commission, culminated in the development of the 2012 Progress Report authored by Peter Fletcher et al. on behalf of the Sharon Conservation Commission. The Progress Report summarizes the findings of early studies and also set the stage for all subsequent studies to date.

The 2012 Progress Report describes plant, animal, hydrologic, and soil studies conducted describing the condition of the GCS at that time. The Progress Report also introduces restoration concept designs including the flow control structure described herein. The report describes benefits of the swamp including natural stormwater retention and filtering (a benefit enhanced by the GCS location within drinking water well contributing areas and headwaters to the Taunton and Neponset Rivers), carbon sequestration in swamp peat soils, and habitat and aesthetic benefits of the Atlantic White Cedar biome.

Vegetation and wildlife studies summarized in the Progress Report document the detrimental impacts of lower water level hydrologic conditions on the native wetland plants, as well as their replacement with more common plants and invasive species. Soils investigations document significant soils subsidence in the range of one to three feet, exposing the root system at the base of standing trees. Subsidence in organic soils in proximity to the ditch is caused by the draining of groundwater and subsequent replacement of groundwater in the pore space with atmospheric gases. The presence of these gases ends the typical anaerobic accumulation of organic material in the peat and instead allows for aerobic decomposition of peat material. In

addition to reducing peat volumes, this process also releases greenhouse gases into the atmosphere.

The Progress Report describes contemporary groundwater and streamflow monitoring efforts. A network of 39 monitoring wells had been installed in and around the GCS, as well as stream staff gages near the Wolomolopoag Street culvert and another staff gage in nearby Lake Massapoag. HW contributed this effort by providing topographic survey of the monitoring wells, staff gages, representative cross sections of the ditch, a longitudinal profile of the ditch bottom, and stormwater infrastructure in the Sharon Heights neighborhood. Two monitoring well and staff gage water level measurement events took place in April and August of 2010; the results of which informed the creation by HW of two water table contour maps for the entire GCS area. The two maps represented high- and low-water periods to explore the seasonal variability in hydrologic conditions. These groundwater contour maps improved understanding of the direction of groundwater flow originating in the GCS, as well as the area of influence on the local groundwater flow caused by the ditch and by action of nearby public water supply wells. Analysis of these water table maps demonstrates that the ditch has the largest anthropogenic influence on the hydrology of the GCS.

The Progress Report also summarizes an analysis of groundwater quality conducted by Curry and McCoy (Bridgewater State University, 2011). This analysis found low levels of nutrients and no coliform bacteria in samples collected within the GCS, while higher levels of nutrients were detected in samples collected under the Heights neighborhood, supporting the conclusion that that the GCS dilutes nutrient inputs from developed areas located between the GCS and public drinking water wells located downgradient from the GCS.

Three proposed concept designs are described in the 2012 Progress Report. These three conceptual designs include a stone rip rap check dam, a flow control structure, and a buried culvert. Shortly after completion of the Progress Report, the Town proceeded with the installation of a stone rip rap check dam located approximately 450 feet upstream of Wolomolopoag Street. The check dam retained water at an elevation approximately one to two feet above the existing ditch bed. Subsequent monitoring of groundwater levels in the upstream vicinity of the check dam showed negligible increases, likely due to the relatively small increase in stream stage in the ditch caused by the check dam.

The remaining concepts include the flow control structure and the buried culvert options. Both options would rely on a similar principle: drainage into the ditch should be allowed in the vicinity of the Sharon Heights neighborhood, but drainage should be minimized where the ditch is further from the neighborhood. A buried culvert would accomplish this by conveying drained water from the vicinity of the Heights through a watertight culvert pipe, buried in the present location of the ditch. Drainage would continue in the Heights as it does now, but the watertight

nature of the culvert pipe would inhibit drainage throughout the remainder of the conveyance out of the GCS. As such, this system would restore near-natural hydrologic conditions in most of the GCS. The construction of a subterranean pipeline over a mile long and subsequent infill of the ditch would make this the most expensive of the three options. In addition, maintaining watertight conditions in buried pipes beneath the water table over long timeframes is difficult to impossible without significant monitoring and maintenance efforts.

Model Simulations Summary Technical Memorandum (HW, June 2024)

With the minimal observed improvement of groundwater levels resulting from the experimental check dam and the expected cost of the buried culvert option, Project Partner focus has coalesced around a plan to elevate water levels in the ditch through the installation of a variable height flow control structure near the downstream end of the ditch. In 2024, HW completed an H&H modeling assessment of the likely water level impacts from the proposed water control structure. That assessment indicated that a water control structure in the ditch could potentially be operated to retain approximately 3 feet of water in the ditch above the elevation of the current rip rap baffle; elevating surrounding groundwater levels in the adjacent areas of the GCS without negatively impacting properties within Sharon Heights. Modeling indicates that retaining water in the ditch would increase groundwater levels for a significant, though far from complete, portion of the GCS (i.e., areas proximal to the ditch). The potential structure would allow for adjustment to the water levels as needed to mitigate any unforeseen impacts and allow for management of seasonally fluctuating groundwater elevations.

The 2024 HW project included review of previously conducted studies, new field data collection, and development of a groundwater model to assess the potential groundwater level impacts to the GCS and surrounding neighborhoods from a hypothetical flow control structure in the ditch. Field data collected during this previous project included groundwater elevation measurements at monitoring well, streamflow measurements, installation of continuous, automated water level loggers in the ditch and surrounding monitoring wells, and survey location of the monitored points. Reports summarizing these projects are attached as Appendices C and D.

Current Project Scope of Work

The current project builds upon these older studies and, most directly, builds upon the January 2024 HW study to advance evaluation and design of a potential water level control structure near the lower end of the ditch. The Scope of Work for the current project includes project management and project team coordination, as well as two primary tasks completing new work. First, supplemental data collection and analyses were performed. This task included review of publicly available records (including septic system design and as-built plans) for nearby low-lying parcels at potential risk from water level increases resulting from the proposed water level

control structure and topographic survey of the proposed structure location was performed. The second task included the development of permit-level (~75%) design for a flow control structure. The permit level design scope of work includes this report, the attached operations and adaptive management plan (Appendix A), the attached engineer's opinion of probable cost, and the associated design plan sheets (Appendix E).

Flow-control Structure Impact Assessment

Groundwater modeling was performed to assess the extent and degree of groundwater elevation increase throughout the GCS anticipated under varying flow control structure configurations. Flow-control structures of varying control elevations and locations were modeled. The Model Simulations Technical Memorandum (HW, January 2024) is attached to this report as Appendix C. The Memorandum is summarized below as it pertains to the design process for the structure.

Model Development

A new, steady-state groundwater model was developed for the GCS and surrounding areas to assess the relative impacts of various flow control structure configurations in the GCS drainage ditch. The model was developed for MODFLOW-2005 using Groundwater Vistas version 8. MODFLOW-2005 is a three-dimensional finite-difference groundwater model. Properties and boundary conditions are assigned to cells within the finite-difference grid using the Groundwater Vistas interface. The MODFLOW-2005 model then performs calculations based on this grid, yielding a numerical solution (model result) representing groundwater conditions which satisfy the conditions set out in the model. The model was calibrated to steady-state (e.g. long-term average) baseline conditions based on water table maps produced in 2010 as well as baseflow conditions in the ditch measured in 2023.

The model domain extends laterally to natural hydrogeologic divides including Lake Massapoag to the northeast, Beaver Brook to the northwest Billings Brook to the southwest, and other Canoe River tributaries to the south. Stream elevations for these boundaries were determined from LiDAR elevations available from MassGIS. Ponds within the model domain were modeled as constant head boundaries. The bottom of the model includes an assumed layer of low conductivity fractured bedrock underlying the extent of the model domain. The elevation of this layer was determined from boring logs available from the IEP Aquifer Study (IEP, 1987).

Hydrologic properties within the model domain were assessed from a variety of sources. Surficial geologic maps (published by the United States Geologic Survey (USGS) and available on MassGIS) were utilized to understand the extent of swamp and other deposits. The depth of swamp deposits was interpreted from a cross section of swamp material depths included in the 2012 Progress Report.

Calibration

Two rounds of groundwater elevation measurement were conducted by HW throughout the GCS and surrounding area in April and August of 2010, as described in the 2012 Progress Report. These measurement rounds were placed into the broader temporal context of groundwater levels in the region by comparison to the USGS index well MA NNW27R. At the time of the April gauging event, water levels in the index well were 1.15 feet higher than their long-term average value. In August, water levels were 1.24 feet lower than average. As such, the mean of the two sampling values was adopted as a representative long-term average condition for the water table in the GCS.

The model produces an estimate of groundwater exiting the aquifer via the drainage ditch, which represents the average baseflow in the ditch. A series of four streamflow measurements were conducted near the proposed structure location during the 2023 field work effort and summarized in the January 2024 HW report (Appendix D). All streamflow measurements occurred during the spring, a period of relatively high regional groundwater levels. During this period, average groundwater levels at the nearest USGS groundwater Index Well (USGS MA-NNW27R) did not fall below 74th percentile values for the period of record, and the average groundwater level was equal to the 87th percentile value.

Streamflow values originally reported in the 2023 Field Work memorandum and subsequently incorporated in the January 2024 modeling report were miscalculated by inadvertently utilizing a wider flow segment width than actual. This calculation error caused streamflow values for that date to be calculated at a value doubled compared to actual streamflows. Modified streamflow values corrected for this error are reported in the table below. Table 1 below also includes two supplemental streamflow measurements collected in April and May of 2024 after the publication of the January 2024 HW report.

Date	SF1 Flow (cfs)	SF1 Flow (cfd)	Riffle Gage Reading	Preceding Rainfall (in)
1/31/23	**2.04**	**176,116**		**1/26/23 1.33**
2/14/23	**0.67***	**57,961**	**2.5**	**2/8/23 0.03 1/31/23 0.06**
3/16/23	**3.77**	**32,5356**	**1.19**	**3/14-15/23 2.73**
4/11/23	**0.47***	**40,198**	**2.51**	**4/6/23 0.02 4/2/23 0.33**
4/26/24	1.12*	97,157	2.44	4/21/24 0.48
5/21/24	1.13	97,200	2.44	5/19/24 0.10 in
Average	1.53	132331		

Bold values are corrected since initial reporting.
Precipitation values from weather station US1MANF0073 (Foxborough 2.3E, MA US)

Table 1: Streamflow measurements in the GCS ditch.

Streamflow measurements conducted on 2/14/23, 4/11/23, and 4/26/24 all occurred after a period of at least 5 days without rainfall, and as such are considered to represent baseflow conditions. The average flow value for these three measurement points, representing an average baseflow condition, is 0.75 cubic feet per second. This value is adequately aligned with the model calculated stream baseflow of 0.89 cubic feet per second.

Representation of Flow Control Structure Configurations

The ditch is represented in the model using the stream boundary condition package in MODFLOW. Several key parameters affect how streams interact with the water table in the MODFLOW modeling program. Physical properties of the stream including the length, width, and conductivity of the stream bed affect the conductivity of the stream channel. The stream bottom elevation represents the lowest stage that the stream can support. The stage parameter describes the elevation of the water surface in the stream channel. In cases where the stage is lower than the surrounding groundwater head, the stream is a *gaining* stream, meaning water exits the aquifer and enters the stream channel. The potential for water to enter the stream,

driven by this elevation difference, is tempered by the conductivity of the stream channel. Conversely, if the stream stage is higher than the groundwater head, then the stream would be a *losing* stream, adding water to the aquifer. Throughout the GCS, the drainage ditch is a gaining stream, as is typical in most New England settings.

The theoretical basis for using a flow control structure to elevate nearby groundwater levels relies on increasing the stage of the stream, thus reducing the potential for water to enter the ditch from the surrounding aquifer (i.e., reducing the degree to which the stream gains water). The height of the structure would dictate the elevation to which the stage in upstream model cells is increased. Higher elevation structures would increase water elevations for a longer extent of the stream. Due to the location of the proposed structure near the downstream extent of the ditch, this means that higher elevation structures would result in increased stream stages reaching upstream closer to the Heights neighborhood. A schematic representation of this theoretical basis is shown in figure 2 below. The upstream extent of increased stream stage and lateral extent of groundwater rise at varying control elevations is shown in Appendix C, Figures 6a-e. As shown in Figures 6b and 6c, a flow control structure with a control elevation two to three feet higher than the current rip rap baffle would cause water levels to increase upstream to the approximate location of Sunset Drive.

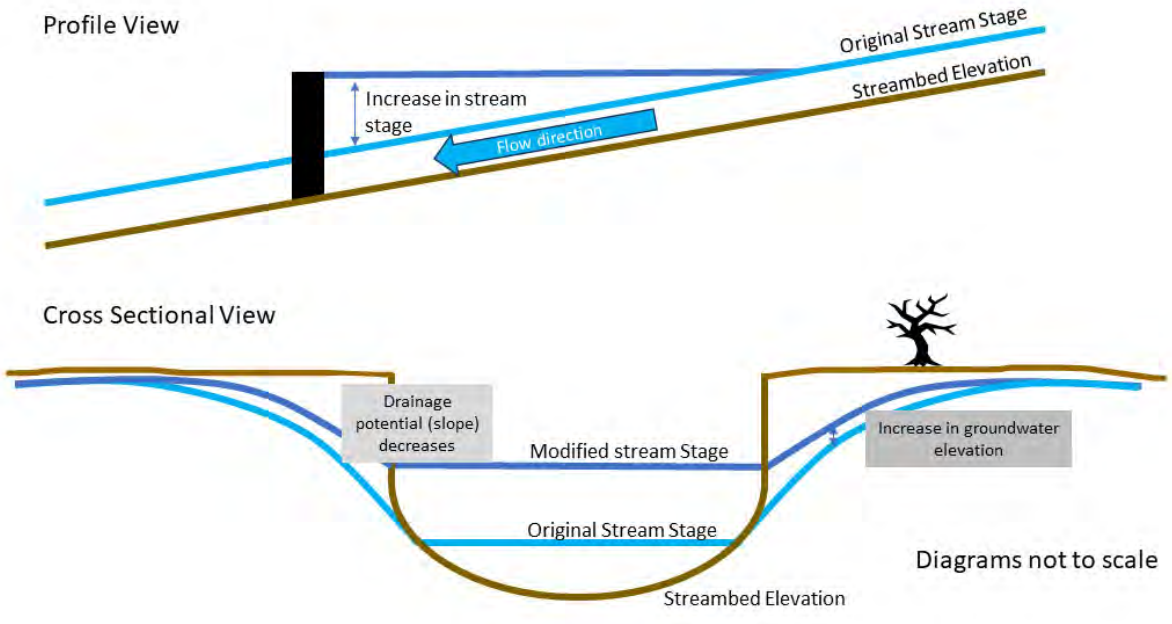

Figure 2: Schematic representation of structure impacts on stream stage and surrounding groundwater

Modifications Since Initial Publication

Modeling completed in 2024 focused on varying structure control elevations and the impacts of the structure on residences located in the Sharon Heights neighborhood. The configurations modeled as part of this effort all assumed that the structure would be located at the same position in the ditch as the existing rip rap baffle. The control elevation determined by the stream, focused on impacts to the Heights, was largely determined by the upstream extent of stage increase and resultant proximity of groundwater elevation increase to the Heights. The control elevation of 253.9 feet in the NAVD88 vertical datum (three feet higher than the current rip rap baffle) would increase water levels in the ditch for a considerable length upstream, beyond the point where the stream currently passes Lee Road. Due to the slope of the ditch, increases to stream stage in this upstream extent would be relatively small. The elevation of water levels at the culvert at the end of Lee Road, for example, was measured to be 254.3 feet during the April 26th, 2024 survey; approximately 0.4 feet higher than the proposed structure's control elevation and, as such, would likely be only minimally altered by the presence of a flow control structure downstream.

Supplemental field data collected since the January 2024 HW report was published indicates that a house at 154 Wolomolopoag Street might be vulnerable to increased groundwater elevations. The residents at 154 Wolomolopoag Street allowed HW survey personnel to locate the elevation of their basement and sump pump during spring 2024 topographic survey operations in support of the proposed flow control structure design. The observed presence and reported use at this property of a sump pump reflects the vulnerability of this location to increased groundwater elevations. The residents report that they do experience intermittent basement flooding, though it is unclear whether those impacts were due to groundwater levels or localized flooding resultant of poor stormwater drainage on Wolomolopoag Street. In either case, the project goal of minimizing impacts necessitated reassessment of any potential impact on this location to avoid potential impacts.

Relocation of the proposed flow control structure to a location approximately 425 feet further upstream from the existing baffle (i.e., further away from the 154 Wolomolopoag Street property) was assessed using the groundwater model. The more upstream location was chosen to be far enough upstream to avoid an abutting privately-owned parcel (identified as 150R Wolomolopoag Street in the Sharon assessors database, see figure 1), while still being within the area for which detailed topographic survey was collected and still being far enough downstream to accomplish the project's goals of elevating groundwater levels for a significant portion of the GCS. Relocation of the structure by approximately 425 feet upstream from the rip rap baffle location was modeled to create a significant reduction in groundwater rise at 154 Wolomolopoag Street relative to that modeled for the existing baffle location. Under the 3-foot structure scenario, the modeled increase in groundwater level was 0.5 feet for the original baffle

location. Relocating the structure 425 feet upstream reduces this increase to <0.2 feet, a value that is likely *de minimis* within the accuracy of the model and the best that can be accomplished while meeting project goals for the GCS. A limited area of reduced GCS water level improvement will result between the original and relocated structure positions from the move. This area is approximately 20 acres in area, though the greatest difference in improvement reduction is closest to the ditch. Figure 3 shows the modeled relative impact of the original and modified structure location.

Because the proposed flow control structure would be adjustable, the height of water retained by it could be reduced if necessary to avoid any observed and unwanted water level impacts at protect 154 Wolomolopoag Street.

Results

Modeling results indicate that a structure with a target control elevation of 253.9 feet would maximize the hydrologic improvements to the swamp while also being protective of surrounding residences. The control elevation for the structure should be variable to account for seasonal control strategies (e.g., retaining more water in the ditch during periods of low groundwater). Designing a structure allowing for variability in control elevations also allows for adjustment to account for inaccuracies in the modeling used to determine the target control elevation. Should unexpected impacts affect nearby residences, the structure control elevation could be lowered to mitigate these impacts. On the other hand, should the target control elevation be determined to be too low, the benefits could be maximized by further increasing the control elevation.

Modeling results define the appropriate range of structure control elevation variability. The minimum control elevation was determined to be 252.4 feet. At this level, modeling indicates that any measurable change in average groundwater elevation would occur within the confines of the swamp itself, with the possible exception of along Niantic Road where an increase of approximately 0.3 feet is simulated. The maximum control elevation was determined to be 254.9 feet, at which elevation the structure would increase water elevations throughout the section of ditch behind the Heights neighborhood but the potential impacts to surrounding low-lying properties would be greater.

Post-installation monitoring and adaptive control strategies for the adjustment of structure control elevations to maximize benefits to the GCS while minimizing impacts to surrounding low properties are described in the Operation and Adaptive Management Plan in Appendix A. Figures of modeling results are available in Appendix C.

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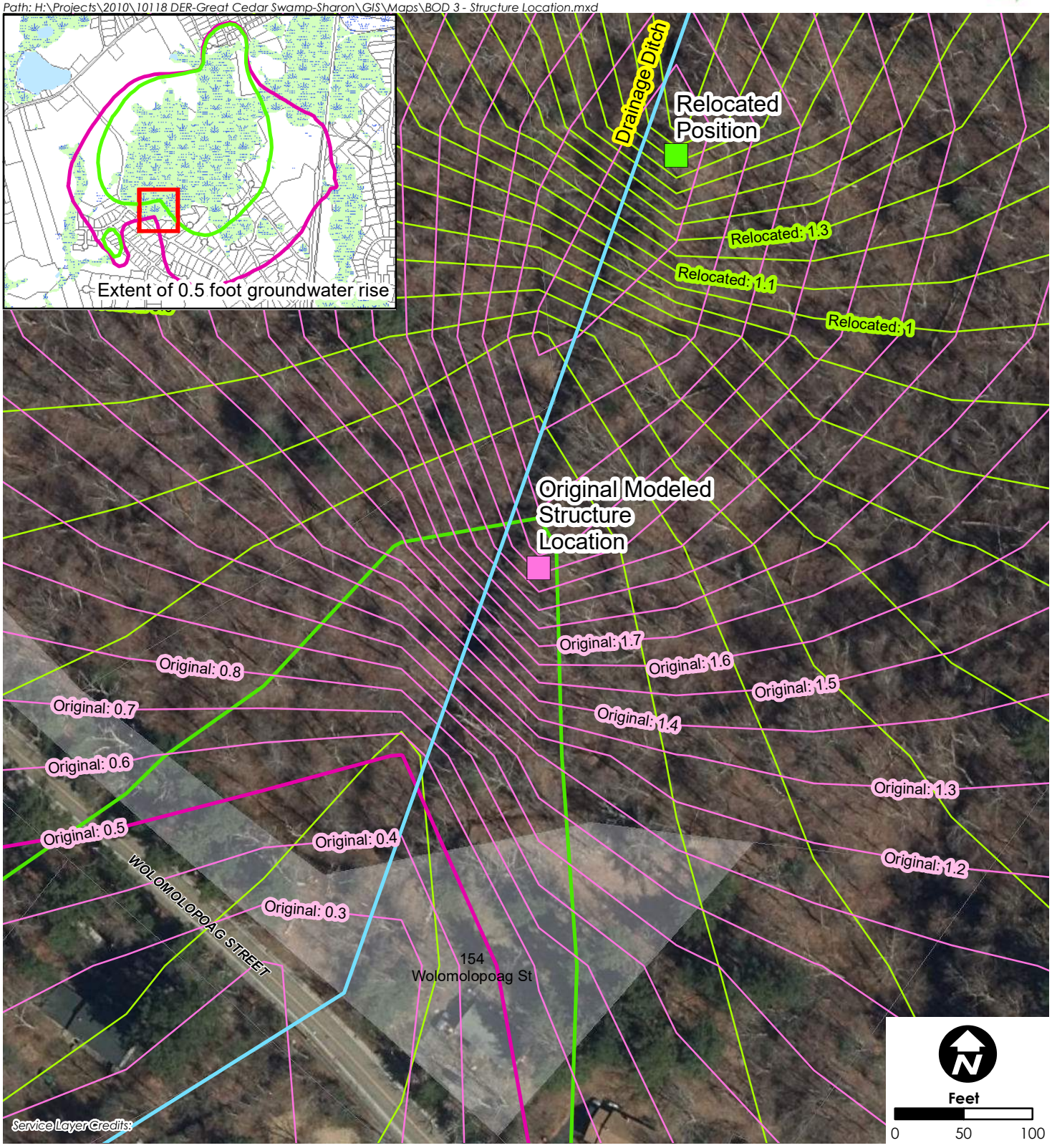
Path: H:\Projects\2010\10118 DER-Great Cedar Swamp-Sharon\GIS\Maps\BOD 3 - Structure Location.mxd



Date: 6/27/2024
Data Sources: Bureau of Geographic Information (MassGIS), ESRI

This map is for informational purposes and may not be suitable for legal, engineering, or surveying purposes.

- Drainage Ditch
- Parcels
- Original Modeled Structure Location
- Relocated Position
- Original location groundwater rise
- -0.5 feet
- Upstream location groundwater rise
- -0.2 feet

Flow Control Structure Basis of Design
Great Cedar Swamp - Sharon, MA

Figure 3
Relative Impact of Original and Relocated Structure Location

Supplementary Data Collection

Topographic Survey

HW conducted two rounds of supplemental topographic survey capturing existing conditions at the downstream extent of the drainage ditch, in the vicinity of the Wolomolopoag Street culvert. The first topographic survey occurred on April 26th, 2024 and the second occurred on May 21st. Concurrent with survey field operations, wetland resource areas in the proposed area of work were delineated. A summary of resource area findings is included below.

Survey was performed by total station. Control points were located within the Massachusetts State Plane and elevations measured relative to the NAVD88 vertical datum by RTK GPS. No property line survey was performed as part of this survey effort. Site access would necessitate crossing a portion of the parcel at 154 Wolomolopoag Street, as well as another privately owned parcel at 150R Wolomolopoag Street (Figure 1). Access through both of these parcels would be accomplished via a 75-foot drainage easement through each parcel, owned by the town. Property line survey to locate these property lines, as well as the specific extent of the easement, would be necessary prior to construction. Property lines presently shown on the existing conditions plan (Appendix E) are approximately located based on the Town of Sharon assessors GIS database. The 75-foot drainage easement is approximately located based on the centerline of the ditch as surveyed.

Septic System Plan Review

HW, with support from the Town of Sharon Conservation Administrator accessed septic plans for residential parcels surrounding the GCS. The goal of the exercise was to identify potentially low-lying parcels which might be vulnerable to increased groundwater levels. Potential vulnerabilities assessed included basement elevations (which may be subject to flooding) and septic system separation from groundwater. Massachusetts Title V septic system requirements include a separation from the bottom of the soil absorption system to high groundwater of 4 feet if the soil percolation rate is more than two minutes per inch or five feet if the percolation rate is less than two minutes per inch. For consistent assessment, all systems were assumed to require five feet of separation. Parcels on the following streets were included in the analysis:

- South:
    - Wolomolopoag Street
    - Barefoot Hill Road
    - Foxfire Drive
    - Furnace Street
    - Chippewa Lane
    - Seminole Circle

- East
    - Niantic Road
    - Manomet Road
    - Nauset Road
- North
    - Essex Road
    - Lee Road
    - Sunset Drive

In total, 141 parcels were investigated, and plans were found for 97 parcels (Figure 4). Key parameters recorded for each plan included foundation elevations (top and bottom), septic elevations (lowest), and groundwater observations (weeping or standing water in test pits, adjusted high groundwater values if available), percolation rates, locations of the building and septic system within the parcel, and datum elevation. Parcels with septic information available were plotted and compared against other data available for the area including previously developed groundwater elevation maps and LiDAR ground surface elevations.

Few septic plans included reference to a defined vertical datum (e.g., NAVD88). Most plans had reasonable elevations (e.g., just below ground surface elevations), but provided no specific datum information. Other parcels had arbitrary reference elevations only (e.g. 100 feet, despite actual elevations being greater than 200 feet) and some had no information available. The variability in vertical datums which might exist between various plans (produced over multiple decades), makes direct comparison of elevations values difficult.

Special attention was paid to areas where ground surface elevations were low relative to the elevation of the surface of the GCS. The GCS is relatively flat, with a surface elevation of 254 to 255 feet throughout. The May 2010 groundwater mapping effort served as a general basemap for regional groundwater elevation. This map indicates groundwater elevations of approximately 249 feet near Wolomolopoag Street, decreasing to the south with distance from the GCS. Groundwater elevations rise to the east, with an area of higher groundwater elevations (257 feet) to the north of Niantic Road. To the north, near Essex Roads and portions of Sunset Drive and Lee Road closest to the GCS, groundwater elevations are approximately 256 feet. Each of these areas is discussed in detail below.

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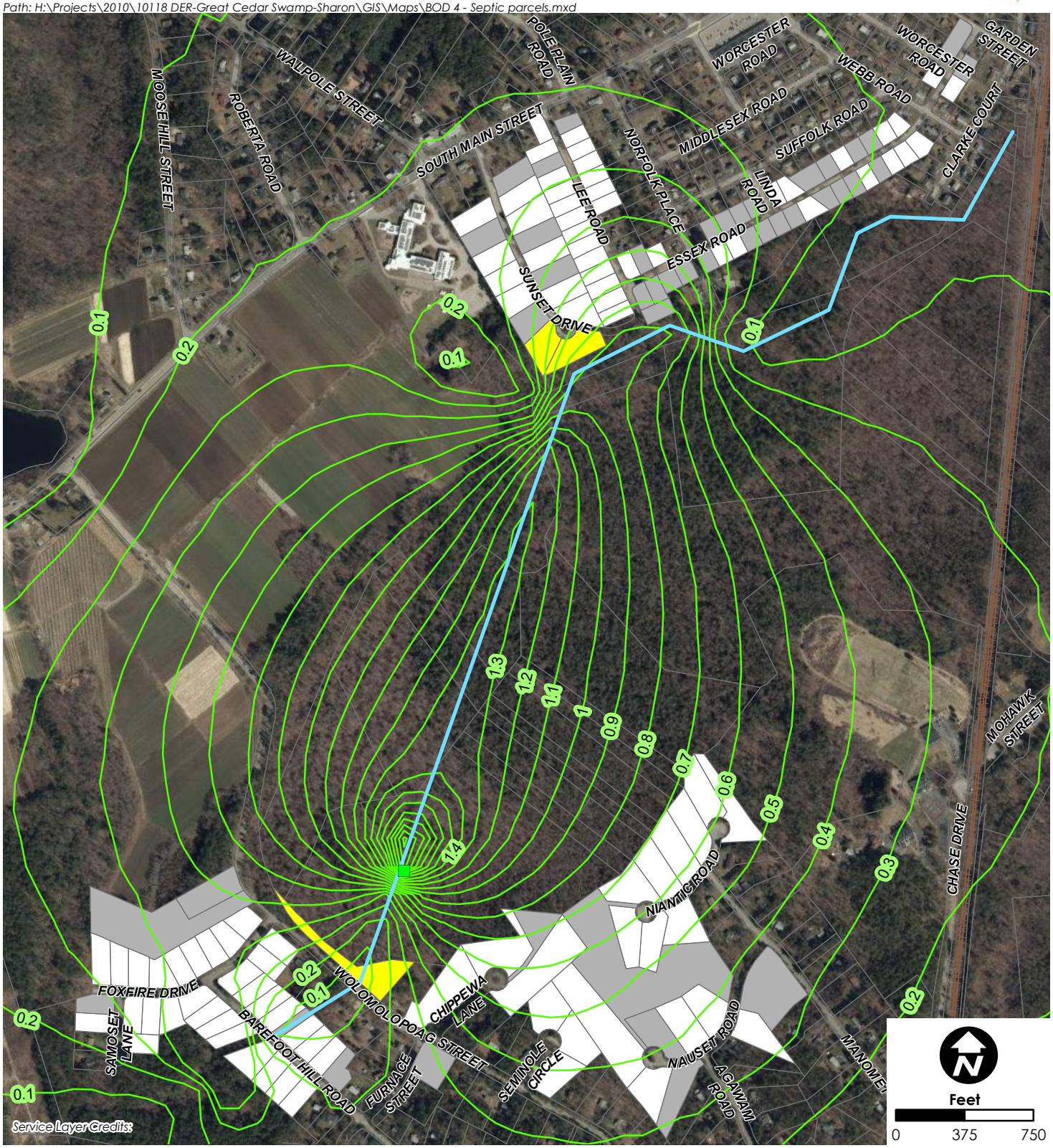

Date: 6/28/2024
Data Sources: Bureau of Geographic Information (MassGIS), ESRI

This map is for informational purposes and may not be suitable for legal, engineering, or surveying purposes.

- Proposed Structure Location
- Drainage Ditch
- 253.9 feet Control Elevation Groundwater Rise

Reviewed Parcels

- Plans Available
- No plans Available
- Priority Parcels for Monitoring

Flow Control Structure Basis of Design
Great Cedar Swamp - Sharon, MA

Figure 4
Septic Parcel Record Availability

Sharon Heights (Sunset Drive, Lee Road, and Essex Road)

LiDAR-derived ground surface elevations in the Sharon Heights neighborhood are generally higher than 261 feet. The lowest parcel is 31 Sunset Drive. The ground surface at 31 Sunset Drive slopes down towards the rear of the house, where the minimum ground surface elevation is 256 feet. The bottom of foundation elevation at this parcel is 255.5 feet and the septic system leach field pipe invert is 259.2 feet, according to available plans. No test pit or groundwater elevation was available with the septic system plan. The foundation and septic system plans do not provide datum information for the elevations provided. Comparison of the invert elevation of the drain invert at the end of Sunset Drive indicated on the plan (251.13 feet) with the drain outlet invert surveyed by HW personnel on February 14th, 2023 (252.91 feet), elevations indicated on the plan may be approximately 1.8 feet lower than elevations relative to the NAVD88 datum. The April 2010 groundwater contour map indicates a groundwater elevation at this location of approximately 255 feet. Modeling indicates a potential for groundwater to rise at this location by approximately 0.5 feet with the installation of the structure, making this parcel a priority for ongoing monitoring following installation.

Other parcels in the Heights neighborhood generally have higher ground surface elevations and lower groundwater elevations. Bottom of foundation elevations (where available) are all higher than 31 Sunset Drive. All septic elevations in the area are also higher than 31 Sunset Drive, or are sufficiently further away from the GCS such that groundwater elevations are lower and anticipated change in groundwater elevation is reduced, with the exception of 35 Sunset Drive, with a reported septic system leach field pipe invert of 256.8.

Two drainage outfalls are present at the ends of Lee Road and Sunset Drive. The outfall at Lee Road has an invert of 253.4 feet. This invert is lower than the target control elevation of the proposed structure, however standing water in this culvert was surveyed at an elevation of 254.3 feet. As such, retention of water in the ditch to a target control elevation of 253.9 feet is not anticipated to worsen existing conditions at this outfall. A similar situation exists at the Sunset Road drain outfall, where the drain outlet invert is 252.9 feet. Standing water was again present in the channel receiving drainage from this outfall, and so no worsening of existing conditions is anticipated from the proposed design.

Niantic Road

Parcels at the end of Niantic Road directly abut the GCS, though the houses and septic systems on these parcels are typically located near the road where ground elevations are approximately eight feet higher than the swamp surface elevation. Groundwater elevations as mapped in April 2010 range from 254 feet at the southern end of Niantic Road to 258 at the northern end. The anticipated rise in groundwater due to the structure at this location is 0.6 feet. Bottom of

foundation elevations indicated on available plans range from 261 to 275 feet. The lowest septic component elevation is at 17 Niantic Road, with a leach pipe invert of 263.3 feet. Other septic systems in the area are above 264.5 feet. As such, modeling suggests that houses in this area would not be negatively impacts by the degree of groundwater rise anticipated by the proposed structure.

Wolomolopoag Street

As discussed above, the basement elevation of 154 Wolomolopoag Street was identified as a potential area of concern. Therefore, the design was modified by relocating the proposed flow control structure further upstream to reduce the impact of the structure near the downstream extent. Relocation of the structure reduces the degree of groundwater rise along Wolomolopoag Street in the vicinity of the ditch crossing as well as in the neighborhood of Foxfire Drive, Barefoot Hill Road, and Furnace Street. Groundwater rise exceeding 0.2 feet is anticipated along Foxfire Drive only within this neighborhood. The water table in this area slopes downward away from the GCS, falling by approximately one foot per hundred feet with a gradient perpendicular to Wolomolopoag Street. Bottom of foundation elevations for all homes with septic plans available in this neighborhood indicate at least ten feet of separation from groundwater, based on the April 2010 water table map. The lowest septic component elevation for systems in this neighborhood indicate at least seven feet of separation from groundwater. As such, the minimal increase anticipated in this region is not expected to impact any residential infrastructure in this area. Similar results were found on Chippewa Lane. Land surface elevations in this area rise, and septic plans reviewed indicate lowest feature elevations at least seven feet above the mapped water table (with the exception of 14 Barefoot Hill Road, with a separation of approximately 5 feet, however no groundwater rise is anticipated at this location).

Parcel	Minimum Elevation (feet)	Minimum Elevation Description	Groundwater Elevation from April, 2010 map (feet)	Anticipated Groundwater Rise (253.9-foot control elevation) (feet)	Elevation Source
154 Wolomolopoag Street	254.59 253.87	Basement Floor Sump	247	0.2	Surveyed (NAVD88)
31 Sunset Drive	255.5 feet	Bottom of Foundation	257	0.5	Septic plan, no datum information
35 Sunset Drive	256.8 feet	Septic System Leach Pipe Invert	257	0.5	Septic plan, no datum information
Sunset Drive drainage outfall	252.9	Invert	Unknown*	0*	Surveyed (NAVD88)
Lee Road drainage outfall	253.4 254.3	Invert Standing water	254.3*	0*	Surveyed (NAVD88)

*Water levels at the Heights drainage outfalls were not surveyed during the 2010 water level mapping efforts. Nearby groundwater levels were interpreted to be 256 feet based on nearby monitoring points, however any influence of the drainage system is not reflected in that effort. Standing water was observed at the Lee Road outlet in 2024 at an elevation higher than the structure control elevation, and as such the outlet control elevation at this location is not anticipated to increase under proposed conditions.

Table 2: Priority areas for impact monitoring

Permit Level Design

HW created a permit-level design for a flow control structure in the ditch. The proposed structure features a concrete wall installed into the sides of the bank, perpendicular to the flow of water. Centered in the concrete wall is a spillway with a removable aluminum stop log system. The stop logs stack into place within the system, aligning to seal off water from flowing between them. The stop logs are configured to change the elevation the stream must reach before flowing past the control structure (the control elevation).

Hydraulic Modeling

HW developed a hydrologic and hydraulic (H&H) model for the proposed flow control structure using HydroCAD software. HydroCAD utilizes TR-20 and TR-55 methodologies, developed by the Department of Agriculture, Soil Conservation Service, to model stormwater runoff from watershed areas during heavy rainfall. Values for precipitation depths were obtained using the NOAA 14+ method, developed by the National Oceanic and Atmospheric Administration Hydrometeorological Design Studies Center. Precipitation depths are shown below in Table 3.

Recurrence Interval	Depth (in)
2-year	3.84
10-year	5.95
25-year	7.59
100-year	10.26

Table 3: NOAA 14+ Precipitation Values for Sharon (Calculated in 2024)

The drainage area contributing runoff to the proposed location of the flow control structure was estimated using StreamStats, a software tool developed by USGS and available online. StreamStats delineates drainage basins using publicly available topographic data (Figure 5Figure) and provides estimates of landcover in those drainage basins using publicly available satellite data. Landcover and total drainage basin contributing areas are listed below in Table 4. Time of concentration, representing the maximum time runoff in the watershed takes to reach the structure, was calculated by the model to be 40.5 minutes. Longer time of concentration yields a lower peak runoff rate, as travel time of rain falling throughout the watershed reaches the point of interest over a longer period. Model results were not sensitive to time of concentration values. Lowering model time of concentration values to the minimum value of six minutes did not significantly alter the result of modeling.

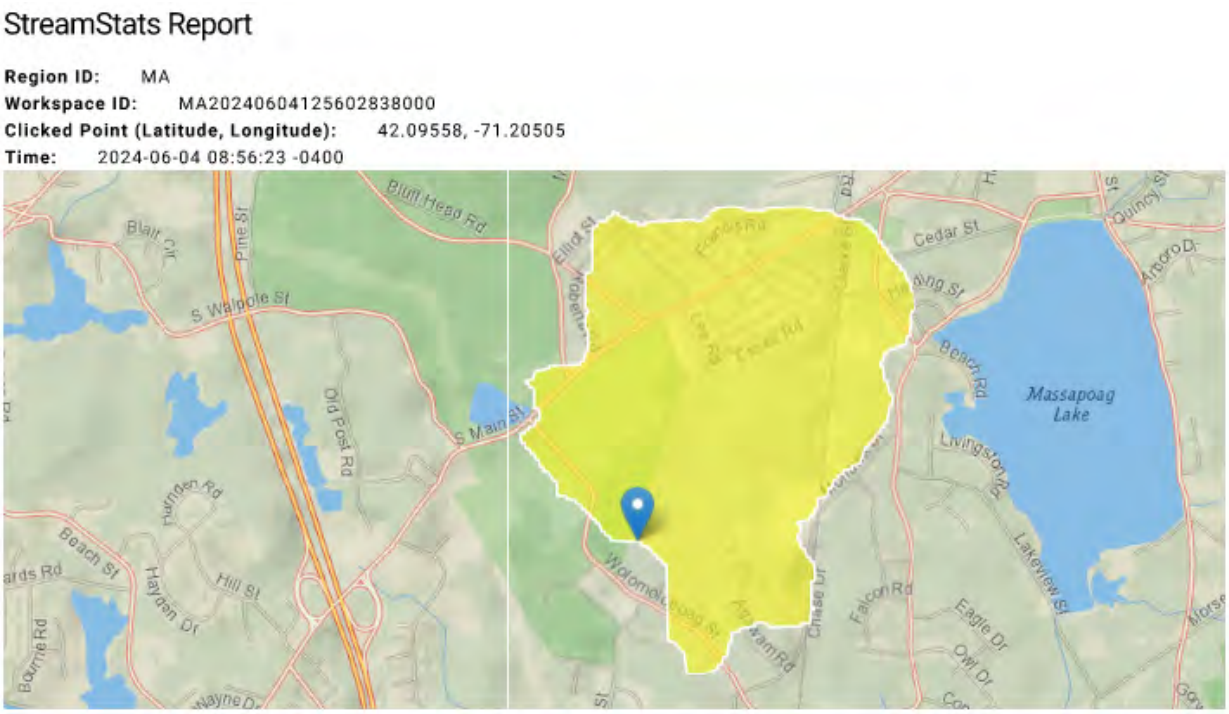

Figure 5. StreamStats Drainage Area Delineation

Landcover (StreamStats)	HydroCAD Description	Area (ac)
Forest	Woods, HSG D	0.469
Lakes and Wetlands	Water, HSG D	0.397
Impervious	Paved Parking, HSG D	0.160
*	Grass, HSG D	0.184
Total	**-**	**1.21**

*remainder when forest, lakes/wetlands, and impervious were subtracted from total drainage area

Table 4: Drainage Area Landcover

The drainage ditch was modeled in HydroCAD as a pond, with a storage volume based on survey data set to equal a 3,800-foot-long, 15-foot-wide, 10.4-foot-high, prismatoid with side slopes of 1.2 horizontal feet to 1 vertical foot. Per the groundwater model described above, a base flow value of 1.86 cubic feet per second (cfs) was set for the ditch.

Structure Design

Using the HydroCAD model, HW evaluated the dimensions of the proposed outlet structure to determine the minimum length of a spillway needed to achieve the target range of water elevations in the ditch. Per the groundwater model, a target water surface elevation range between 253.9 and 254.9 feet NAVD88 was determined in order to maximally elevate groundwater levels in the GCS while remaining protective of nearby infrastructure.

The HydroCAD model indicated that a 1-foot-wide main spillway set at elevation 253.9 would be sufficiently wide to prevent water surface levels from reaching 254.9 feet during the 100-year storm, a conservative storm event used to ensure that ponding would not impact abutting properties. The structure top was represented by a 15-foot-wide secondary spillway in the HydroCAD model at the maximum control elevation of 254.9 feet. The secondary spillway was not activated during model simulations (i.e., water never overtopped the structure). Results of the hydraulic modeling analysis are shown in Figure 6 below.

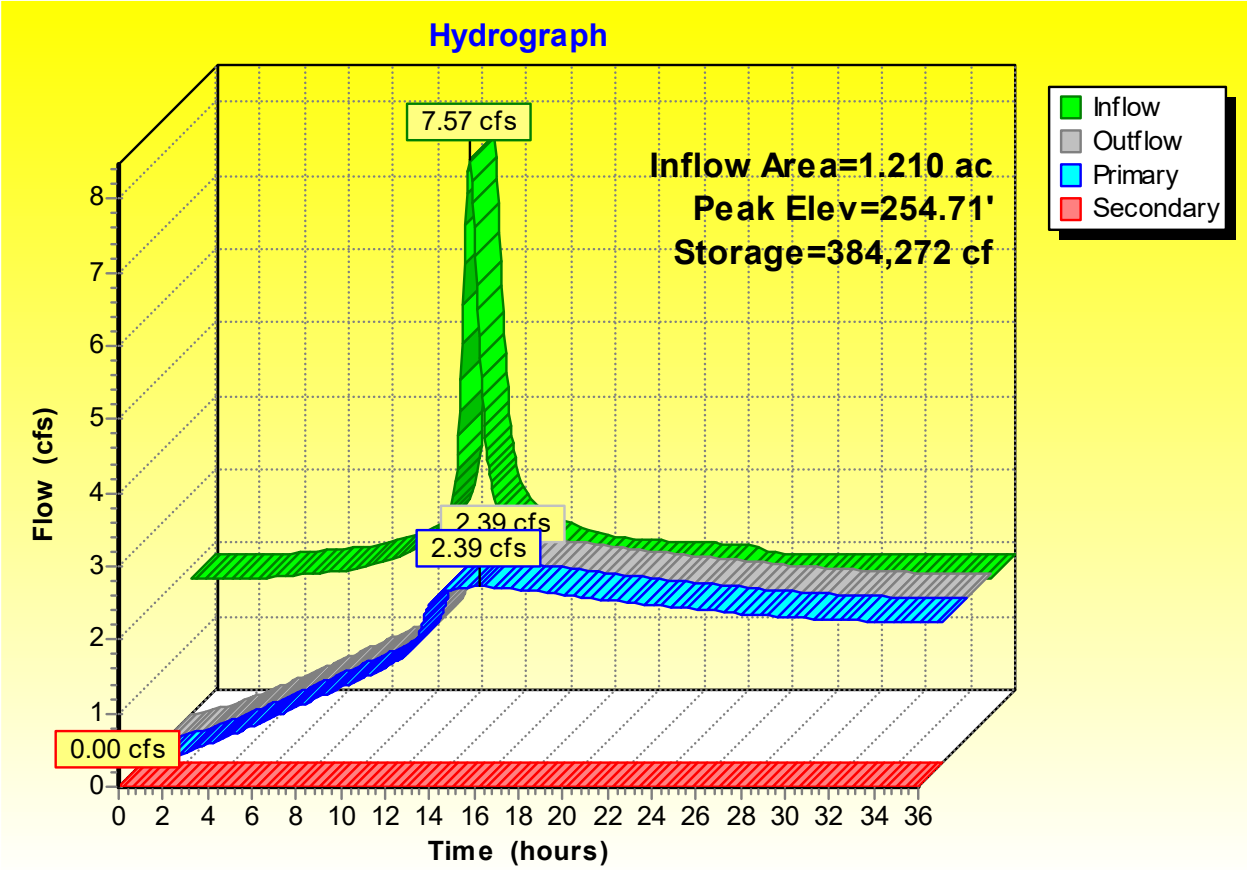

Figure 6. HydroCAD Output: 100-year Storm Event

Based on these results, a 1-foot-wide main spillway at elevation 253.9 is predicted to be sufficient to achieve target water elevations. As an added factor of safety, HW advanced a design with a 5-foot-wide main spillway, with removable checks at elevations lower than 253.9 to provide resiliency against heavier storms that may occur more frequently in the future due to climate change. The proposed design is included in the permit-level plans (Appendix E).

Engineers Opinion of Probable Cost

An engineers estimate of probable cost was developed for the proposed design. Costs were taken from the most recent year's weighted bid prices median value for the Massachusetts Department of Transportation District 5, manufacturer quotes, and professional judgement of dewatering costs. The probable cost of the proposed construction ranges from $269,200 to $380,000, including contingencies. A breakdown of costs is provided in Appendix E.

Resource Area Delineation

HW assessed and delineated wetland resource areas during two site visits to the GCS in April and June of 2024. The wetland resource area boundaries within the vicinity of the stream corridor were delineated starting at the culvert at Wolomolopoag Street to approximately 800 feet upstream of the culvert (the Project Site). HW delineated the boundaries resource areas on site with flags, which were located either by or with RTK GPS for inclusion in the existing conditions plan (Appendix E). Details of the resource area determination and findings are provided in Appendix B.

Summary and Next Steps

This report describes the process and outcomes for a permit-level design of a flow control structure to mitigate hydrologic impairment in the GCS currently caused by the presence of a drainage ditch within the GCS. HW undertook field investigation, groundwater modeling, review of available records, hydraulic modeling, and design efforts to design the flow control structure proposed in this report to maximize the beneficial effects of the structure while avoiding detrimental impacts to nearby infrastructure. The design described herein builds upon more than a decade of investigations into the problems caused by the ditch and previous efforts to mitigate those problems.

The outcome of this process yields design plans (Appendix E) describing a variable flow control structure capable of retaining water to an elevation range between 252.4 to 254.9 feet NAVD88, or one and one half to four feet above the current control elevation of the riprap baffle structure currently in place at the. Retaining water to these elevations would increase stage in the drainage ditch, reducing the potential for groundwater to enter the ditch from the peat soils and underlying sand and gravel material in the GCS. With decreased potential for water to drain into

the ditch, groundwater levels in the vicinity of the ditch are anticipated to increase. The degree of increase becomes smaller at further distances from the ditch. As such, the total amount and extent of increase becomes a function of the control elevation of the ditch.

Modeling efforts completed by HW in January 2024 and subsequent refinement of the model determined the target control elevation (253.9) and the range of variability appropriate for adaptive management of water levels within the GCS. Review of available information regarding the elevations and locations of nearby sensitive infrastructure (residential basements and septic systems) which might be impacted by increased groundwater elevations. The target elevation and variability of the structure was designed to maximize the amount of groundwater increase while avoiding any negative consequences to nearby property. The variable nature of the proposed structure allows for fine-tuning of the control elevation to further ensure a protective equilibrium may be established. The attached Operation and Adaptive Management Plan (Appendix A) outlines the proposed procedure for this tuning process and recommendations for related monitoring to analyze the performance of the proposed design.

The design completed during this project is at the permit-level (75% design). Further evaluation and design would be needed prior to construction. Such efforts are anticipated to include at least the following:

- Property line survey to determine the precise location of the property lines for the parcel at 150R Wolomolopoag Street, as well as the drainage easement crossing 150R Wolomolopoag Street and
- Geotechnical borings at the proposed area of work to confirm that the proposed structure location is suitable for construction of a concrete headwall and associated flow control structure components.
- Further discussion with manufacturers and Town staff to refine structure design to ease adaption and O&M requirements.
- Implementation of a monitoring plan as described in the Operation and Adaptive Management Plan (Appendix A) to ensure that changes in groundwater level near sensitive parcels (specifically 157 Wolomolopoag Street, 31 Sunset Drive, and 35 Sunset Drive) are not negatively impacted by water elevations exceeding the anticipated impacts.
- Educational materials and public outreach opportunities should take place to inform members of the community about the ongoing work and proposed project. HW has been contracted by the Southern New England Program (SNEP) to support these efforts, in conjunction with the Town of Sharon, throughout summer 2024.

Appendix A: Operation and Adaptive Management Plan

Operation and Adaptive Management Plan

Introduction

This plan describes the recommendations for routine maintenance as well as monitoring and adaptive management procedures pertaining to the variable flow control structure proposed for the drainage ditch in the Great Cedar Swamp (GCS) in Sharon, Massachusetts. The structure is designed with a five-foot wide primary spillway, with the control elevation of the spillway set by stop logs inserted into the embedded grooves installed on the vertical sidewalls of the spillway. The vertical range of stop log configurations spans from a minimum elevation of 252.4 feet NAVD88 (with all stop logs removed) to 254.9 feet (all stop logs installed). Typical stop logs are six inches in height, and an inventory of five logs would be needed to facilitate the range of configurations.

Operation of the Structure

The structure retains water through the incorporation of stop logs which slide vertically into embedded grooves in the primary structure body. Stop logs may be added or removed to increase or decrease the level of water retained by the structure.

The maximum control elevation of the structure was determined to be 254.9 feet, with a typical control elevation of 253.9 feet based on modeling analyses of anticipated impacts. A conservative approach to achieving this target control elevation is by slowly increasing the water level retained by the structure from the minimum elevation (252.4 feet, no Stop Logs) by low increments. The evaluation of positive and potentially negative impacts from operation of the structure would be informed by continuous monitoring of water levels at key locations throughout the project area in proximity to identified low-lying properties should be employed in the manner described below to assess the impact of each new control elevation.

Monitoring Plan

Monitoring groundwater conditions near the structure should be performed to assess the effectiveness of the structure at increasing water levels within the GCS and to provide an early warning of potential unanticipated impacts to nearby homes. Existing monitoring wells are present near Sharon Heights which should be incorporated into the monitoring plan. Additional wells should be installed in key locations to supplement the coverage of monitoring. Stream stage monitoring points utilized in previous studies at locations in upstream and downstream of the structure, as well as near the Heights neighborhood should be monitored, to assess impacts of the structure on water levels in the ditch. The location of the existing and proposed wells and stream stage monitoring points to be included are shown on Figure A1. New monitoring wells

and stream stage monitoring points should be surveyed for position and elevation using a common vertical and horizontal datum.

Monitoring should begin approximately a year prior to the construction of the structure, if possible, in order to assess baseline conditions and prevailing seasonal variations. Monitoring should include a combination of water level loggers, recording water pressures at regular intervals, and routine manual depth to water measurements. Once a sufficient baseline has been established, water level data from each monitoring point should be correlated against USGS Index Well MA NNW27R to allow for the evaluation of potential future water level trends that may be related to the flow control structure or other site-specific factors, as opposed to regional climatic variation as depicted by the USGS Index Well.

Following the establishment of a baseline condition and construction of the flow control structure, initial configuration of the structure should include no stop log (control elevation of 252.4 feet). Modeling indicates that this lowest elevation configuration of the structure poses little to no risk to any surrounding infrastructure. As such, monitoring for a full year is not necessary and the first stop log may be installed after approximately six months. Following the installation of the first stop log (control elevation 252.9 feet), monitoring should continue for a year to assess the impacts of this configuration prior to the installation of additional stop logs.

Installation of each subsequent stop log should be followed with another year of monitoring to allow for hydrologic response to equilibrate and to assess impacts under a range of conditions. The following timeline of control elevations is proposed:

- Year 0: Baseline monitoring of existing conditions
- First 6 months: Installation of the structure and monitoring of impacts at a control elevation of 252.4 feet.
- Months 6 - 18: Installation of first stop log and monitoring of impacts at a control elevation of 252.9 feet.
- Months 18 - 30: Installation of second stop log and monitoring of impacts at a control elevation of 253.4 feet.
- Months 30 - 42: Installation of third stop log and monitoring of impacts at a control elevation of 253.9 feet. This elevation represents the target control elevation.
- Two additional stop logs may be added to the structure beyond the target elevation to a maximum control elevation of 254.9 feet. Installation of stop logs above 253.9 feet may be warranted following equilibration at the target control elevation, based on the assessment of monitoring data collected in prior monitoring phases.

Data collected during each monitoring phase must be interpreted to determine each subsequent action. The following items should be considered during interpretation of each monitoring phase:

- What is the annual range of groundwater elevations throughout the monitoring phase at each monitoring point?
- How do observed water level fluctuations at each monitoring point vary relative to the control data from USGS Index Well X in consideration for the correlation relationship established during baseline monitoring?
- What is the lateral extent of groundwater change during the monitoring phase?
    - Comparisons should be made for similar times of year (and thus similar hydrologic condition with respect to seasonal weather patterns) to ensure that findings relate apples-to-apples effects of the structure.
    - Special attention should be paid to monitoring wells near priority infrastructure, including the monitoring wells nearest 154 Wolomolopoag Street and Sunset Drive.

Maintenance of the Structure

The General Contractor shall provide, inspect, and maintain all temporary control measures during the construction phase of the project. The Town of Sharon, as owner of the site, would be responsible for on-going operation and maintenance activities. The flow control structure should require minimal maintenance, however periodic inspection and routine maintenance ensures prolonged performance in line with the project objectives. The structure should be inspected when stop logs are added to the structure during the initial monitoring phase, and annually afterwards for the lifetime of the structure, as well as following large storm events. Inspection should include the following items:

- The spillway should be maintained free of debris (litter, tree limbs, other vegetative debris, sediment, etc.) which might impede the flow of water, alter the control elevation of the spillway, or inhibit the installation or removal of stop logs. Debris should be cleared from the spillway and embedded grooves.
- Inspect for erosion and scour near the structure and spillway. Erosion, undercutting, or scour can undermine the structure. Erosion should be reported, and if significant erosion is noticed mitigation measures should be evaluated.
- Inspect the concrete portion of the structure for settlement, cracking, or spalling.
- High water marks on and around the structure indicating flood levels should be recorded.

Detailed instructions for the installation, maintenance, and troubleshooting of stop logs provided by the manufacturer are included in the following pages.

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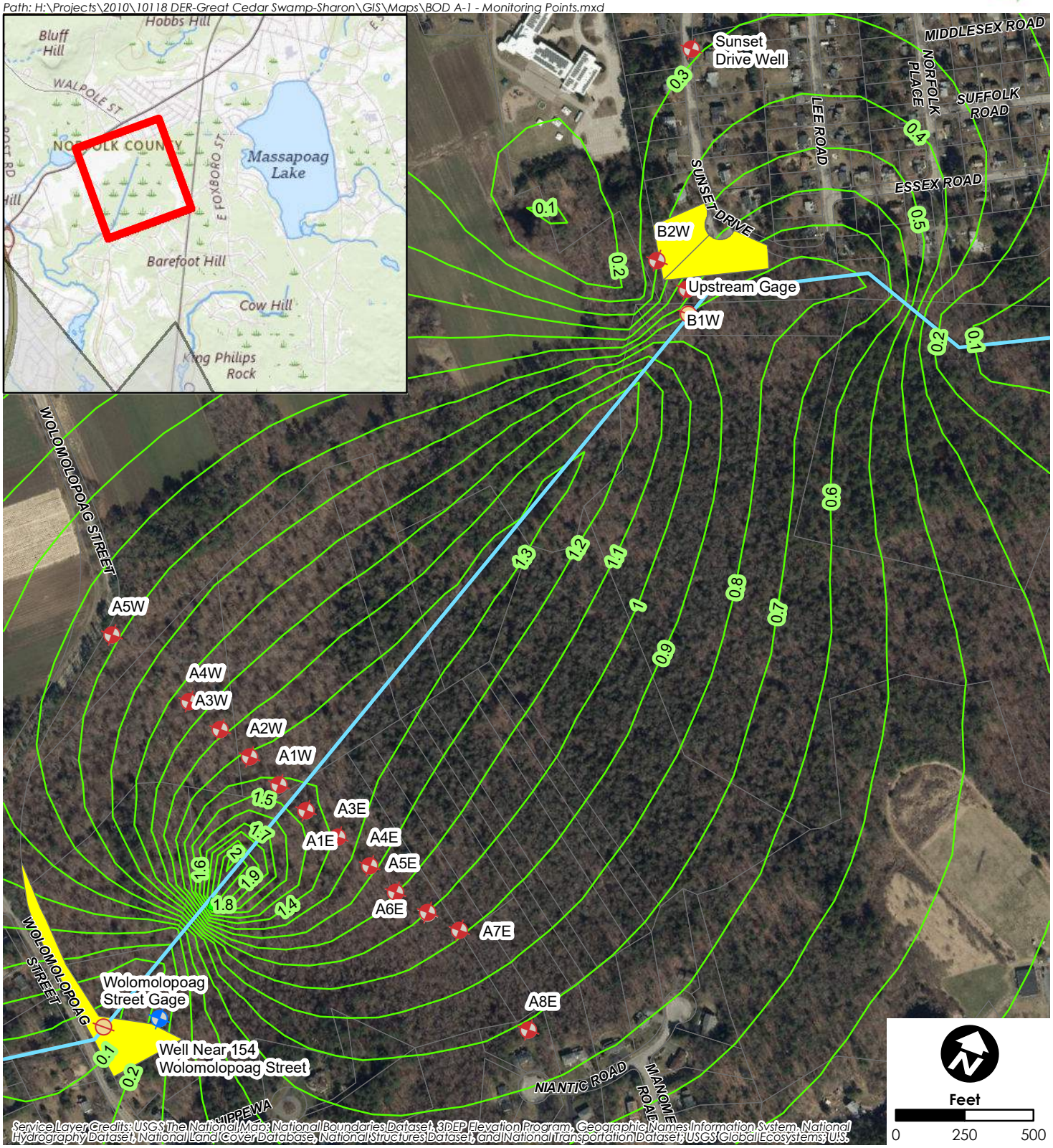
Path: H:\Projects\2010\10118 DER-Great Cedar Swamp-Sharon\GIS\Maps\BOD A-1 - Monitoring Points.mxd



Service Layer Credits: USGS The National Map: National Boundaries Dataset, 3DEP Elevation Program, Geographic Names Information System, National Hydrography Dataset, National Land Cover Database, National Structures Dataset, and National Transportation Dataset; USGS Global Ecosystems; US...

Date: 6/28/2024
Data Sources: Bureau of Geographic Information (MassGIS), ESRI

This map is for informational purposes and may not be suitable for legal, engineering, or surveying purposes.

- Proposed Structure Location
- Drainage Ditch
- Priority Parcels for Monitoring
- Existing, Staff Gage
- Existing, Well
- New, Well
- 253.9 feet Control Elevation Groundwater Rise

Flow Control Structure Basis of Design
Great Cedar Swamp - Sharon, MA

Appendix A: Figure 1
Monitoring Points

Stop Log Assemblies

Installation Instructions

Introduction

This manual describes the recommended methods of installation, initial operation and maintenance for Whipps, Inc. stop log assemblies and related components. This manual should be used in conjunction with the approved installation drawings provided by Whipps, Inc.
Whipps, Inc. stop logs are custom built to meet the requirements of each specific application. The stop logs provided have low leakage characteristics. However, care must be taken in the handling, storage and installation of the equipment to ensure that it will function as intended and restrict leakage within the specified parameters.
The information in this manual is intended only as a recommendation for the proper and satisfactory installation of our equipment. Whipps, Inc. assumes no liability, expressed or implied, for the interpretation of the recommendations or faulty installation of the stop logs. Whipps, Inc.'s responsibility is limited to defects in manufacturing.

Handling and Storage

To prevent personal injury or equipment damage, follow standard safety procedures when handling equipment and be sure rigging equipment is properly set and in safe working condition.

When unloading the equipment from the box trailer or flat bed truck, use care during removal and storage. If the equipment has been shipped mounted to a wooden skid, lift the skidded material from the bottom.

If damage has occurred in transit, file the necessary report with the freight carrier and contact Whipps, Inc. immediately.

Thoroughly review the packing list and compare the items on the list to the equipment received.

Although Whipps Inc. stop logs are durable and heavily constructed, care is necessary during storage, handling and installation. The seals along the sides and bottom of each log should be protected from damage.

Equipment should be stored on planks or timbers on a flat surface to keep them off the ground and to prevent distortion. Equipment should be covered with tarps to protect the equipment from foreign matter while stored. Where there are a number of stop logs or grooves (aka guide frames) and where storage space is limited, it maybe necessary to stack them with heavy timber blocking placed between the equipment to prevent damage. When stacking equipment, take care to avoid damaging the seals on the stop logs or distorting the grooves.

Installation

Installation - General

The most important aspects of a stop log installation are listed in this section. If these recommendations are followed, a proper installation is assured.

Carefully review the installation drawing for each assembly prior to installation to confirm proper setting. If the installation drawings are not available, please contact Whipps Inc. at 978-249-7924 or www.whipps.com.

Installation - Embedded Grooves (aka Guide Frames)

1. On grooves with embedded guide frames and/or an embedded invert member, box-outs or recesses are required in the channel walls and/or the channel floor during the concrete pour. The box-outs shall be of sufficient size to accommodate the groove assembly. See installation drawings for dimensional information.

2. The groove assembly must be well supported prior to the addition of grout to prevent distortion. Distortion of the groove may make installation of the stop logs difficult and may cause excessive leakage.

3. The invert member shall be mounted flat on the horizontal plane and the guide frames shall be perpendicular to the invert member. This interface between the bottom of the guide frames and the invert member is important to ensure proper sealing.

4. Care should be taken to keep the inside the groove and the seals on the stop logs free from grout or debris.

Installation – Grooves Mounted with Anchor Bolts

Note: We recommend that a minimum space of 1" be allowed for any gate or accessory that requires grouting.

When anchor bolts are furnished for mounting the grooves, the location and projection of the anchor bolts will be shown on the installation drawing. In most cases, epoxy or wedge type anchor bolts will be utilized. When hook type anchor bolts are utilized, the anchor bolts should be placed in the holes drilled in the forms at locations indicated on the drawings. The hook ends of the anchor bolts should then be wired to the opposite form or to reinforcing rods to hold the bolts firmly in place.

Where grooves are mounted with anchor bolts it is necessary that a uniform grout pad (non-shrink grout) or a resilient gasket be placed between the groove and the concrete wall. This grout or gasket is necessary to serve as a seal between the groove and the wall and the type will be indicated on the installation drawings. The projection of the anchor bolts, shown on the installation drawings, includes provisions for the grout or gasket.

When a gasket is utilized to seal between the groove and the wall, the wall will need to be straight and plumb. If the wall is not straight and plumb, leakage can occur between the groove and the wall. Removal of the groove, modifications to the wall and re-installation of the groove may be required to rectify this situation.

Grooves should not be mounted directly to a wall without grout or a gasket as this will result in leakage between the groove and the wall.

1. All anchor bolts should be checked prior to installation to ensure that the threads are undamaged. Anchor bolts should be installed as recommended by the anchor bolt manufacturer.

2. For face-mounted installations, install the grooves using the jacking nuts on the anchor bolts as shown on the installation drawing. If the jacking nuts are not installed and the outside nuts are overtightened, groove distortion can occur and this can lead to difficulty installing and removing the stop logs and excessive leakage. Groove distortion can provide an inconsistent mounting surface for the seals that are mounted on each stop log. Note that jacking nuts are not shown or applicable for channel-mounted installations. Shims may be used to level channel-mounted grooves if needed.

3. In most cases, two nuts will be provided for each anchor bolt. Refer to the installation drawings for details. The jacking nut, should be installed on the anchor bolt prior to mounting the groove, **leaving approximately 1 inch for the insertion of grout**. The jacking nut needs to be positioned to ensure that the groove will be mounted vertically even if the concrete wall is not straight and plumb.

4. After anchor bolt and jacking nut installation, the groove should be lifted and carefully set in place in such a way as to not damage the threads on the bolts. After the groove is mounted on the anchor bolts, attach the other nuts on the anchor bolts. The use of the double nut arrangement helps to ensure that the groove will be mounted straight and plumb and can be firmly tightened into position without distortion.

5. With the groove located approximately 1 inch from the wall, forms should be mounted around the flange and a non-shrink grout should be placed between the groove and the concrete wall. The grout needs to be completely applied around the perimeter of the groove as shown on the installation drawings. All voids should be filled with grout to ensure that leakage cannot occur between the groove and the wall.

6. Care should be taken to avoid getting grout on the inside of the groove or on the seals. All grout that adheres to the inside of the groove or the stop log seals should be removed.

7. Channel-mounted invert sill plates should be installed as shown using the jacking nuts provided to level the plate. Grout should be filled as needed between the underside of the sill plate and the channel invert in the same manner as the stop log grooves.

Nut Tightening Torque

Proper tightening of the nuts on anchor bolts holding the groove to the wall may prevent serious problems in operation or performance of the equipment. Tabulated below, are recommended torque values for common fastener sizes.

*DIAMETER (in.)	TORQUE (ft.-lb.)
1/2	35
5/8	75
3/4	100
7/8	150
1	200

Installation /Removal – Stop Logs

After the groove assembly has been installed and cleaned of all debris, stop logs can be added. If provided, use the stop log lifter in conjunction with a crane or similar lifting device to raise the lifter and the stop log. Make sure that the stop log is positioned correctly. The seals should be on the "dry side" so that the water pressure pushes the stop log seals against the inside of the stop log groove. Lower the lifter and stop log into the groove and release using the lanyard release on the lifter. This procedure should be repeated depending on the number of stop logs.

Check the installation drawing to ensure that the correct quantity of stop logs is installed. In some cases, it will be necessary to review the installation drawings to confirm the proper sequence for adding the stop logs. (Some stop logs may be provided with orifices, differing heights, etc.)

The procedure should be reversed when removing the stop logs. When removing the stop logs, handle the stop logs with care. Install the stop logs in the storage racks or in a stack. Handle and store in such a way as to avoid damaging the seals.

Installation Inspection Check List

1. Check the groove assembly to ensure it has been installed straight and plumb.

2. Check the inside of each groove assembly and check each stop log for grout, debris or damage.

3. Check the outside nuts on the anchor bolts for proper tightness.

4. On wall mounted groove, check the grout or gasket used to seal between the groove assembly and the wall.

Maintenance Instructions

Grooves and stop logs should be visually inspected at regular intervals (at least every six months) for signs of damage or corrosive attack. Please keep in mind that corrosion, when it occurs, is generally most prominent at the water line.

Installation Drawings

The drawings submitted by Whipps, Inc. for approval and/or field use, are planned so that the installation drawing is the master reference.

The drawings depict as much as possible of the structure surrounding the supplied equipment. The identification of fasteners and components is done by calling out physical sizes and/or assembly or detail drawing numbers. More information is available on the detail drawings, which have been included with the installation drawing.

Spare Parts

Whipps, Inc. does not typically recommend the stocking of spare parts for stop logs since the equipment is designed for a very long service life.

If a repair part is required, contact the PARTS DEPARTMENT at Whipps, Inc. at 978-249-7924 or www.whipps.com with as much of the following information as possible:

1. Plant name and location.

2. Original (four or five digit) shop order number which is indicated on correspondence and installation drawings.

3. The installation drawing number, and a description of the part, with any other available drawing numbers or the size (width x height) and location of the groove in the plant.

4. Description of damage and cause. (Digital photos of damage are useful.)

5. Approximate delivery requirements.

Field Service Policy

The equipment furnished on this order has been inspected prior to leaving the factory and has been accepted by the freight carrier. Please check the packing list accompanying the shipment for shortages and examine the equipment for damages prior to accepting the shipment. Before handling, storing or installing this equipment, read the installation manual that accompanies the shipment.

Damage In Transit

If the equipment has been damaged in transit, the purchaser is responsible for filing the claim with the transport company. Contact Whipps, Inc. for assistance in filing the claim.

Installation, Inspection and Adjustment

Installation supervision, inspection of installed equipment and certification of satisfactory initial operation are not included unless specifically indicated on the customer's purchase order and accepted by the company. Otherwise, Whipps, Inc. will provide this service at the standard published charges.

Field Issues

If trouble develops either in the installation, operation or performance of the equipment, the installation manual and drawings should be checked to determine if the equipment has been installed properly. If proper performance or operation cannot be obtained and assistance from the factory is desired, please contact Whipps Inc or the local representative. Arrangements will be made to send a service technician to the job site if this is required. The service technician will make a thorough examination of the problem and if the equipment is faulty in workmanship or material, the necessary repairs will be made by the factory at no cost to the purchaser if within the warranty period.

If, however, the problem is due to faulty installation or adjustment, the cost of the field service will be charged to the purchaser.

If repairs are made in the field by the purchaser or authorized by the purchaser, backcharges for these repairs will not be accepted by the company unless the company has been notified prior to the incurring of these costs and has accepted the responsibility for these repairs.

Whipps, Inc. will not be liable for contingent costs or costs of delays due to the faulty equipment and the repairs thereof.

Field Service Charges

Field service charges begin from the time of departure until the return of the service person and include a daily rate plus travel and subsistence expenses. Premium day and hour rates will be charge on Saturdays, Sundays, and Holidays and for time spent before 6 a.m. or after 5 p.m., or over eight hours per day. A schedule of Field Service charges is available from the Whipps, Inc. Field Service Department.

Whipps, inc.

P.O. Box 1058 • 370 South Athol Rd.
Athol, Massachusetts 01331
Phone: (978) 249-7924
Fax: (978) 249-3072

TROUBLE SHOOTING GUIDE - STOP LOGS

As outlined in the beginning of this manual, the installation is the single most important aspect and consideration of a properly functioning Stop Logs.

Symptom	Cause	Remedy
Excessive Leakage (through stop logs)	Concrete, grout or debris on the slide, seats and/or seals.	***Remove foreign material.***
Excessive Leakage (through stop logs)	Distorted frame (i.e. anchor bolts tightened unevenly or insufficient bracing during embedment.	***Remove groove from wall and re-mount if undamaged. Consult factory if damaged.***
Excessive Leakage (through stop logs)	Damaged seals.	***Open gate and visually Inspect seals for damage. Consult factory for replacement seals.***

For problems not covered in this guide, please consult the Whipps, Inc. Service Department between 8:00am - 5:00pm EST at (978) 249-7924.

If possible, please have the five-digit Shop Order Number and/or the Whipps drawing number of the equipment when calling.

Preventative Maintenance

Stem - Where stem guides are used they should be aligned on the channel to provide uniform clearance between the stem and the stem guide halves. Visually inspect the threaded portion of the stem. It must be clean and free of foreign matter, including dirt or sand, and lubricated with suitable industrial grease. Clean with a dedicated stainless steel brush to prevent cross contamination, apply grease. Do not use a carbons steel brush! The stem should be thoroughly cleaned and lubricated, with a heavy-duty grease at start-up and every six months. We suggest every 3 months whenever exposed to severe dust or heavy particles. The stem must remain free of grit and be sufficiently lubricated to prevent accelerated wear to the operating nut (mounted on the slide).

Seals- Open gate and visually inspect seals for damage. Consult factory for replacement seals.

Manual Operators - Manual operators are lubricated at the factory and do not require further lubrication before initial operation. The stop collar should be threaded on to the stem only after the operator has been installed and the gate is in the fully closed position. There should be approximately 1/16" of clearance between the bottom of the stop collar and the top of the operating nut. The set screw should then be tightened to hold the stop collar in place. The crank or handwheel should turn easily. If there is any binding in operation, check to be sure that the stem guides and stem are properly aligned. At least once a year, all grease fittings should be lubricated with a small amount of Mobil Grease Special or equal, which will not harden in cold weather or become liquid in warm weather.

Electric Operators - The instruction manual furnished with electric motor-driven floor stands should be read carefully before the unit is installed. The gate must be manually opened about 3" before initial electric operation is attempted. Check motor rotation by activating the "open" circuit making certain the gate travels in the "open" direction. Revise motor leads to obtain proper rotation if necessary. The gate should not be operated electrically through its full travel until both "closed" and "open" geared limit switches have been properly set. Adjust the "open" switch so that the opening cycle does not allow the end of the thread of the operating stem to enter the floor stand nut. Geared limit switches cannot be factory set and must be set by the contractor at the jobsite. The torque switches have been properly set at the factory and should not need adjustment.

Appendix B: Wetland Resource Area Memorandum

MEMORANDUM

To: Eric Ford, Division of Ecological Restoration

From: Ben Wollman, Horsley Witten Group, Inc.

Date: June 30, 2024

Re: Wetland Resource Areas – Sharon Great Cedar Swamp

cc: Josh Philibert, Town of Sharon; Kimberly Groff, Southeast New England Program

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This memorandum documents data collection and site findings from wetland resource area and associated boundaries determinations performed by Horsely Witten Group, Inc. (HW) wetlands biologists during site visits in April and June 2024. HW wetland resource area delineations and data collections were performed along and adjacent to a section of an unnamed perennial stream that flows south through the Great Cedar Swamp wetlands in Sharon, MA. The wetland resource area boundaries within the vicinity of the stream corridor were delineated starting at the culvert at Wolomolopoag Street to approximately 800 feet upstream of the culvert (the Project Site).

Wetland Resource Areas

The Project Site supports freshwater wetland resource areas, as defined under the Massachusetts *Wetlands Protection Act* (M.G.L. Ch. 131 § 40) and regulations (310 CMR 10.00), as well as the Town of Sharon Wetland Protection Bylaw (Chapter 262) and associated Rules and Regulations of the Sharon Conservation Commission. An HW wetlands biologist identified and delineated these resource areas during site visits on April 26 and June 7, 2024. Jurisdictional areas identified at or adjacent to the site include Inland Bank (State and Town); Bordering Vegetated Wetland (BVW); Riverfront Area (RA); Land Under Water Bodies and Waterways (LUW), and the 100-foot Buffer to Bank and BVW.

HW followed wetland resource area identification and on-site delineation procedure guidelines described in the Massachusetts *Wetlands Protection Act* (M.G.L. Ch. 131 § 40), the Town of Sharon Wetlands Protection Bylaw (Chapter 262), and their associated regulations, including those described in the Massachusetts Department of Environmental Protection (MassDEP) handbook, entitled *Massachusetts Handbook for Delineation of Bordering Vegetated Wetlands* (September, 2022).

Prior to conducting field delineations, HW reviewed existing source data, including USGS Geological Survey 7.5 minute topographic maps, Massachusetts Department of Environmental Protection (MassDEP) wetlands source data available through the Massachusetts Geographic Information System (MassGIS), United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) soils survey, U.S. Fish and Wildlife Service National Wetland Inventory (NWI) maps, and other source data to identify the presence of jurisdictional wetlands and waters of the United States within the site. This information was used to compile base maps to assist in the understanding of the hydrologic variables, soils conditions, and vegetation communities prior to commencing field visits.

A brief description of the regulatory definitions and the observed resources areas is provided below.

Bank (Inland)

Bank is defined at 310 CMR 10.54(2)(a) as "*the portion of land surface which normally abuts and confines a water body. It occurs between a water body and a vegetated bordering wetland and adjacent floodplain, or, in the absence of these, it occurs between a water body and an upland. A Bank may be partially or totally vegetated, or it may be comprised of exposed soil, gravel or stone. The upper boundary of a Bank is the first observable break in the slope or the mean annual flood level, whichever is lower. The lower boundary of a Bank is the mean annual low flow level*" [310 CMR 10.54(2)(c)].

The Town of Sharon Wetlands Protection Bylaw (Chapter 262-2) defines Bank as, "*An area which normally abuts and confines a water body; the lower boundary being the mean annual low flow level, and the upper boundary being the first observable break in the slope or the mean annual flood level, whichever is higher.*"

The site supports both State-defined and Town-defined Banks along both sides of the unnamed perennial stream channel (**Photos 1-4**). The upper boundary of the Town-defined Bank was delineated at the first break in slope upgradient of the stream. The upper boundary of the State-defined Bank was delineated at the mean annual flood/high water (MAHW) elevation, which was determined by a combination of field indicators of bankfull conditions including changes in slope, changes in vegetation, stain lines, changes in bank material, and bank undercuts.

The unnamed perennial stream occurs within a drainage ditch that lies along the Great Cedar Swamp's western edge, which was created to drain water from the swamp to prevent flooding of an adjacent residential subdivision ("The Heights"). The soil material that was excavated to create the ditch was placed on each side of the ditch, simultaneously creating elevated berms/dikes between the ditch and adjacent forested wetland areas of the Great Cedar Swamp. The stream is contained by the ditch, which has tall, steep slopes rising from the stream channel

to the top of the berms to the east and west. The lower section of the Bank (up to the MAHW elevation) is mostly unvegetated and comprised of an unconsolidated mix of materials that include cobbles, gravel, sand, and silt, where the slope is subject to more frequent erosive forces of the stream during flooding events, as is evident from the lack of vegetation, collapsing sections of the slope, and undercutting occurring in some sections. The upper section of the Bank (up to the first break in slope) is vegetated with a mix of trees, shrubs, and herbaceous species including red maple (*Acer rubrum*), spruce (*Picea sp.*), black cherry (*Prunus serotina*), white pine (*Pinus strobus*), sweet-pepperbush (*Clethra alnifolia*), common blackberry (*Rubus allegheniensis*), multiflora rose (*Rosa multiflora*), Japanese barberry (*Berberis thunbergia*), Japanese knotweed (*Fallopia japonica*), Canada-mayflower (*Maianthemum canadense*), and hay-scented fern (*Dennstaedtia punctibobula*).

HW delineated the landward boundary of each Bank with a series of consecutive blue flagging stations labeled as follows:

STATE BANKS

- MAHW 100 to MAHW 123 (west side of stream), and
- MAHW 200 to MAHW 216 (east side of the stream);

TOWN BANKS

- BANK 100 to BANK 120 (west side of the stream), and
- BANK 200 to BANK 216 (east side of the stream);



Photo 1. Looking south toward the Wolomolopoag Street culvert from the stream bed.



Photo 2. Looking southeast at the existing rip rap baffle water control structure from the top of the western side of the ditch.



Photo 3. Looking west at the lower and upper sections of the western Bank rising from the stream channel. Lower Bank (up to MAHW) is evident from the lack of vegetation and water stain line.



Photo 4. Looking north at the stream and associated Bank sections from the top of the slope at the northern end of the Project Site.

Bordering Vegetated Wetland

Bordering Vegetated Wetland (BVW) is defined at 310 CMR 10.55(2)(a) as: "*freshwater wetlands that border on creeks, rivers, streams, ponds and lakes. The types of freshwater wetlands are wet meadows, marshes, swamps and bogs. Bordering Vegetated Wetlands are areas where the soils are saturated and/or inundated such that they support a predominance of wetland indicator plants. The boundary of Bordering Vegetated Wetland is defined at 310 CMR 10.55 (2)(c) as the line within which 50% or more of the vegetational community consists of wetland indicator plants and saturated or inundated conditions exist.*"

The Town of Sharon Wetlands Protection Bylaw states that, "*Bordering vegetated wetlands are freshwater wetlands. These are resource areas where groundwater discharges to the surface and where, under some circumstances, surface water discharges to the groundwater. Vegetated/freshwater wetlands are likely to be significant to public or private water supply, to groundwater supply, to flood control, to storm damage prevention, to prevention of pollution, to the protection of fisheries, to the protection of shellfish, and wildlife. The physical characteristics of vegetated wetlands are critical to the protection of interests specified in MGL c. 131, § 40. Types of freshwater wetlands are wet meadows, marshes, swamps, and bogs. They are areas where the topography is low and flat, and where the soils are annually saturated. Said resource area shall be protected, whether or not they border surface waters.*" (Chapter 262-2).

Freshwater Wetlands are defined by the Sharon Wetlands Protection Bylaw (Chapter 262-2) as, "*Vegetated wetlands, and consist of any area of at least 2,000 square feet where surface water and/or groundwater, or ice at or near the surface of the ground, supports a plant community dominated (at least 50%) by wetland species and/or exhibits other evidence of hydrology. They are otherwise defined in MGL c. 131, § 40.*"

The Project Site supports red maple swamp BVW areas within broad, flat, low-lying basins adjacent to both sides of the unnamed stream channel/ditch (**Photos 5-8**). There is a small section/strip of upland area between the top of the ditch slope and the boundary of the BVW on both sides of the ditch, which represents the elevated berms that were created from the excavation of the ditch.

Plant species observed within the BVW areas include red maple (*Acer rubrum*), river birch (*Betula nigra*), black gum (*Nyssa sylvatica*), white pine (*Pinus strobus*), sweet-pepperbush (*Clethra alnifolia*), highbush blueberry (*Vaccinium corymbosum*), round-leaf greenbrier (*Smilax rotundifolia*), Canada mayflower (*Maianthemum canadense*), and skunk-cabbage (*Syplocarpus foetidus*).

HW delineated the landward boundary of each BVW area with a series of consecutive pink flagging stations labeled BVW 100 to BVW 118 (west side of stream) and BVW 200 to BVW 222 (east side of stream).



Photo 5. Looking north at the western red maple swamp BVW area from the top of the berm upland area, near the southern end of the Project Site.



Photo 6. Looking north at the upland berm area between the stream ditch and the western BVW area, near the southern end of the Project Site.



Photo 7. Looking south toward the 154 Wolomolopoag Street property from within the red maple swamp BVW on the eastern side of the stream ditch.



Photo 8. Looking north at a representative section of the upland berm area between the stream ditch and the eastern BVW area, north of the 154 Wolomolopoag Street property.

Riverfront Area

Riverfront Area is defined at 310 CMR 10.58(2)(a)(3) as *"the area of land between a river's mean annual high-water line measured horizontally outward from the river and a parallel line located 200 feet away (…)*

> *2. Mean Annual High-water Line of a river is the line that is apparent from visible markings or changes in the character of soils or vegetation due to the prolonged presence of water and that distinguishes between predominantly aquatic and predominantly terrestrial land. (…).*"

The Sharon Wetland Protection Bylaw applies the same Riverfront Area definition found in 310 CMR, as stated above.

The mean annual high-water (MAHW) line of the unnamed stream at the Project Site coincides with the upper boundary of the State Banks along the edge of the channel and was determined by a combination of field indicators of bankfull conditions including changes in slope, changes in vegetation, stain lines, changes in bank material, and bank undercuts. The Riverfront Area extends outward from the MAHW line and overlaps with the adjacent Town Bank and BVW areas, as well as the buffer zones associated with Bank, BVW, and the sections of upland along the top of the berms.

FEMA Designation

According to the Federal Emergency Management Agency (FEMA) National Flood Hazard Map (Community Panel Numbers 25021C0352E & 25021C0354E, effective July 17, 2012) the majority of the Project Site occurs within Other Areas of Flood Hazard, Zone X (0.2% annual chance of flooding, areas of 1% annual chance of flooding with average depth less than one foot or with drainage areas of less than one square mile), while a small section of the Project Site (adjacent to Wolomolopoag Street) occurs withing a Special Flood Hazard Area, Regulatory Floodway (Error! Reference source not found.).

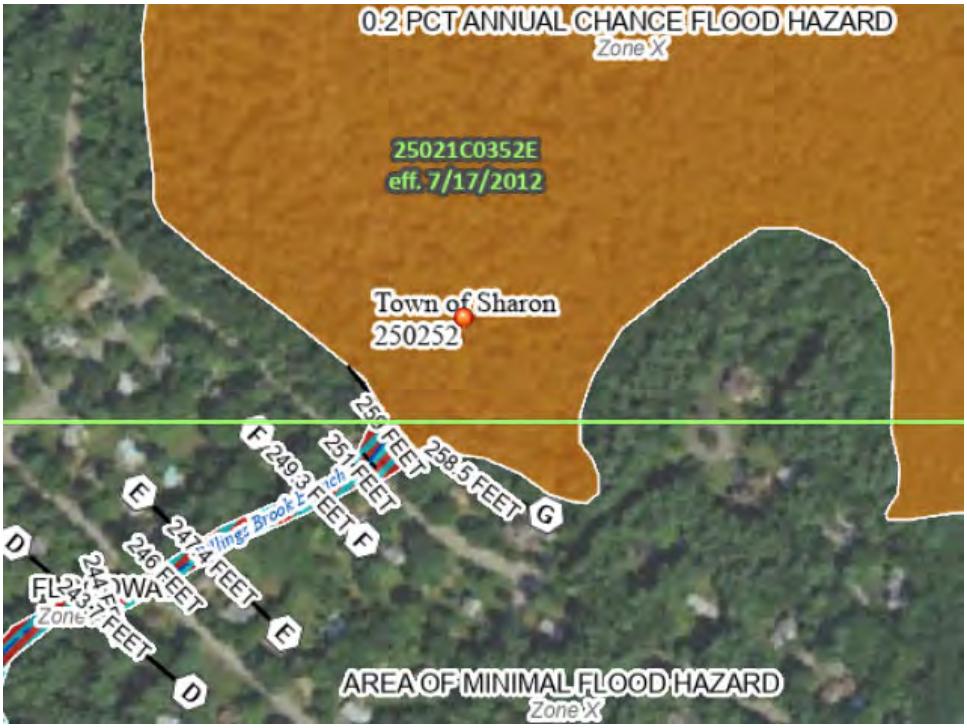

Figure 1. Excerpt from the Federal Emergency Management Agency (FEMA) FIRMette for the site.

State-listed Rare Species Habitat

According to the most recent version of the *Massachusetts Natural Heritage Atlas* (15th Edition, August 1, 2021), the Project Site is not located within areas mapped as *Estimated Habitat of Rare Wildlife* (EH 453) and *Priority Habitat of Rare Species* (PH 559) as designated by the Massachusetts Natural Heritage and Endangered Species Program (NHESP). Additionally, there are no *Certified Vernal Pools* or *Potential Vernal Pools* mapped within the vicinity of the Project Site.

Invasive Species

Invasive species (as defined by the Massachusetts Invasive Plant Advisory Group or MIPAG) were present at the Project Site within some sections of the wetland resource areas and buffer zones. MIPAG identifies invasive plant species as "non-native species that have spread into native or minimally managed plant systems in Massachusetts," and which "cause economic or environmental harm by developing self-sustaining populations and becoming dominant and/or disruptive to those systems." Invasive species observed include multiflora rose, Asian bittersweet, Japanese knotweed, and Japanese barberry. Invasive plant densities were low at the site overall, but some sections along the eastern Town Bank area were observed to contain a relatively high-density population of Japanese knotweed (**Photo 9**). For future planning purposes, the Town of Sharon may wish to develop a management plan for reducing or eliminating these plants at this site to allow for the establishment of naturally vegetated protective buffers to the wetland resource areas.



Photo 9. Higher density Japanese knotweed population observed along the eastern State Bank area.

References

Sharon Conservation Commission. Accessed June 2024 via https://www.townofsharon.net/conservation-commission.

Federal Emergency Management Agency (FEMA) Flood Map Service Center. Accessed June 2024 via https://msc.fema.gov/portal/home.

Massachusetts Office of Geographic and Environmental Information (MassGIS). Accessed June 2024 via http://www.mass.gov/mgis/.

Massachusetts Wetlands Protection Act (WPA). Massachusetts Department of Environmental Protection. 2017. Accessed June 2024 via https://www.mass.gov/doc/310-cmr-1000-the-wetlands-protection-act/download.

Massachusetts Invasive Plant Advisory Group. Accessed June 2024 via https://massnrc.org/MIPAG/.

Appendix C: Model Simulations Technical Memorandum (February, 2024)

Horsley Witten Group
Sustainable Environmental Solutions
90 Route 6A • Unit 1 • Sandwich, MA 02563
508-833-6600 • horsleywitten.com

MEMORANDUM

To: Kimberly Groff, Southeast New England Program

From: Neal Price

Date: February 16, 2024

Re: Model Simulations Summary Technical Memorandum – Sharon Great Cedar Swamp

cc: Josh Philibert, Town of Sharon; Eric Ford, Division of Ecological Restoration

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The Horsley Witten Group, Inc. (HW) is pleased to submit this memorandum summarizing groundwater modeling conducted this past summer and autumn for the area in and around the Great Cedar Swamp (GCS) in Sharon, Massachusetts (the Site). Modeling was conducted to evaluate five potential control structure elevations for the existing drainage ditch. This work builds upon the conceptual MODFLOW groundwater model for the area developed during a previous phase of this project. MODFLOW simulates aquifers as a three-dimensional grid of interconnected cells. Properties (e.g. hydraulic conductivity, and recharge) are prescribed for each cell such that the model can simultaneously solve groundwater flow equations for each cell to its adjacent cells, and so on throughout the grid. Ultimately, simultaneous solutions of the thousands of interconnected cell equations produce predicted water levels for each cell based on the various input parameters so that a groundwater contour map can be produced and an understanding gained of how groundwater moves through the aquifer. Changing the prescribed parameters of cells allows the modeler to simulate scenarios of different groundwater conditions,

The field work that supported the development of this model, which included manual water level measurements, manual streamflow measurements, surveying, and the use of water level loggers, was summarized in a previous technical memorandum.

Background

The GCS consists of 250± acres of land within the Town of Sharon in Norfolk County, Massachusetts. The site is owned and managed by the Town and consists of a large wetland complex that includes an Atlantic White Cedar (*Chamaecyparis thyoides*) swamp – a globally rare community type. The GCS is an important natural resource to the Town due to the important open space and habitat protection services it offers. In addition, because the GCS is located at relatively high elevation in the local aquifer system and is undeveloped, it provides clean groundwater recharge that contributes to most of the Town's public water supply wells. The GCS also feeds Lake Massapoag, a popular recreational resource.

The GCS is currently impaired by a drainage ditch that lies along its western edge. This ditch was created during the construction of an adjacent residential subdivision (“The Heights”). The ditch drained groundwater from the near surface, lowering groundwater levels and increasing separation for the construction of septic systems, basements, catch basins, and other subsurface infrastructure. The ditch also had the effect of draining water and reducing groundwater levels within the GCS. The ditch has lowered groundwater levels beneath a large portion of the GCS sufficiently to cause die-off of the wetland vegetation and replacement with more upland facultative species. The decline in groundwater levels has led to a noticeable amount of soil subsidence within the GCS.

The value of the GCS and the detrimental impacts of the drainage ditch have become more recognized and understood since the Heights were originally developed. In an early attempt by the Town to mitigate these impacts, a small baffle structure of rip rap stone was constructed in the ditch approximately 200-feet upstream of the Wolomolopoag Street culvert. The structure, composed of a riprap base atop the streambed and additional riprap reinforcing the streambank, impounded a small amount of water (approximately 1.5 feet above the streambed) and caused minimal increase to groundwater levels in the GCS surrounding the ditch.

The modeling effort described in this report was undertaken to assess the impacts of proposed a flow control structure designed to impound water within the GCS ditch to a higher elevation than the existing rip rap baffles. The hypothetical flow control structure assessed in this report would be installed in a similar position in the ditch to the existing baffle structures. The flow control structure would retain water behind it, reducing overall discharge downstream through the Wolomolopoag Street culvert. The retained water, impounded behind the structure, would increase the stream stage in the ditch to the elevation of the flow control structure. As the stream stage increases, the potential for water to enter the stream from the surrounding aquifer (e.g. the potential to drain) decreases correspondingly. Five scenarios, representing five different impoundment elevations for the flow control structure, were assessed in this study. Model results were assessed to balance the competing interests of increasing water levels in the GCS while avoiding negative impacts to subsurface infrastructure in the Heights neighborhood.

Data Sources Used in Model Development

Several data sources are needed to produce a groundwater model. In this effort, modeling combined data sources from publicly available resources, previously written reports, and data collected on site in 2010 and 2023 by the project team. The following data sources were primarily utilized to determine physical characteristics of the area including surficial geologic materials and ground surface elevations.

- Surficial geology was determined based on the MassGIS Surficial Geology (24K) layer.
- Surface elevation data was determined based on the 2021 Central Eastern MA LiDAR survey.
- The location of the drainage ditch, Canoe River, and Beaver Brook were based on the MassGIS 25K hyrdrography dataset. Elevations for these surface water features were

determined based on the LiDAR elevation for each model cell along each of those watercourses.
- A USGS index well (MA NNW27R) was incorporated to understand the temporal context of the regional hydrology.
- Numerous borings in the GCS and surrounding area were compiled by IEP Inc for an Aquifer Protection Study in 1987, providing local information about subsurface conditions.

The results of multiple fieldwork efforts provided further detail about the GCS and the drainage ditch, as well as calibration targets for the model. Two separate water level measurement events occurred throughout the GCS in spring and summer of 2010, producing interpolated water table surfaces for the entire GCS for each sampling event, as well as specific elevations for water table elevation at monitoring points throughout the area. These water table maps served as the basis for calibration, as discussed later in this report. In 2011, surveyors from Horsley Witten Group produced a transect profile of the drainage ditch elevation as well as cross sections of the stream channel at two locations.

In 2023, Horsley Witten, in partnership with DER and the Town of Sharon, undertook additional field work on the site installing continuous water level loggers in monitoring wells and stream gauging sites, locating infrastructure features, and further characterizing the GCS ditch in the vicinity of the Wolomolopoag Street culvert. Locations and elevations of features were recorded using a real time kinematic global positioning system (RTK GPS) during the 2023 monitoring effort. The 2023 monitoring effort is described in detail in a Field Data Technical Memo delivered to DER on June 30, 2023.

MODFLOW Groundwater Model Development

Model development was performed using Groundwater Vistas version 8 (Environmental Systems, Inc.) and MODFLOW-2005 (United States Geological Survey). MODFLOW-2005 is a three-dimensional finite-difference groundwater model. Properties and boundary conditions are assigned to cells within the finite-difference grid using the Groundwater Vistas interface. The MODFLOW-2005 model then performs calculations based on this grid, yielding a numerical solution (model result) representing groundwater conditions which satisfy the conditions set out in the model. Following initial development of the model, parameters were adjusted through a manual calibration process to yield results which reasonably reflect observed groundwater conditions in the project locus.

Model Domain

The model domain was chosen to include significant features of the four surface water watersheds which surround the GCS. To the east, the model domain extends into Massapoag Lake. To the north, the model includes approximately 9,300 linear feet of Beaver Brook, including a reach of the Brook which runs approximately east-west. The hydrogeologic divide created by this reach was included as the approximate northern extent of the model. The western extent of the model includes the Billings Brook tributaries to the Cedar Swamp Brooks, which is the ultimate drainage of the ditch within the GCS. The southern extent of the model

domain includes the Canoe River headwaters south of the GCS. Approximately 9,000 linear feet of the Canoe River's highest reaches were included to characterize this drainage area's influence on hydrology of the GCS. The model domain is indicated in Figure 1.

Boundary Conditions

Boundary conditions included in the model represent stream cells (for the GCS ditch, Beaver Brook, and Canoe River), constant head cells (for major surface water bodies), and no-flow cells at the outer edge of the model domain. The boundary conditions in the model are shown in Figure 1.

Constant head boundaries are only defined by a single parameter (head), which must be met in the solution to the model. Constant head boundaries selected for this model represent major surface waterbodies within the model domain. Heads of these waterbodies were determined based on LiDAR derived water body surface elevations. The LiDAR elevation of Massapoag Lake was assessed against field measured values to determine that the elevations used for constant head boundaries represent long term average conditions. The nearest modeled constant head boundaries to the Heights neighborhood and the GCS are Wolomolopoag Pond, approximately 3,000 feet to the west, and Lake Massapoag, approximately 2,000 feet to the east.

Stream cell boundaries allow for water to enter or exit the aquifer system via stream cells. The parameters defining a stream boundary include the bottom elevation of the stream, the stage of the stream, and components of streambed conductance. The bottom elevation of streams in the model was determined based on high resolution LiDAR imagery of the model domain. For the GCS drainage ditch, LiDAR elevations were validated against a stream profile survey performed by HW in 2011 and survey points located with RTK GPS in 2023. Stage throughout the ditch was estimated based on average stage data collected during the field monitoring task of this project, interpolated between gauging sites. Conductance of the stream was adjusted to calibrate model mass balance results with field-estimated baseflow discharge volumes for the drainage ditch.

Physical Characteristics, Model Properties

Surficial geology of the GCS is swamp deposits overlaying coarse glacially stratified deposits. The depth of swamp deposits in the GCS were based on observations of swamp sediment depth observable in the drainage ditch as well as an organic sediment depth transect conducted along the eastern edge of the railroad bed within the GCS, included in the Sharon GCS Progress Report, June 30, 2012 (Fletcher et al, 2012). Combined, these data points suggest the swamp deposits extend approximately 10 feet below ground surface throughout the swamp. Outside of the GCS and nearby swamp areas, the predominant surficial geology is coarse glacially stratified deposits. Till is present to the south and east of the GCS in the vicinity of Barefoot and Dudley Hills. It is assumed that glacial till underlays the entire model domain, including in areas where it is not present on the surface. Areas of shallow bedrock were identified from surface geology maps. The depths to bedrock in areas of till and glacial stratified deposits were interpolated from boring information produced during the Aquifer Protection Study

conducted by IEP in 1987. “Refusal” in these boring logs was assumed to represent a low-conductivity fractured bedrock underlayment.

Based on the information reviewed, the model was vertically separated into 5 layers. The lowest layer represents low conductivity fractured bedrock which is assumed to underlay the entire model domain. This layer has a flat bottom at an elevation of 100 feet. The top of this layer and all other layers were assigned relative to surface elevations. The top two layers are each 10 feet thick. In the vicinity of the GCS, the top layer represents swamp deposits. The third layer is 20 feet thick. The fourth layer is 40 feet thick. The lowest layer varies in thickness, with a top elevation 80 feet below the ground surface and constant bottom elevation of 100 feet. Surface elevation contours and top layer conductivity and recharge properties are shown in Figure 2.

Calibration

Development of the baseline model sought to represent long-term average conditions in the GCS. Calibration objectives included head values at monitoring well locations and baseflow contributions from the aquifer to the ditch. The primary means of adjusting calibration of the model involved the adjustment of the lateral and vertical conductivity parameters for the various aquifer materials in the model, as well as adjusting recharge values throughout the model. Aquifer losses to the drainage ditch were calibrated by adjusting the conductance terms of the streambed. Combined, aquifer material properties and streambed conductance properties were calibrated to match head targets for select target points throughout and surrounding the GCS as well as streamflow volumes. Calibration information, including the baseline model water table contours, is indicated in Figure 3.

Head Calibration

One of the primary outputs from a numerical groundwater model is hydrologic head simulated for each cell in the model domain. Head is the hydrostatic pressure within the aquifer, which can be measured as the water elevation measured in a monitoring well. For unconfined aquifers such as the GCS, the hydraulic head measured in a well with a screened interval below the water table will be approximately equal to the elevation of the water table at that location. The modeling team assessed previous groundwater measurements taken at the GCS in 2010, data from the 2023 monitoring effort, and the long-term trends at an index well (MA NNW27R) to determine long-term average groundwater conditions at the project site.

Two rounds of water level measurement were performed in 2010 – one on April 7 and another on August 10. During each event, depth to water was measured at nearly every well in the area yielding specific water elevation values as well as an interpolated water table contour map of the area. At the time of the April 7, 2010 gauging, water levels at the index well were 1.15 feet higher than the long-term average condition. During the August 10, 2010, gauging effort, water levels at the index well were 1.24 feet lower than the long-term average water level. Based on this relationship, the modeling team determined that the average of the water levels observed during these two monitoring events would be a sufficiently accurate approximation of long-term

average groundwater levels in the GCS. Monitoring points sampled during these two events were selected to be calibration targets based on spatial variability throughout the area of interest. Water level measurements at those target locations are summarized in Table 1 below.

Figure 3 shows the location of these calibration targets and the residual differences between model-calculated and target heads throughout the site. The root-mean-squared error for the model targets was computed to be 2.46 feet and the range of target heads was 28.07 feet. Residuals are generally lower in the vicinity of the GCS, and higher at targets further from the GCS. While there is no common standard regarding the allowable amount of error in a groundwater model, it is a common practice to compare the RMS error of residual values to the range of target heads as a percentage to assess model accuracy. By this metric, the calibrated model reflects an RMS error of 8.8%, which is within the commonly accepted 10% standard for this metric.

Table 1: GCS 2020 Groundwater Monitoring Results

			Monitoring Events		
	Northing	Easting	4/7/2010	8/10/2010	Average of 2010 monitoring values
HARDING ST	2864160.8	740572.6	253.61	251.65	252.63
MOHAWK ST	2861280.6	740062.19	258.46	254.76	256.61
S MAIN AND HAMPSHIRE	2864682.5	738043.09	236.16	232.82	234.49
A8E	2859848.3	737400.73	256.92	255	255.96
S Main and sunset	2863810.1	736333.02	256.5	252	254.25
49 Garden St	2864565.4	739149.86	249.19	244.67	246.93
B2W	2862568.7	737018.11	256.22	254.16	255.19
A1E	2860315.9	736371.41	255.28	251.01	253.145
A2E	2860265.6	736503.67	256.11	251.67	253.89
A1W	2860372.1	736246.99	254.29	250.69	252.49
Essex	2863415.1	738047.48	259.19	254.28	256.735
Sunset	2863393.9	736735.95	258.12	254.26	256.19
MA NNW27R			157.7	155.31	156.505
		Calculated Long Term Average			156.55

Ditch Baseflow Calibration

Streamflow was not measured during the 2010 groundwater monitoring events which formed the basis for head calibration. Streamflow was measured four times at two monitoring points during the 2023 monitoring period. Streamflow measurements at SF1, the further-downstream of the monitoring points, are summarized in Table 2 below.

Table 2: Streamflow measurement events (2023)

Date	SF1 Flow (cfs)	SF1 Flow (cfd)
1/31/23	4.08	352,512
2/14/23	1.34	115,776
3/16/23	7.53	650,592
4/11/23	0.93	80,352
Average	3.47	299,808

Stream stage was continuously measured using Van Essen Diver pressure transducers which record water column height above the diver at regular intervals. Hydrographs of stream stage were analyzed to relate streamflow measurements to general hydrologic condition in order to assess the baseflow contribution of groundwater into the stream. Due to the frequency of precipitation events during the monitoring period, stream stage did not stabilize following precipitation events before the next precipitation event began. Falling leg hydrograph slopes remained in steep decline until the next rising leg began. Only one period of relatively stable, uninterrupted streamflow was observed between February 16 and February 22, 2023. Between February 22 and the end of monitoring on April 11, 2023, streamflow values did not stabilize to a steady baseflow condition.

As discussed in the Field Data Technical Memo, the 2023 monitoring period occurred in wetter-than-average regional hydrologic conditions. Groundwater elevations in USGS MA-NNW27R did not fall below 74th percentile values during the monitoring period, and the average value was the 87th percentile. December and January (preceding the monitoring period) and March (during the monitoring period) all exceeded long term average precipitation volumes. Given the regional hydrologic conditions during which monitoring took place, coupled with the lack of hydrograph stabilization, the lowest streamflow measurement on April 11, 2023 was interpreted to most closely resemble a long-term-average baseflow condition.

Streams in the model were represented using the Streamflow Routing package (STR). Streams are primarily characterized in the model based on their bottom elevation, stage, and the conductance of the streambed. Stream bottom elevations were determined based on minimum LiDAR elevations within specific stream cells, adjusted as appropriate based on elevation data collected during the 2010 and 2023 surveys. Stream stage was estimated based on the average depth of water in the ditch during the monitoring period, added above the stream bottom elevation. Conductance refers to the ability of water to pass through the streambed material between the stream channel and the underlying aquifer. The drainage ditch is cut into the GCS providing an almost direct connection to the surrounding material. As such, stream cells were assigned a streambed thickness of 1 foot and a hydraulic conductivity of 75 feet per day to represent a condition where the streambed offers this direct connection with the surrounding material.

Stage, bottom, and conductance allow the model to calculate the amount of water entering or exiting the aquifer in each stream cell. The potential of the stream to gain or lose water is determined by comparing the stream stage with the aquifer head at each cell. This head-stage potential is tempered in the model by the conductance of the streambed, however due to the high conductance of ditch streambed materials reflected in the model this resistance is minimal. The high conductance afforded to stream cells in the model, representing the direct connection of the stream to the surrounding material, means that the elevation of the stream bed and the stage of the stream are the key factors for modeling stream dynamics in this case. Mass balance results for the drainage ditch upstream of Wolomolopoag Street in the baseline model yields a net stream gain of 77,614 CFD. This value is 97% of that of the April 11th streamflow measurement and was determined by the modeling team to adequately represent approximate baseflow conditions for the system.

Subsurface Infrastructure Constraints

The drainage ditch within the GCS was originally built to lower groundwater elevations within the Heights neighborhood. Drainage was necessary to provide sufficient clearance for residential infrastructure within the neighborhood (such as stormwater drains and conveyances, septic systems, and residential basements) from groundwater. Even with the ditch draining the GCS, separation from seasonal high groundwater is low in this neighborhood. Horsley Witten Group reviewed septic installation reports provided by the Town of Sharon and surveyed stormwater infrastructure on Sunset Drive and Lee Road to understand the limitation on potential groundwater level increases which this infrastructure imposes.

Septic system engineering plans for 10 residential properties in the Heights neighborhood were provided by the Town and reviewed by HW. During 2023 field work, stormwater structures at the end of Sunset Road and the intersection of Essex and Lee Road were surveyed. Data from these sources is shown in Figure 4. Septic system elevation data is limited in its application due to a lack of information relating elevations on the plan to an established vertical datum. Four of the septic plans reviewed did have an elevation benchmark associated and the plans state that elevations approximately refer to the USGS NGVD datum (NGVD29). LiDAR ground surface elevations and RTK-measured points located during 2023 field work are vertically referenced to the NAVD88 datum. The difference between NAVD88 and NGVD29 orthometric elevations vary spatially. In the vicinity of the GCS, NGVD29 elevations of 0.76 feet higher than the same elevation measured in the NAVD88 datum.

The remaining plans do not mention any datum information. The range of elevations included in the plans without a specified vertical datum are within the range of actual ground surface elevations in the Heights, suggesting that the septic system designers did not use a made-up “reference” elevation (e.g. 100 feet), but rather did attempt to utilize elevations relative to some vertical datum despite not specifying which.

The differences between the NGVD29, NAVD88, or approximate-assumed datums used in reviewed septic plans limits their direct ability to be compared to other data sets in the project. Additionally, the imprecise nature of a groundwater model and the residual differences which remain between calibration targets and modeled heads further complicate the pursuit of direct elevation-to-elevation assessment of modeled scenario impacts. As such, rather than attempting to compare specific septic system bottom elevations to the modeled groundwater elevation at that location, the change in groundwater elevation under proposed conditions was compared to the baseline conditions model results in order to assess the risk of impact on septic elevations.

Six of the septic plans reviewed included information regarding the elevation adjusted elevation of groundwater at that location based on observations during the septic system design process. The septic systems located at 30 Clark Court and 15 Essex Road had the least separation from groundwater, in addition to being the closest systems reviewed to the GCS ditch. Groundwater at 15 Essex Road was noted at a depth 6.72 feet below the design leach field invert, and the adjusted groundwater elevation was just 4.39 feet below the leach field. The percolation rate at 15 Essex Road was reported as 2 minutes per inch, which qualifies this location for the 4-foot

vertical separation requirement under Massachusetts Title V Septic System Requirements. As such, at this most-sensitive receptor, an increase in groundwater elevations by as little as 0.39 feet would violate the septic system requirements as defined in Title V.

	Key Elevations				Datum			
Address	Basement	Leach Field	High Ground Water	Ground Water		Adjusted Groundwater Separation	Groundwater separation	Best GW Data
15 Essex Road	263.38	265.69	261.3	258.9667	Approximate NGVD	**4.39**	6.723333333	Adjusted
30 Clarke Court	256.0*	263.35	No TP	257.8	No datum specified		**5.55**	Groundwater Elv
22 Norfolk Place	258.27	260.1	255.1	253	Approximate NGVD	5	7.1	Adjusted
19 Middlesex Road	264.45	266.19	<259.6	<258.2	Approximate NGVD	6.59	7.99	Adjusted
21 Norfolk Place	261.31	262.87	<257.4	<254.4	Approximate NGVD	5.47	8.47	Adjusted
9 Middlesex Road	257.43*	263.06		255.4	No datum specified		7.66	Groundwater Elv
Systems without test pit information								
22 Lee Road	258.03	260.81			No datum specified	No groundwater data		
24 Suffolk Road	263.63	264.75			Elevation assumed	No groundwater data		
30 Sunset Drive	257.3	258.09			No datum specified	No groundwater data		
4 Lee Road		260.89			No datum specified	No groundwater data		
*Based on reported top of foundation elevation, minus 10 feet								
All elevations are approximate								

Table 3: Septic system key elevations and data limitations based on plans reviewed.

Flow Control Structures Evaluations

The drainage ditch in MODFLOW is represented using the STR (Streamflow Routing) package. This package takes physical characteristics of the stream and calculates the interactions between the stream and the aquifer based on those characteristics. As discussed above, key parameters which define this relationship are the stage of the stream and the conductance of the streambed. If the stage of the stream is higher than the head of the aquifer, then water in the stream has the potential to flow from the stream into the aquifer. Inversely, if the head in the aquifer exceeds the stage of the stream, then water from the aquifer has the potential to flow into the stream. When water enters the aquifer the condition of the stream is called a “losing stream”. When water enters the stream from the aquifer, the stream is considered a “gaining stream”. The ditch in the GCS is a gaining stream throughout the GCS.

Flow control structures will affect the river-aquifer interface by retaining runoff and normal groundwater inflows to the drainage ditch upstream of the flow control structure. In so doing, the flow control structure increases water levels in the stream, increasing its stage. The water retained behind the flow control structure decreases the potential of the stream to drain water from the aquifer, either decreasing the amount that the stream gains or, if the increase causes the stream stage to exceed the local head of the aquifer, causing the stream to contribute water to the aquifer.

The goal of this modeling effort is to assess whether additional water entering the aquifer as a result of a flow control structure in the ditch will impact residential basements, septic systems, and stormwater structures in the Heights neighborhood. Various flow control structures elevations were represented in the model. For each flow control structure, the stage of stream

cells upstream of the structure were adjusted to match the top elevation of the structure, to the point upstream where baseline stream stages exceeded this elevation. The steady-state model solution was calculated, and the resulting water table was compared to the baseline water table elevation to indicate the magnitude and extent of change resultant of each proposed scenario. Results are reported in terms of this difference (i.e., “mounding”) to demonstrate the lateral extent of water table increases under various proposed scenarios.

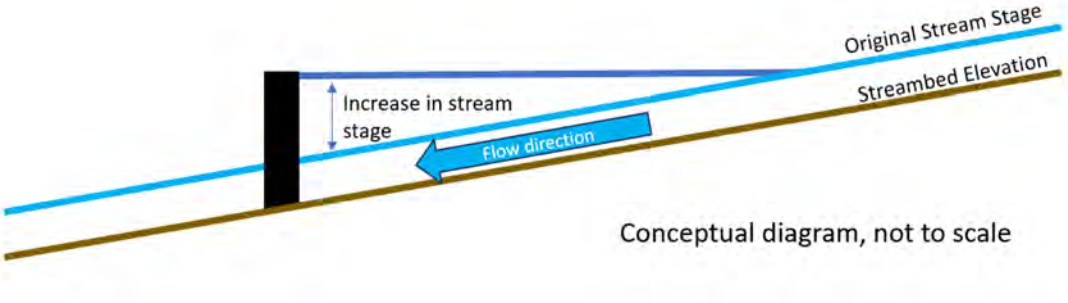

Figure 5: Conceptual diagram of the influence of a flow control structure on stream stage.

Flow control structure scenarios are constrained by the cross-sectional stream channel profile in the vicinity of the target location of the structure. Cross-sectional profiles in the vicinity of the existing riffle structures were surveyed by HW in 2011. At the Wolomolopoag Street culvert, the stream channel was 6.1 feet below the bank, with the banks approximately 23 feet apart. Approximately 1,100 feet upstream, a second cross-sectional profile characterized the ditch with the channel 8 feet below the lowest bank and the banks approximately 27 feet apart. Two existing rip rap baffles were surveyed using RTK GPS during field work efforts in spring of 2023. These two existing structures are approximately 10 feet apart, with the upstream baffle approximately 225 feet upstream of the Wolomolopoag street culvert. The upstream baffle had a top elevation of 251.0 feet. The tops of bank adjacent to this baffle were 257.45 to the southeast and 258.34 to the northwest. The distance bank to bank at this location is approximately 37 feet. The top of this upstream baffle is approximately 1-foot higher than the channel elevation immediate upstream of the baffle.

The riprap baffle structure was assumed to be the prospective location of a proposed flow control structure for modeling purposes. The baffle, located near the downstream extent of the drainage ditch within the GCS, would allow for the maximum amount of water to be retained in the ditch channel at the location furthest away from the infrastructure constraints in the Heights. Stream stage upstream of the baffle in the baseline model was assumed to be equal to the top elevation of the baffle, remaining at this elevation upstream to the point that stream stages, estimated based on stream stage data collected during the monitoring period, exceeded this elevation. As such, the baseline scenario modeled includes the existing baffle. Flow control structure scenarios include additional stream stage above this existing baseline. The theoretical maximum hydraulic retention height for a flow control structure built above this baffle would

therefore be limited to 6.4 feet, which would retain water up to the height of the banks of the ditch. Realistically, a flow control structure would not be built to impound to this maximum amount of water. Flow control structures were modeled in 1-foot increments above the existing baffle structure to a maximum height of an additional 5-feet of water impounded.

The slope of the stream channel determines how far upstream water could be impounded by a 5-foot-tall flow control structure above the existing baffle. Given the existing baffle top elevation of 251 feet, a flow control structure of 5 additional feet would retain water to an elevation of approximately 256 feet. At this elevation, stream stages would increase above typical levels approximately as far upstream as Lee Road. The increase would be less than 1-foot further upstream than Sunset Drive. Lower elevation flow control structures were assessed at 1-foot increments to balance the objectives of maximizing resaturation of the GCS while minimizing impacts to neighboring residents.

Results

For each modeled flow control structure scenario, the resultant water table was compared to the baseline scenario and the difference is reported in Figures 6a-e. Subsurface infrastructure in the Heights neighborhood, such as septic systems and stormwater drainage infrastructure, inherently constrains any effort to mitigate the detrimental impacts of the drainage ditch on the GCS. As shown in figure 6c, a flow control structure raising stream stages to 3-feet above the current baffle structure (to an elevation of 254 feet) would increase long-term average groundwater elevations in the vicinity of 15 Essex Road by approximately 0.3 feet, which is the maximum amount of groundwater elevation increase that could occur without impacting the Title V compliance of the system as designed.

The primary factor which determines the lateral extent of groundwater rise under these scenarios is the distance upstream from the flow control structure to which water is impounded above the baseline stream stage. The highest bottom elevation of the ditch is only 3 feet higher than the top of the existing riffle structure, meaning that a flow control structure which impounds 4 feet of water would cause increased water elevations beyond Sunset Drive, and a 5-foot structure would increase water elevations throughout the entire ditch. The physical proximity of the ditch to the neighborhood means that even a small increase in water elevation could potentially cause negative impacts.

The representation of proposed condition scenarios in the model reflect steady state solutions where the stage of the stream is consistently maintained equal to the proposed flow control structure top elevation until groundwater levels throughout the system form an equilibrium with that elevation. In reality, baseflow and surface runoff contributions to the ditch may not provide enough input to maintain this impoundment consistently and the ditch may not retain water to the control elevation of the structure at all times, resulting in less groundwater elevation increase than that reflected in model results. In this way, the results of the model are inherently conservative.

Given the above considerations, it is recommend that a height-adjustable flow control structure capable of holding back water up to a maximum of 4 feet above existing conditions (to

approximately 255 feet NAVD88) should be the largest considered for further evaluations. Under typical operation, such a structure would likely be set to retain an elevation of 3 feet of water above the existing riprap baffle structure elevation (to approximately 254 feet NAVD88). Building the structure with another foot of excess capacity would provide managers with the flexibility to temporarily retain additional water to compensate for periods of low streamflow and depressed groundwater elevations. And of course, the structure should be capable of retaining water at levels less than 3 feet above existing conditions such that adaptive management of the structure could be practiced based on results observed for various settings.

Such a structure will significantly increase water table elevations throughout a large area of the GCS while posing minimal risk of negative impact to subsurface infrastructure present in the Heights neighborhood.

Next Steps for Field Data Collection, Modeling, and Other Analyses

Next steps for the project should include preliminary design of a structure similar to the one described above, permitting, and public outreach to assess support from abutters and other affected parties. Preliminary design should include additional survey and research regarding other potentially low-lying infrastructure such as stormwater conduits and septic systems in the vicinity of Wolomolopoag Street and neighborhoods off Chase Drive, which were not assessed during this current effort.

Should a flow control structure be installed, an active monitoring program should be implemented to provide early alert to any potential negative impacts to sensitive receptors such as septic systems. Such a program would likely involve regular monitoring of water levels in wells between the ditch and the Heights to observe the changes in groundwater elevation in this area. Monitoring of impoundment depth in the ditch itself would aid in understanding the relationship between impounded water and surrounding groundwater elevations.

<u>Modeling Limitations</u>

Great care was taken to develop the model described in this memorandum in a manner to address the objectives of the modeling effort. Available data from numerous sources was included to accurately reflect the physical characteristics of the area. Field-measured water table values, adjusted based on regional temporal context, served as the baseline to calibrate the model. Conceptual representations of proposed conditions were carefully considered by the modeling team to adequately represent the potential impacts of those conditions. Despite these best efforts, no model can reflect the real world with complete accuracy.

As with any modeling process, additional efforts could be made to yield a more robust result. Additional field research could be undertaken to better characterize key parameters of the model. Aquifer testing could yield more accurate hydraulic conductivity parameters for key materials in the area, such as the swamp deposits which comprise the GCS or the deposits which underlie the Heights. Data could be collected on local aquifer recharge rates throughout these areas as well. A sensitivity analysis, where model parameters are systematically adjusted

through a range of possible values, could also be performed to understand the potential range of error involved in the model.

The modeling effort described in this memorandum represented a steady-state condition, without regard to seasonal variability or the fluctuation in water levels impounded by a flow control structure. Modeling included no analysis of meteorological patterns, runoff patterns, or streamflow hydraulic properties which might cause seasonal variation in streamflow.

Despite these limitations, the modeling discussed here effectively demonstrates the anticipated aquifer responses from simulated flow control structures relative to existing conditions and each other.

While not necessary to advance potential design and permitting of a Town-selected flow control structure, further modeling efforts could include transient modeling, where model parameters vary through time, to better characterize dynamics of the system. Transient modeling may provide further insight into system dynamics such as the rate at which groundwater levels will rise after water is impounded behind a flow control structure and the rate at which groundwater levels will fall when the flow control structure elevation is lowered. Additionally, transient simulations could further develop understanding of the impacts of varying impoundment depths behind the structure due to fluctuations in surface runoff and baseflow to the drainage ditch.

Attached Figures:

Figure 1: Model domain and context; model boundary conditions
Figure 2: Model top layer physical characteristics including conductivity zones, recharge zones, and elevation contours.
Figure 3: Model calibration result targets
Figure 4: Surrounding infrastructure points with the Heights neighborhood
Figure 5: Conceptual diagram of flow-control structure modeling approach (in text)
Figure 6a-e: Mounding contour results from scenarios representing 1-foot (6a), 2-foot (6b), 3-foot (6c), 4-foot (6d), and 5-foot (6e) flow control structures above the existing upstream rip rap baffle.

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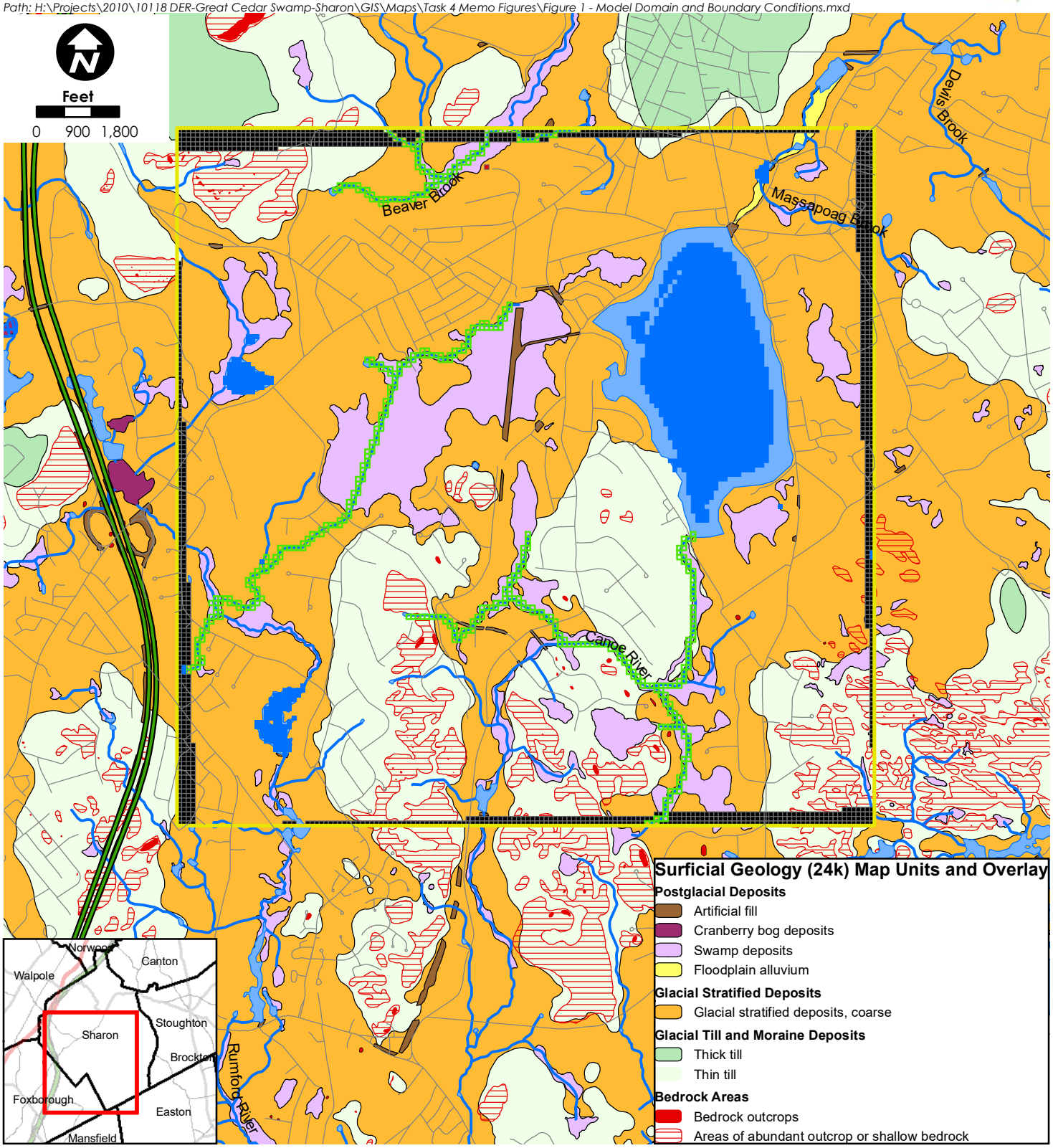
Path: H:\Projects\2010\10118 DER-Great Cedar Swamp-Sharon\GIS\Maps\Task 4 Memo Figures\Figure 1 - Model Domain and Boundary Conditions.mxd



Date: 11/6/2023
Data Sources: Bureau of Geographic Information (MassGIS), ESRI

This map is for informational purposes and may not be suitable for legal, engineering, or surveying purposes.

- Model Domain
- Constant Head Boundaries
- No Flow Boundaries
- Stream Boundaries
- Pumping Well Boundaries

MassDOT Roads
Administrative Type
- Interstate
- U.S. Highway
- Non-numbered route

NHD Water Features
- Lake/Pond
- Flowline

Great Cedar Swamp
Sharon, MA

Figure 1
Model domain and boundary conditions

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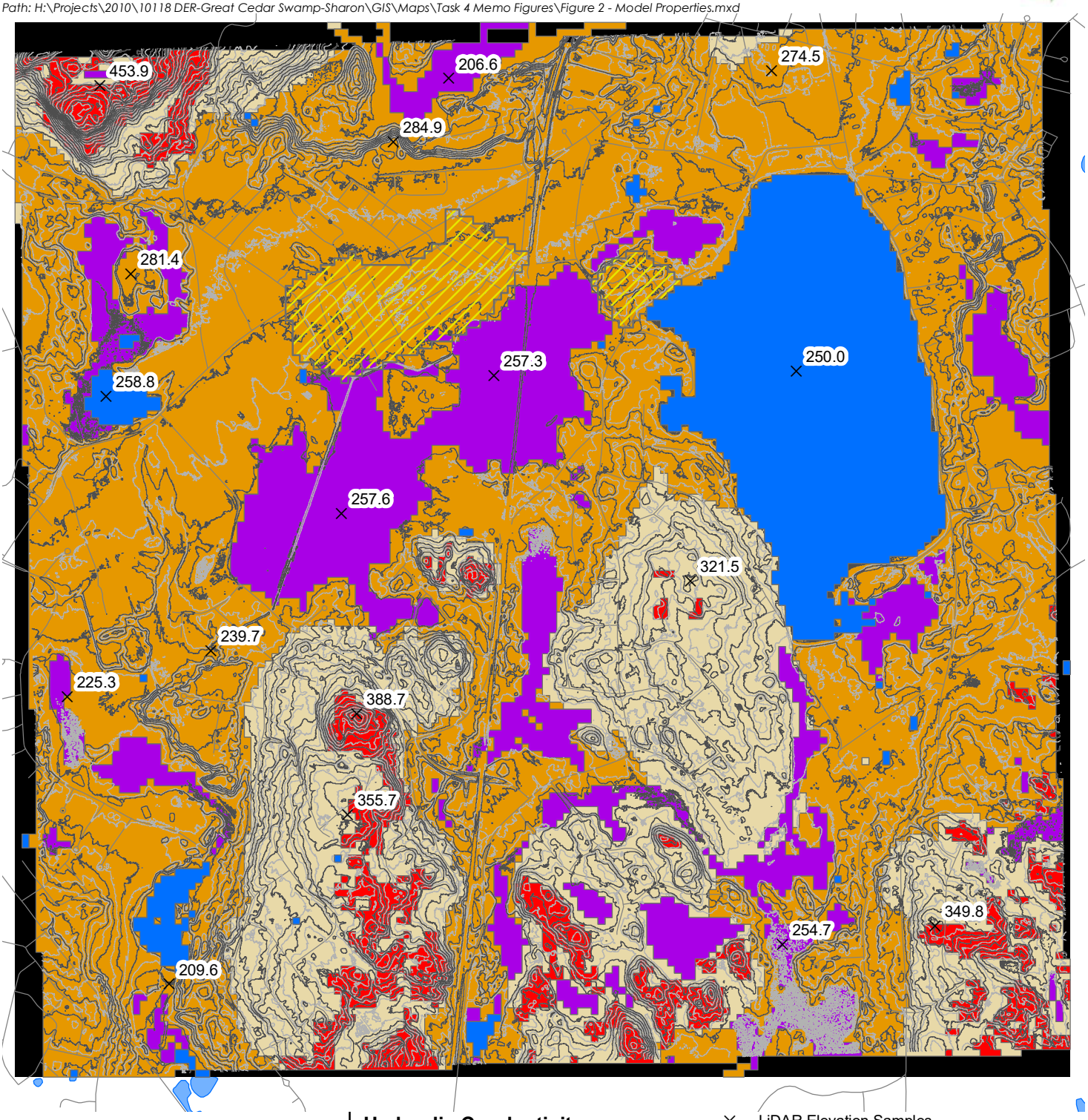
Path: H:\Projects\2010\10118 DER-Great Cedar Swamp-Sharon\GIS\Maps\Task 4 Memo Figures\Figure 2 - Model Properties.mxd



Hydraulic Conductivity

Horzontal/Vertical Conductivity (feet/day), [recharge (inches/year)]

- Bedrock Outcrop 8/8, [27.6]
- Till 10.5/8, [27.6]
- Swamp Deposits 30/10, [15.3]
- Outwash Deposits 60/20 [27.6]
- Water 50,000/50,000, [no recharge]
- Elevated Recharge (61.4 in/yr)

- LiDAR Elevation Samples
- 10-foot elevation contours
- 5-foot elevation contours

MassDOT Roads

Administrative Type

- Non-numbered route

NHD Water Feature

- Lake/Pond

Feet
0 900 1,800

Date: 11/7/2023
Data Sources: Bureau of Geographic Information (MassGIS), ESRI

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Great Cedar Swamp
Sharon, MA

Figure 2
Model properties

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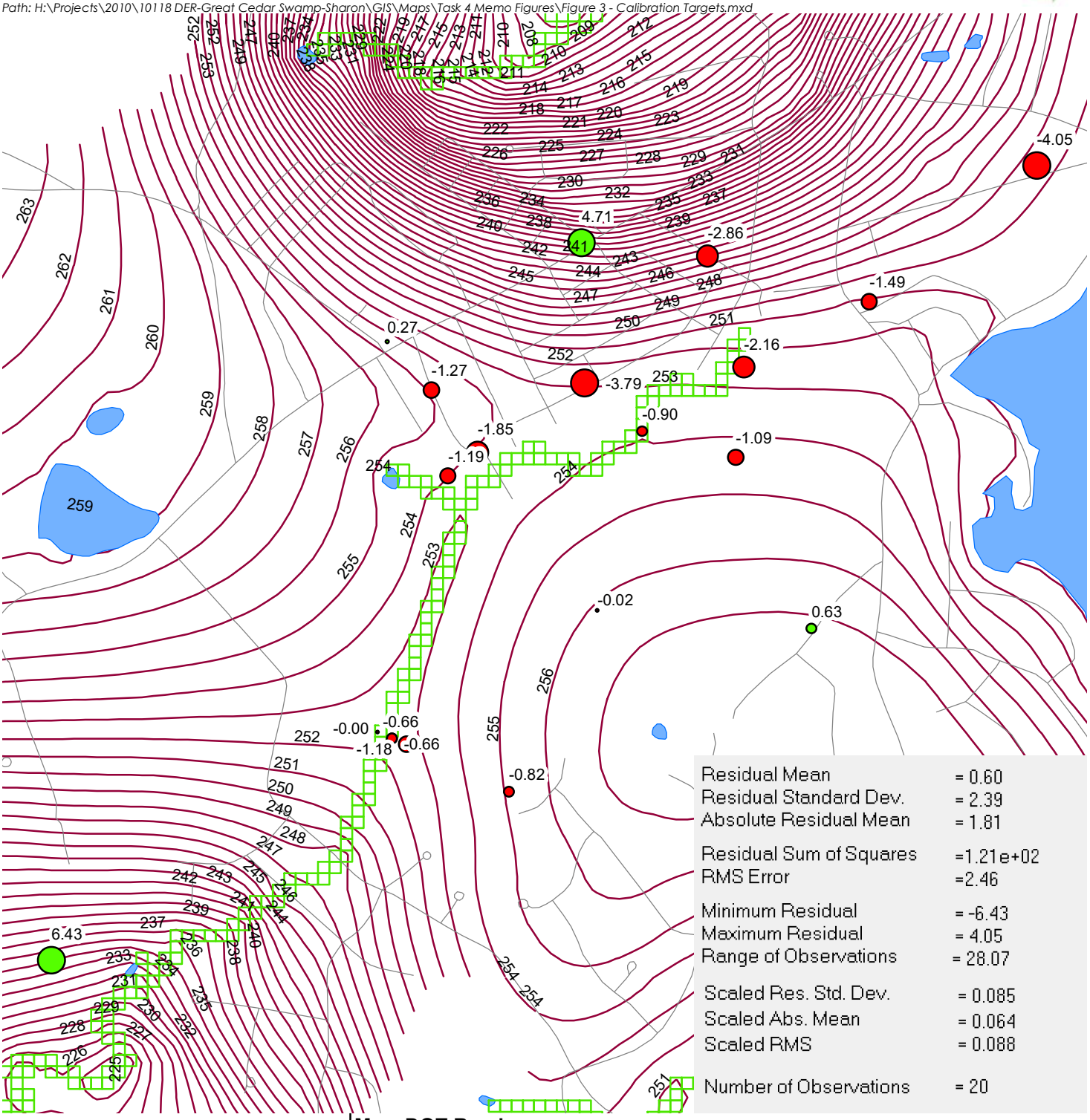
Path: H:\Projects\2010\10118 DER-Great Cedar Swamp-Sharon\GIS\Maps\Task 4 Memo Figures\Figure 3 - Calibration Targets.mxd



Date: 11/7/2023
Data Sources: Bureau of Geographic Information (MassGIS), ESRI

This map is for informational purposes and may not be suitable for legal, engineering, or surveying purposes.

MassDOT Roads

Administrative Type

Non-numbered route

NHD Water Features

Lake/Pond

Baseline Model Head Countours

Stream Boundaries

Negative residuals
(Model values below measured values)

Positive Residuals
(Model values above measured values)

N

Feet
0 450 900

Great Cedar Swamp
Sharon, MA

Figure 3
Model calibration target results

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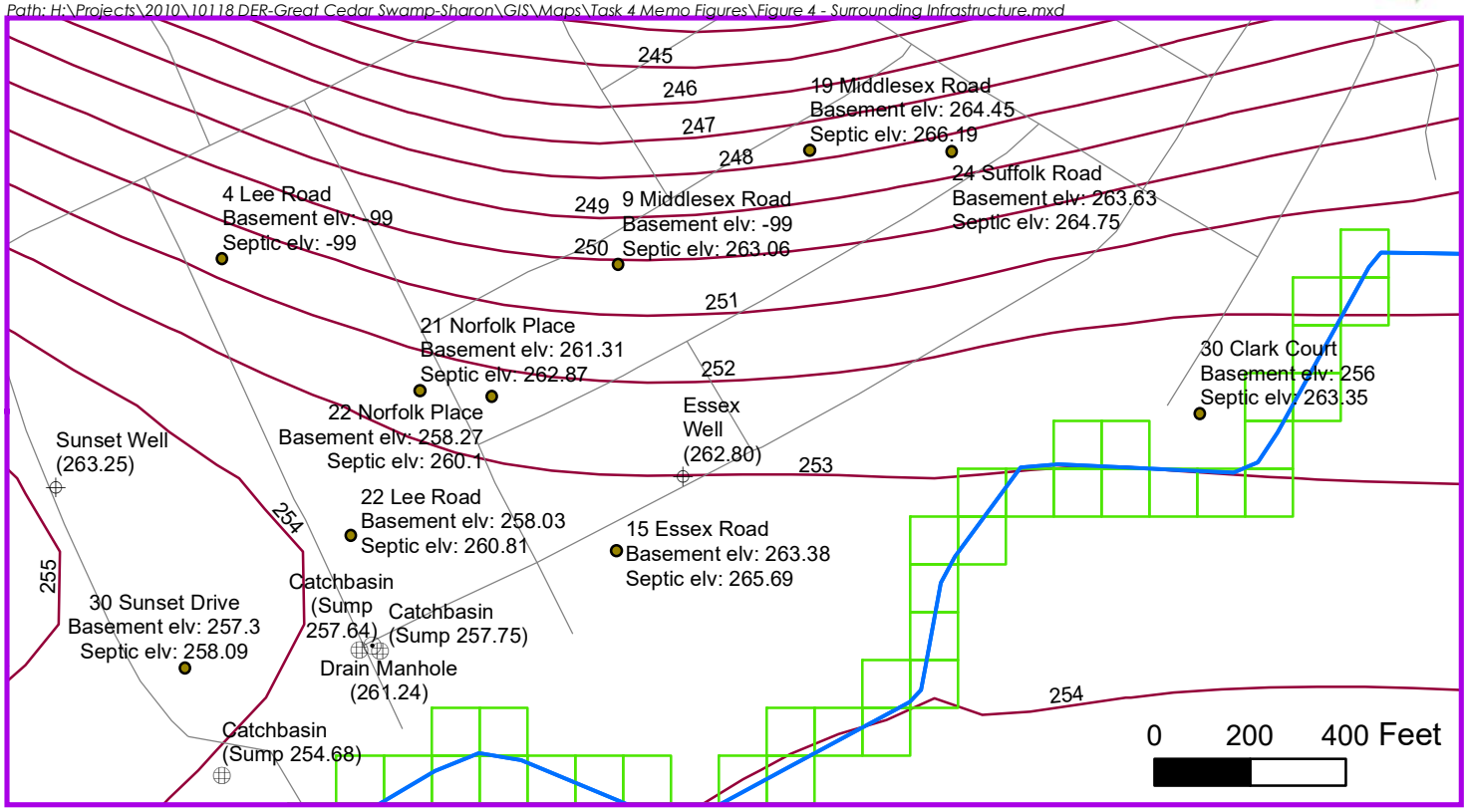
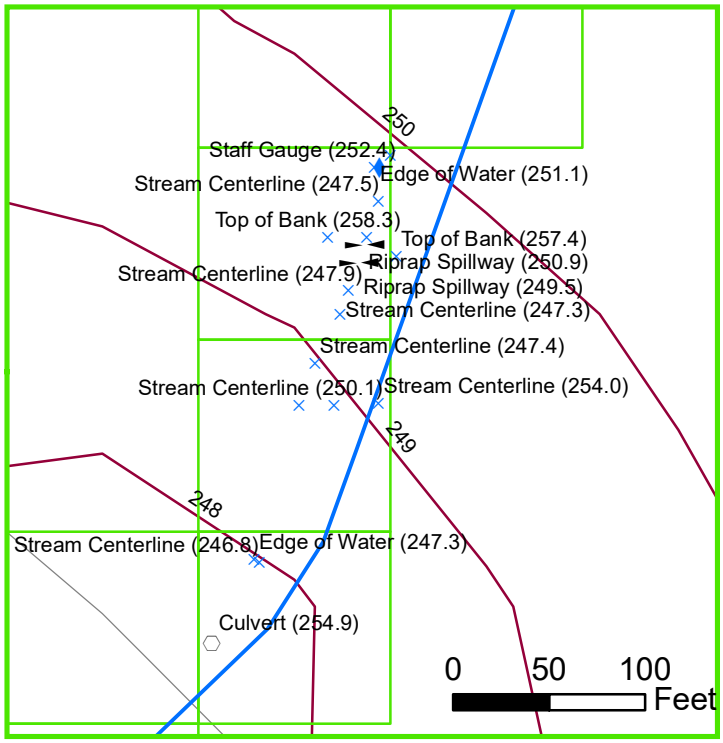
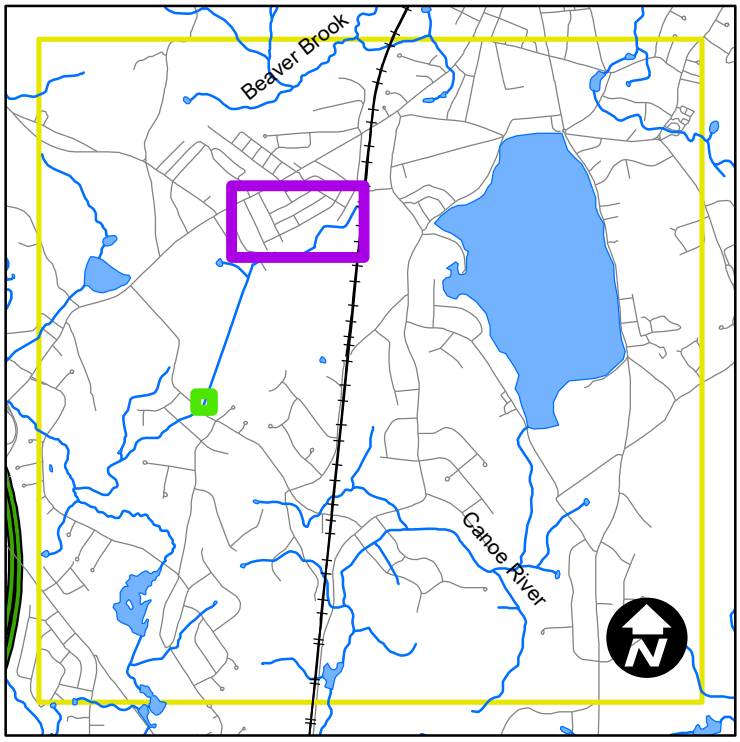
Path: H:\Projects\2010\10118 DER-Great Cedar Swamp-Sharon\GIS\Maps\Task 4 Memo Figures\Figure 4 - Surrounding Infrastructure.mxd







Date: 11/16/2023
Data Sources: Bureau of Geographic Information (MassGIS), ESRI

This map is for informational purposes and may not be suitable for legal, engineering, or surveying purposes.

Model Domain
Stream Boundaries
Baseline Model Head Countours
NHD Flowline
Approximate septic location and elevation from health department records

RTK Survey Points
Topographic Features
Catchbasin
Culvert
Drain Manhole
Essex Well
Sunset Well
Riprap Spillway
Staff Gauge

MassDOT Roads
Administrative Type
Interstate
U.S. Highway
State Route
Non-numbered route

Great Cedar Swamp
Sharon, MA

Figure 4
RTK survey data and estimated local infrastructure

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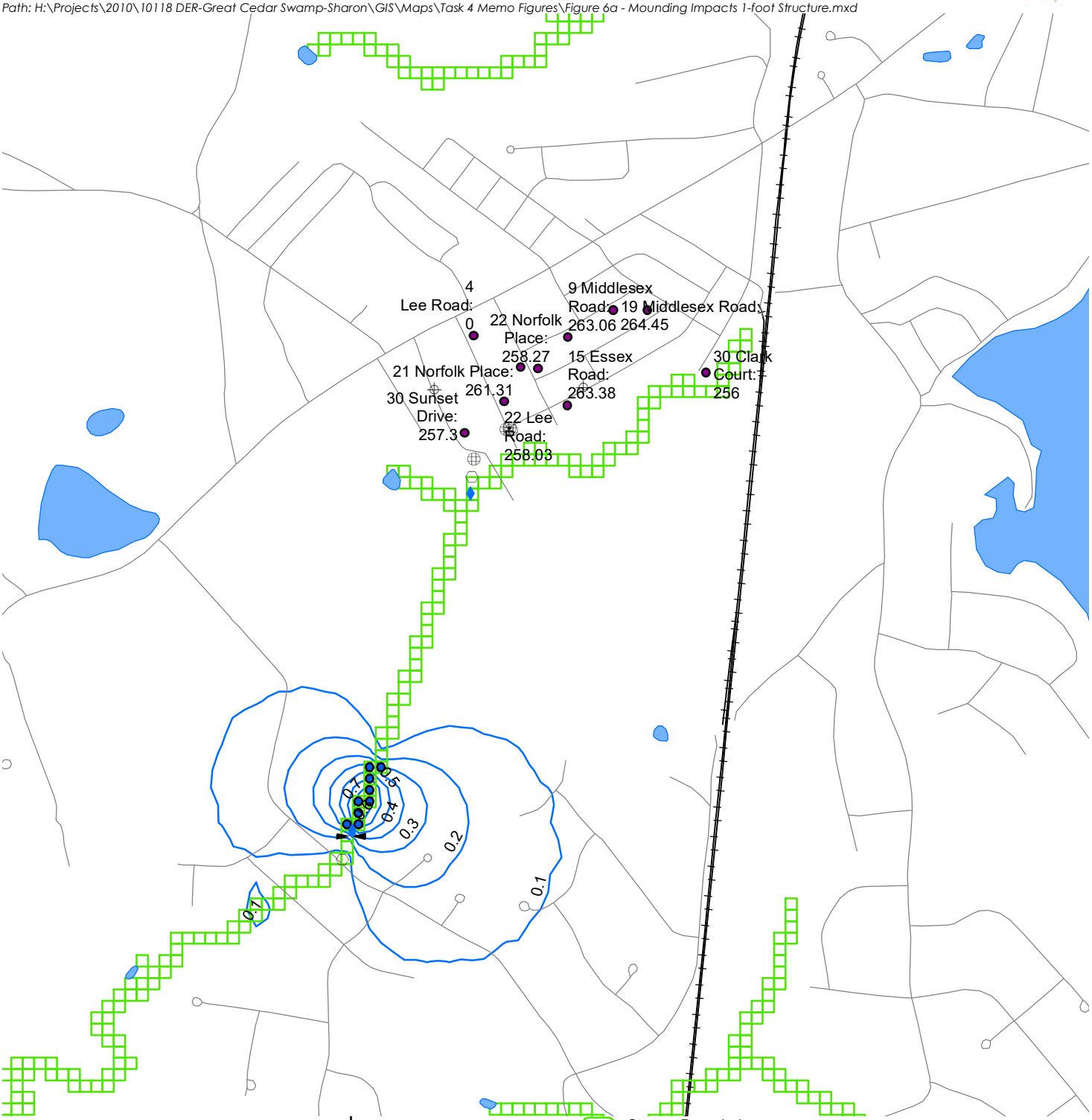
Path: H:\Projects\2010\10118 DER-Great Cedar Swamp-Sharon\GIS\Maps\Task 4 Memo Figures\Figure 6a - Mounding Impacts 1-foot Structure.mxd



Date: 11/27/2023
Data Sources: Bureau of Geographic Information (MassGIS), ESRI

This map is for informational purposes and may not be suitable for legal, engineering, or surveying purposes.

Note: Groundwater modeling represents steady-state (e.g. long term average) conditions.

RTK Survey Points

Label

- Catchbasin
- Culvert
- Drain Manhole
- Essex Well
- Sunset Well
- Riprap Spillway
- Staff Gauge

- Stream Boundaries
- Stream cells with stage affected by structure (251.9)
- Septics_approx
- Mounding contours (height above baseline)

NHD Water Features

- Lake/Pond

MassDOT Roads

- Non-numbered route
- Active Rail Service

N

Feet

0 450 900

Great Cedar Swamp
Sharon, MA

Figure 6a
Groundwater mounding impacts of a 1-foot flow control structure

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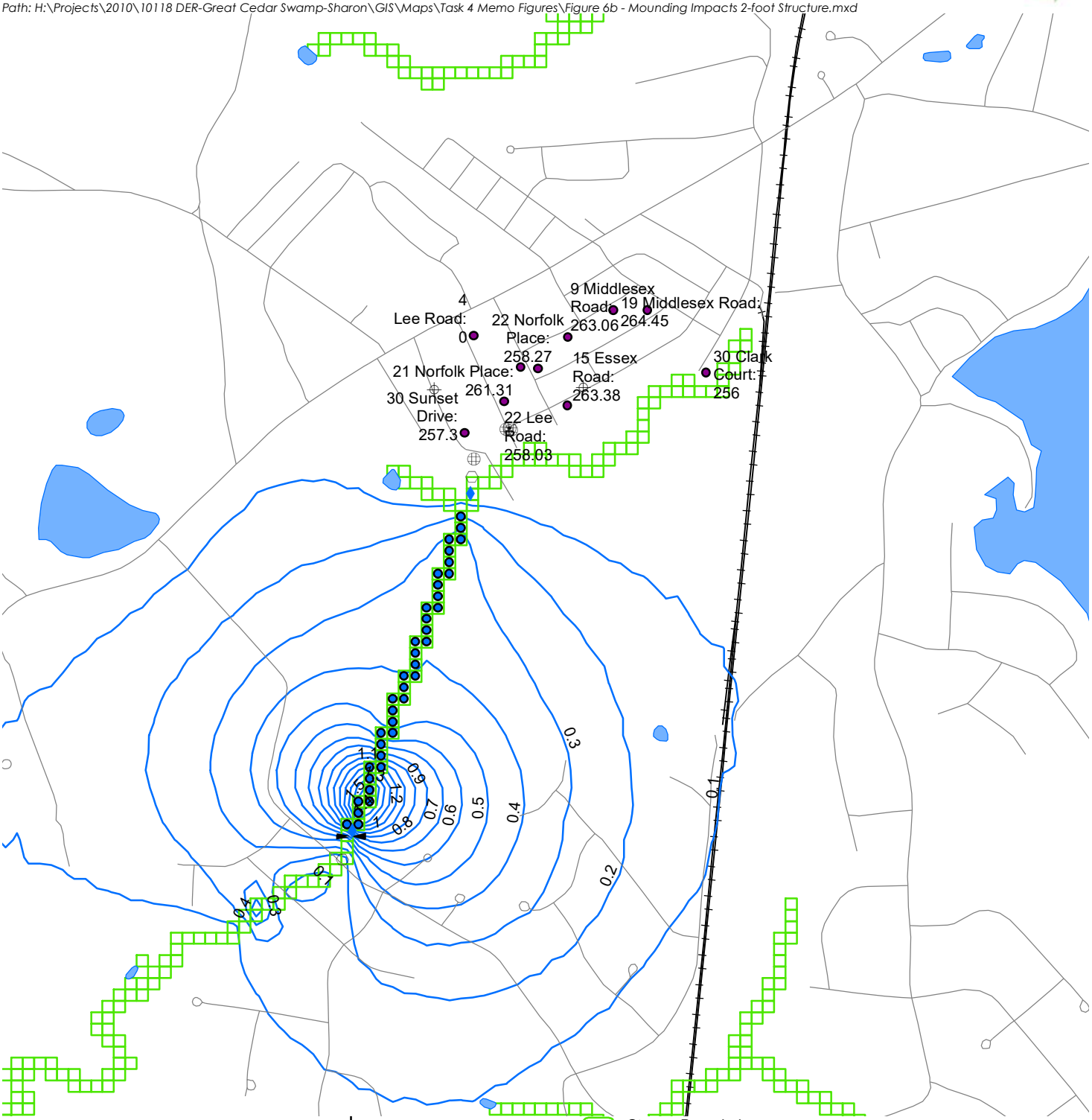
Path: H:\Projects\2010\10118 DER-Great Cedar Swamp-Sharon\GIS\Maps\Task 4 Memo Figures\Figure 6b - Mounding Impacts 2-foot Structure.mxd



Date: 11/27/2023
Data Sources: Bureau of Geographic Information (MassGIS), ESRI

This map is for informational purposes and may not be suitable for legal, engineering, or surveying purposes.

Note: Groundwater modeling represents steady-state (e.g. long term average) conditions.

RTK Survey Points

Label

- Catchbasin
- Culvert
- Drain Manhole
- Essex Well
- Sunset Well
- Riprap Spillway
- Staff Gauge

- Stream Boundaries
- Stream cells with stage affected by structure (252.9)
- Septics_approx
- Mounding contours (height above baseline)

NHD Water Features

- Lake/Pond

MassDOT Roads

- Non-numbered route
- Active Rail Service

N

Feet

0 450 900

Great Cedar Swamp
Sharon, MA

Figure 6b
Groundwater mounding impacts of a 2-foot flow control structure

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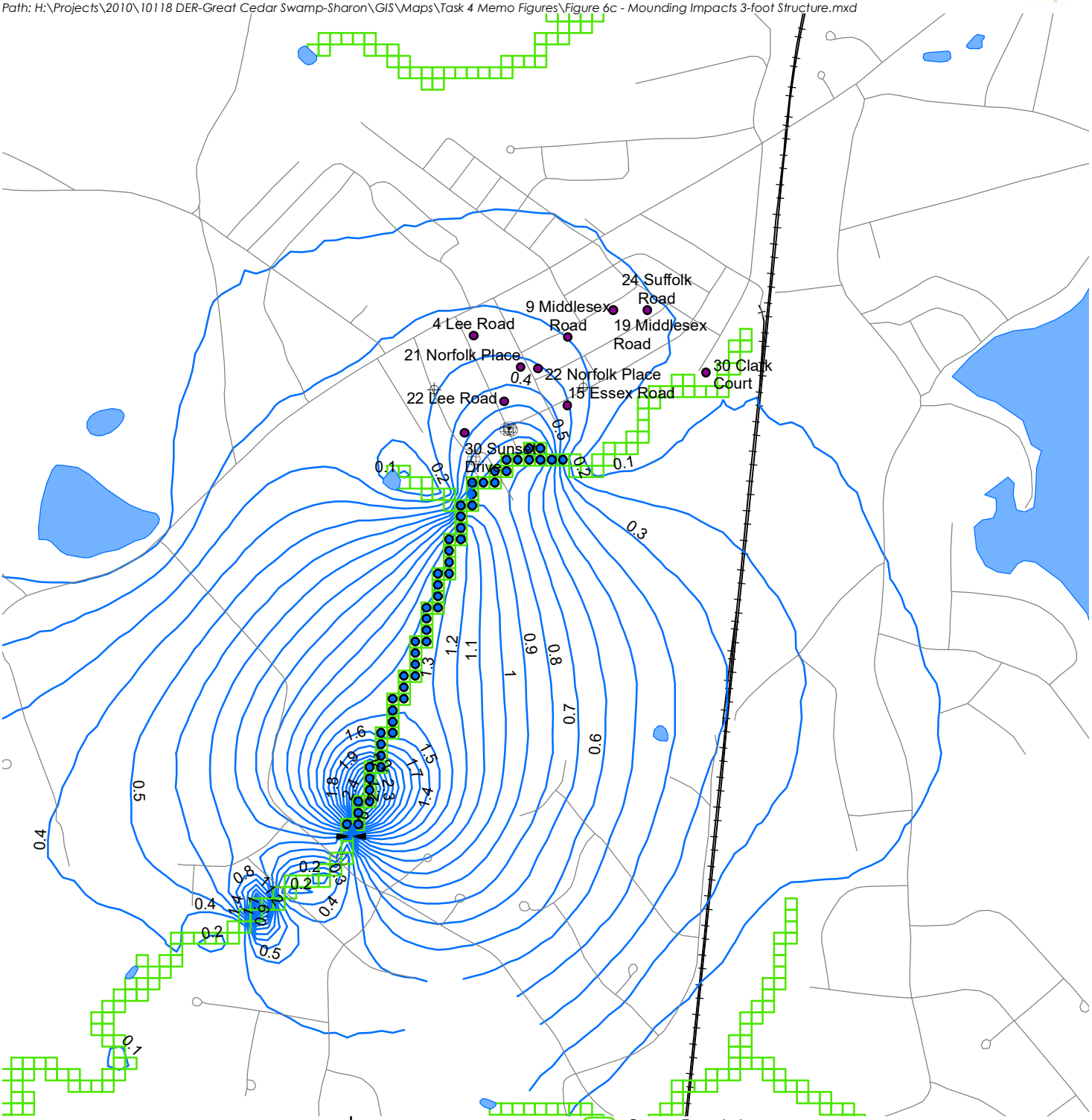
Path: H:\Projects\2010\10118 DER-Great Cedar Swamp-Sharon\GIS\Maps\Task 4 Memo Figures\Figure 6c - Mounding Impacts 3-foot Structure.mxd



Date: 11/27/2023
Data Sources: Bureau of Geographic Information (MassGIS), ESRI

This map is for informational purposes and may not be suitable for legal, engineering, or surveying purposes.

Note: Groundwater modeling represents steady-state (e.g. long term average) conditions.

RTK Survey Points

Label

- Catchbasin
- Culvert
- Drain Manhole
- Essex Well
- Sunset Well
- Riprap Spillway
- Staff Gauge

- Stream Boundaries
- Elevated Stream Stage Cells
- Septics_approx
- Mounding contours (height above baseline)

NHD Water Features

- Lake/Pond

MassDOT Roads

- Non-numbered route
- Active Rail Service

N

Feet
0 450 900

Great Cedar Swamp
Sharon, MA

Figure 6c
Groundwater mounding impacts of a 3-foot flow control structure

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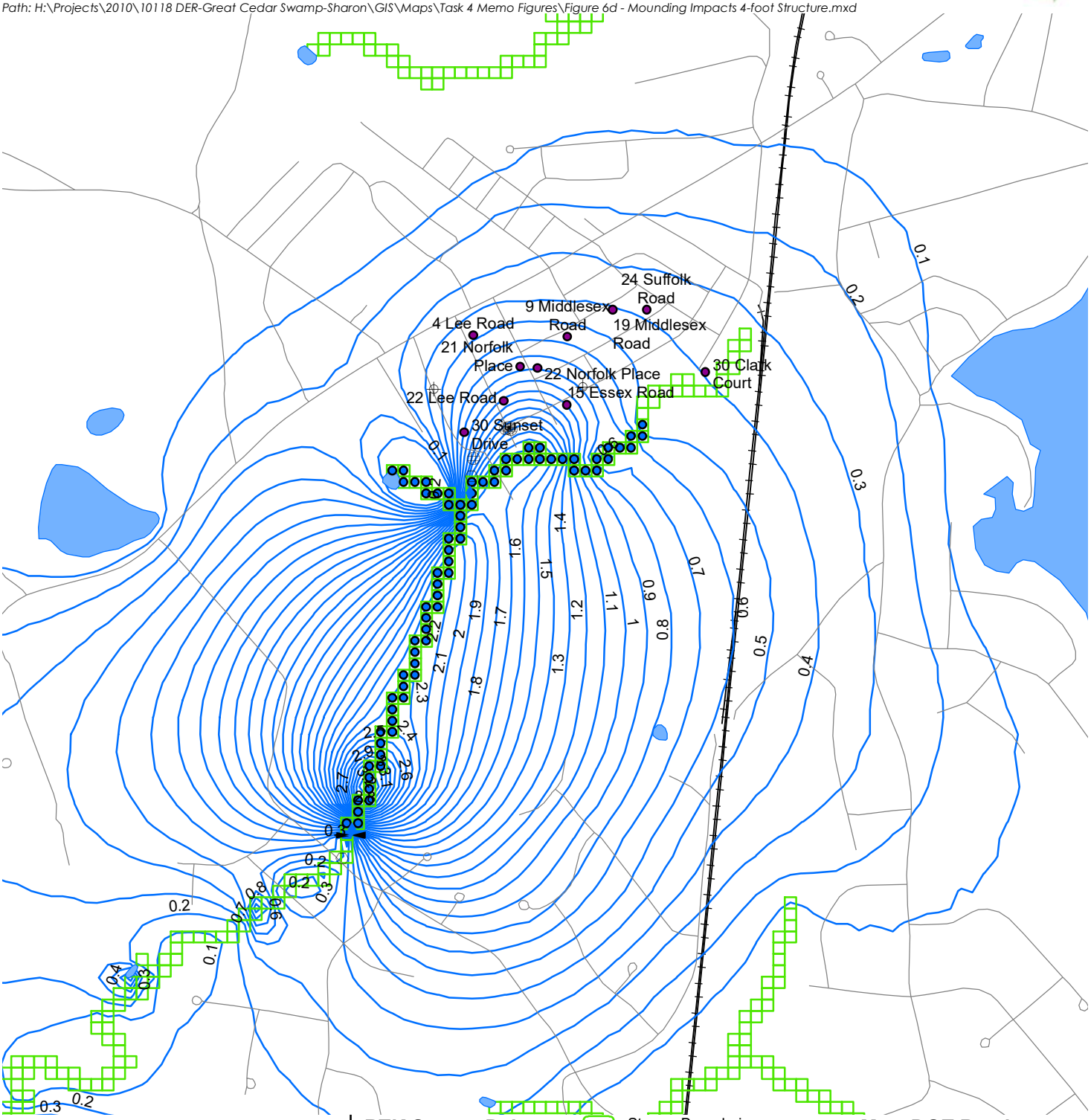
Path: H:\Projects\2010\10118 DER-Great Cedar Swamp-Sharon\GIS\Maps\Task 4 Memo Figures\Figure 6d - Mounding Impacts 4-foot Structure.mxd



Date: 11/13/2023
Data Sources: Bureau of Geographic Information (MassGIS), ESRI

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Note: Groundwater modeling represents steady-state (e.g. long term average) conditions.

RTK Survey Points

Label

- Catchbasin
- Culvert
- Drain Manhole
- Essex Well
- Sunset Well
- Riprap Spillway
- Staff Gauge

- Stream Boundaries
- Stream cells with stage affected by structure (254.9)
- Septics_approx

NHD Water Features

- Lake/Pond
- Mounding contours (height above baseline)

MassDOT Roads

- Non-numbered route
- Active Rail Service

N

Feet
0 450 900

Great Cedar Swamp
Sharon, MA

Figure 6d
Groundwater mounding impacts of a 4-foot flow control structure

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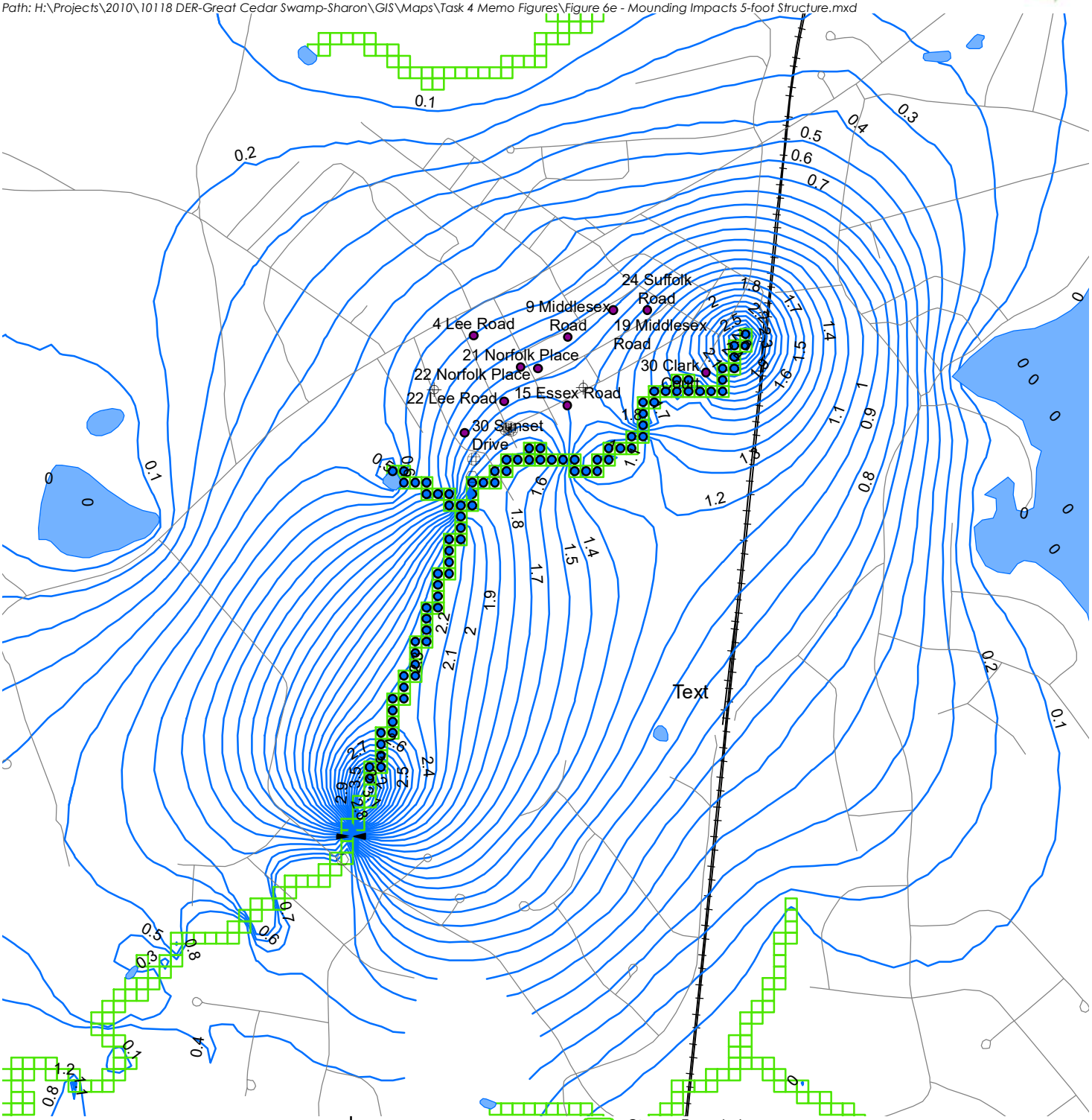
Path: H:\Projects\2010\10118 DER-Great Cedar Swamp-Sharon\GIS\Maps\Task 4 Memo Figures\Figure 6e - Mounding Impacts 5-foot Structure.mxd



Date: 11/27/2023
Data Sources: Bureau of Geographic Information (MassGIS), ESRI

This map is for informational purposes and may not be suitable for legal, engineering, or surveying purposes.

Note: Groundwater modeling represents steady-state (e.g. long term average) conditions.

RTK Survey Points

Label

- Catchbasin
- Culvert
- Drain Manhole
- Essex Well
- Sunset Well
- Riprap Spillway
- Staff Gauge

- Stream Boundaries
- Stream cells with stage affected by structure (255.9)
- Septic Systems with Available Data

NHD Water Features

- Lake/Pond
- Mounding contours (height above baseline)

MassDOT Roads

- Non-numbered route
- Active Rail Service

N

Feet
0 450 900

Great Cedar Swamp
Sharon, MA

Figure 6e
Groundwater mounding impacts of a 5-foot flow control structure

Appendix D: Field Data Technical Memorandum (June, 2023)

Horsley Witten Group
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MEMORANDUM

To: Eric Ford, Massachusetts Division of Ecological Restoration

From: Neal Price

Date: June 30, 2023

Re: Field Work Summary Technical Memorandum – Sharon Great Cedar Swamp

cc: Josh Philibert, Town of Sharon; Kimberly Groff, Southeast New England Program

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The Horsley Witten Group, Inc. (HW) is pleased to submit this memorandum summarizing field work completed this past winter and spring at the Great Cedar Swamp (GCS) in Sharon, Massachusetts (the Site). Field work was conducted to support ongoing hydrologic and hydraulic (H&H) evaluations of the GCS area and was primarily conducted by HW, with assistance from the Town of Sharon (the Town) and the Massachusetts Division of Ecological Restoration (DER).

As described more fully below, the completed field work includes:

- Collection of water level data manually and using automated water level data loggers;
- Four rounds of streamflow measurements obtained from the GCS drainage ditch; and
- Survey of all monitoring locations and key drainage infrastructure.

Monitoring locations consist of monitoring wells (Essex, Sunset Drive, B1W, B1E, A1W, A2W, A1E, and A3E), staff gauges (SG-1, SG-2, SG-3, and SG-4), and streamflow stations (SF1 and SF2). See Figure 1 for a map of all monitoring locations. All figures for this memorandum are attached.

Background

The GCS consists of 250± acres of land within the Town of Sharon in Norfolk County, Massachusetts. The site is owned and managed by the Town and consists of a large wetland complex that includes an Atlantic White Cedar (*Chamaecyparis thyoides*) swamp – a globally rare community type. The GCS is an important natural resource to the Town due to the important open space and habitat protection services it offers. In addition, because the GCS is located at relatively high elevation in the local aquifer system and is undeveloped, it provides clean groundwater recharge that contributes to most of the Town's public water supply wells and also feeds Lake Massapoag, a popular recreational resource.

The GCS is currently impaired by a drainage ditch that lies along its western edge. This ditch was created during the construction of an adjacent residential subdivision ("The Heights") in order to lower groundwater levels beneath the Heights sufficiently to allow for the construction of septic systems, basements, catch basins, and other subsurface infrastructure. The ditch also

had the effect of draining water away from, and reducing groundwater levels beneath the GCS. The ditch has lowered groundwater levels beneath a large portion of the GCS sufficiently to cause die-off of the wetland vegetation and replacement with more upland facultative species. The decline in groundwater levels has led to a noticeable amount of soil subsidence within the GCS.

The goal of the field work conducted was to collect additional hydrology data that will support efforts to create and calibrate an existing conditions groundwater model for the area. This model will be used in future phases of work to evaluate the potential groundwater impacts to the Sharon GCS and adjacent Heights neighborhood from the potential installation and use of a water control structure in the drainage ditch.

Collection of Water Level Data

Water level data in and around the GCS was collected from monitoring wells and the drainage ditch itself both manually and with the use of automated water level loggers. A map detailing the monitoring locations is attached here as Figure 1. Manual measurements were taken by HW staff four times on a monthly basis between January 31st 2023 and April 11th 2023. An additional round of manual measurements was taken on May 10th 2023 by the Sharon Conservation Administrator. At monitoring wells, depth-to-water (DTW) measurements were taken with a Heron Instruments Dipper-T water level meter. At staff gages these measurements were taken by measuring down with a tape measure from the top of the gage to the water surface. Automated water level loggers were set to record water level measurements every hour between the dates of installation and retrieval. Water level monitoring locations are described below and the data collected is included in Table 1. Please note that all monitoring wells are pre-existing of this study, were named by others previously, and those names are carried over here, as discussed below. Some staff gauge locations in the drainage ditch appear to be pre-existing but no prior naming conventions for those staff gauges are known to HW. All staff gauges discussed herein, both pre-existing locations and new ones established for this study, have been named by HW as SG-1 through SG-4, as discussed below.

Monitoring wells:

- Essex – two-inch monitoring well located in the front yard of 18 Essex Road. A HOBO data logger provided by DER was placed in this well on February 14th, 2023 and removed on April 11th, 2023.
- Sunset Drive – two-inch monitoring well located in the eastern sidewalk in front of 11 Sunset Drive. A logger was not placed in this well.
- B1W – two-inch monitoring well located just west of the drainage ditch in the shortly to the south of Sunset Drive. HW placed a Van Essen TD-Diver data logger in this well on January 31st, 2023. This logger was removed on April 11th, 2023.
- B1E – two-inch monitoring well located just east of the drainage ditch across from B1W. HW emplaced a Van Essen TD-Diver in this well on January 31st, 2023. This logger was removed on April 11th, 2023.
- A1W – two-inch monitoring well located west of the drainage ditch approximately a half mile south of the Heights and a quarter mile upstream from Wolomolopoag Street.
- A2W – two-inch monitoring well located a little further west of the drainage ditch in line with A1W.

- A1E – two-inch monitoring well located east of the drainage ditch in line with A1W and A2W.
- A3E – two-inch monitoring well located a little further east of the drainage ditch in line with A1E.

Staff Gages:

- Staff Gauge 1 (SG-1) – two-inch perforated PVC pipe located in the drainage ditch approximately 100 feet upstream of the riprap baffle, and 360 feet upstream from the culvert at Wolomolopoag Street. A Van Essen TD-Diver data logger was placed within SG-1 on January 31st, 2023. This logger was removed on April 11th, 2023.
- Staff Gauge 2 (SG-2) – DER RIFLS gauge, made of a wooden board connected to a two-inch metal pipe, located in the drainage ditch approximately 60 feet upstream of the culvert at Wolomolopoag Street.
- Staff Gauge 3 (SG-3) – Four-foot wooden beanpole located just upstream of the culvert at Wolomolopoag Street. A Van Essen TD-Diver data logger was secured to this staff gauge on February 14th, 2023. This logger was removed on April 11th, 2023.
- Staff Gauge 4 (SG-4) – DER RIFLS gauge located in the drainage ditch in near proximity to the Heights approximately 40 feet south of monitoring wells B1W and B1E. This gauge is a metal square pipe with large openings to allow water through the interior. A HOBO data logger provided by DER was placed here on February 14th, 2023 and removed on April 11th, 2023.

Survey of Monitoring Locations and Key Infrastructure

To supplement previous surveying efforts in the GCS, HW surveyed monitoring locations and key infrastructure using survey grade, real time kinematic (RTK) GPS equipment on February 14th, 2023. The spatial data is in reference to the North American Datum of 1983 (NAD 83), feet, and the elevation data is in reference to the North American Vertical Datum of 1988 (NAVD 88), feet. The location and elevation data for the monitoring locations, from this study and previous surveying efforts, are listed below in Table 1. The monitoring reference elevations are the top of PVC elevations at monitoring wells and the tops of the staff gauges.

Table 1. Elevation and location data for GCS monitoring locations

Monitoring Location	Northing (ft)	Easting (ft)	Reference Elevation (ft)
Essex	2863415	738047.5	262.35
Sunset Drive	2863394	736735.9	262.80
B1W	2862568.7	737018.1	257.40
B1E	2862534.4	737133.8	257.60
SG-4	2862487	737055.3	254.65 (top of metal)
SG-1	2859521	736016.6	252.43
SG-2	2859323	735952.8	Unavailable
SG-3	Couldn't get signal	Couldn't get signal	248.84 (top of wood)
A1W	2860372.07	736246.99	257.41
A2W	2860429.9	736111.71	257.84
A1E	2860315.94	736371.41	257.79
A3E	2860265.33	736515.39	257.50

The infrastructure surveyed included culverts on both the upstream and downstream sections of the drainage ditch, rim elevations of catchbasins and drainage manholes, centerline of the ditch and contributing side channel (tributary), the tops of the banks, and the riprap riffle structure installed by others approximately a decade ago. See Table 2 for elevation and location data for these additional survey points and Figure 1 to see the mapped locations of these points.

Table 2. Elevation and location data for additional survey points in GCS vicinity

Survey Point	Northing (ft)	Easting (ft)	Reference Elevation (ft)
Lee Road drainage manhole rim	2863061	737397.3	261.25
Lee Road catchbasin rim; 3.05' depth to invert	2863054	737370.5	260.69
Lee Road catchbasin rim; 3.2' depth to invert	2863050	737413.6	260.95
Sunset Dr. catchbasin rim; 3.3' depth to invert	2862790	737083.3	257.98
Ditch centerline	2859527	736022.1	248.14
Ditch centerline	2859504	736015.8	247.51
Ditch centerline	2859419	735982.7	247.42
Ditch centerline	2859445	735995.9	247.34
Top of ditch bank	2859485	735989.4	258.34
Top of ditch bank	2859475	736024.9	257.45
Centerline of tributary at ditch/tributary confluence	2859397	735974.5	248.03
Centerline of tributary	2859397	735992.9	250.14
Centerline of tributary	2859398	736015.8	254.02
Ditch centerline	2859316	735953.8	246.85
Edge of water	2859317	735951.1	247.33
Edge of water	2859729	736087.2	250.86
Edge of water	2859521	736013.9	251.20
Ditch centerline	2859484	736009.7	250.06
Ditch centerline	2859457	735999.9	247.91
Riprap baffle	2859471	736005.8	249.51
Riprap baffle	2859480	736008.6	250.97
Upstream headwall of Wolomolopoag Street culvert; 9.0' depth to invert (sediment bottom)	2859273	735929.1	255.00
Headwall of Sunset Drive culvert; 3.54' depth to invert (sediment bottom)	2862636	737066.3	256.45

After the initial site visit on January 31st, 2023, it was decided that additional loggers would be added to the monitoring plan and installed on the following site visit. DER personnel sent three HOBO brand data loggers to HW for installation at Essex well, SG-4, and a barometric location. HW provided an additional logger for installation at SG-2. However, during the subsequent site visit on February 14th, HW staff determined there was not enough water at SG-2 for proper logger installation and data collection. Therefore, this additional logger was moved a little further downstream from SG-2 to a new locationSG-3, just upstream from Wolomolopoag Street where the water was deeper. On February 14th HW staff were unable to locate monitoring wells A1W, A2W, A1E, and A3E for manual measurements. Sometime between the March 16th and April 11th site visits, the A3E well casing above the ground surface was broken off, so the DTW measurement on April 11th was taken from the top of the remaining casing and, therefore, no elevation can be provided for that April 11th water level reading. The top of remaining casing will need to be resurveyed in order to use this well to collect water level elevation information in the future.

Water Elevations

After each site visit the DTW measurements were used in conjunction with the survey data to determine the groundwater elevation at each monitoring location (Table 3). The survey data was also used in conjunction with the logger data to determine water elevations through time. A graph of water elevations from the automated water level loggers over the project monitoring period (January 31 to April 11, 2023) is attached (Figure 2). Precipitation data from National Weather Service (NWS) station Walpole 2 COOP is plotted on the secondary y-axis of Figure 2. This weather station is located approximately 4.6 miles away from the western border of the Great Cedar GCS.

Table 3. Water level measurements (feet) at GCS monitoring locations

Monitoring Location	January 31st, 2023		February 14th, 2023		March 16th, 2023		April 11th, 2023		May 10th, 2023	
	DTW	Elev.	DTW	Elev.	DTW	Elev.	DTW	Elev.	DTW	Elev.
Essex	-	-	4.36	257.99	3.13	259.22	4.12	258.23	4.80	257.55
Sunset Drive	-	-	7.83	254.97	6.94	255.86	4.46	258.34	7.45	255.35
B1W	2.41	254.99	2.82	254.58	1.49	255.91	2.82	254.58	2.81	254.59
B1E	2.85	254.75	3.30	254.30	2.40	255.20	3.29	254.31	3.21	254.39
SG-4	-	-	1.52	253.13	-	-	1.65	253.00	-	-
SG-1	1.34	251.09	1.60	250.83	1.29	251.14	1.65	250.78	1.47	250.96
SG-2	2.35	-	2.50	-	1.19	-	2.51	-	1.51	-
SG-3	-	-	0.71	248.13	0.49	248.35	0.82	248.02	-	-
A1W	4.58	252.83	-	-	2.20	255.21	5.23	252.18	5.06	252.35
A2W	3.58	253.26	-	-	1.95	255.89	3.35 *	-	-	-
A1E	2.63	255.16	-	-	2.58	255.21	3.14	254.65	3.12	254.67
A3E	1.65	255.85	-	-	1.53	255.97	1.98	255.52	2.00	255.50

*April 11th, 2023 measurement from top of broken casing, not surveyed top of well

The following are preliminary observations regarding the collected water level data:

- The first three weeks of the monitoring period were quite dry and, despite the mid-wintertime of year with no significant evapotranspiration occurring, this time period is marked by a gradual and consistent decrease in water elevations for all monitored groundwater and surface water stations. This response can be thought of as the gradual drainage of water from aquifer storage, primarily out through the drainage ditch.
- The next two plus weeks from late February through early March were characterized by frequent smaller precipitation events and two larger ones. Water levels increased at all stations in response to the increased precipitation. Interestingly, monitoring wells B1E and B1W demonstrated greater responses to precipitation events than the other monitoring stations (B1W particularly), including the surface water stations in the ditch where one might normally expect to see more rapid precipitation responses. Wells B1E and B1W are located in short distance on either side of the ditch near the stormwater outfall at Sunset Drive. One possibility is that these wells, particularly B1W closest to the stormwater outfall and which only sticks up several inches above the ground surface,

becomes inundated by stormwater runoff during larger rain events such that the well tops are flooded over and the pressure response in the wells than becomes abnormally high. B1W is located in a depression and separated from the ditch by a berm such that it is possible for stormwater runoff to pond here prior to discharge to the ditch.

It would be instructive to look at the surface water response at the same time and location to help explain the observed precipitation response at the B1 series wells. SG-4 is the ditch surface water station at approximately that same location. Unfortunately, the SG-4 water elevation record has several time gaps to it associated with negative recorded pressure values, generally indicative that the logger was out of the water. It does not make sense that the logger could have been out of the water during this wet period unless the logger itself moved. The SG-4 logger was originally placed six to eight inches below the water surface inside a pre-existing hollow metal post with large holes in it. Woody debris had to be cleared from within the post during logger installation. One possibility is that entry of more woody debris into the post floated the logger upwards, breaking the surface of the water. So, unfortunately, the SG-4 record is not helpful for explaining the observed response at the B1 series wells for late February. However, further downstream in the ditch, precipitation responses at the SG-3 and SG-1 staff gauges are much less than at the B1 series wells suggesting that the observed responses at the B1 series wells are localized in nature. Negative pressure values at SG-4 are also observed at other time periods, not all associated with rain events.

- Following a week of dry weather, the most significant precipitation event of the monitoring period occurred between March 14th and March 15th, with 2.94 inches of rain over those two days. The water level response pattern to this large event played out similarly to that described above for late March. However, in this case the SG-4 data record is complete throughout this time period. Once again well B1W showed a sudden spike of water levels approximately double that of any other monitoring location. The anomalously high spike in water levels compared to all other groundwater and surface water stations reinforces the theory stated above that the precipitation response at B1W is likely due to its location receiving direct and localized influence from the Sunset Drive stormwater outfall.
- Following these rapid precipitation responses discussed above, all monitoring stations exhibit fairly rapid declines in water levels, with surface stations declining more rapidly than groundwater wells, as that direct surface water runoff influence is drained out through the ditch. During extended dry periods of several days or more, the decline in water levels is even more gradual as the slower process of ditch drainage of aquifer storage occurs following the runoff drainage period.

Streamflow Measurements

HW recorded streamflow depth and velocity measurements across transects at two locations in the drainage ditch on four separate occasions. Using these measurements HW computed the discharge for these locations at the monitoring times. These discharges, in cubic feet per second (cfs) are listed in Table 4 and are attached in a plot (Figure 3). Descriptions of the streamflow monitoring locations are below:

- SF1 is located approximately 150 feet upstream of the culvert at Wolomolopoag Street, and downstream of a tributary feeding into the ditch from the east.
- SF2 is located approximately 80 feet upstream of SF1, upstream from the tributary feeding in from the east, and roughly 30 feet downstream of the riprap baffle within the ditch.

Discharge from the tributary is estimated by subtracting SF2 from SF1. Due to the limited distance between these two monitoring locations, it is assumed that the majority of the difference in flow between SF1 and SF2 is due to the addition of water from the tributary, rather than a significant additional groundwater influence.

Table 4. Calculated discharges within the drainage ditch

Date	SF1 Flow (cfs)	SF2 Flow (cfs)	Tributary Flow Contribution (cfs)	Percentage of SF1 Flow from Tributary Contribution
1/31/23	4.08	3.06	1.02	25%
2/14/23	1.34	0.95	0.39	29%
3/16/23	7.53	6.35	1.18	16%
4/11/23	0.93	0.75	0.18	19%

During the February 14th and April 11th measurement events, the tributary appeared dry. However, there were still significant differences in flow between the two streamflow measurement locations, likely attributable to additional groundwater inputs between the two stations and measurement accuracy limitations. Therefore the percentage of tributary flow contribution shown above in Table 4 are likely a significant overestimate, with some 10-20% of the flow difference between the two stations derived from additional groundwater inflow, rather than from the tributary.

Stage-Discharge Relationships

Stage-discharge relationships were developed for both streamflow monitoring locations SF1 and SF2. Once established, stage-discharge relationships are used to estimate the discharge of a river based on stage measurements alone. In this case these stage-discharge relationships can be used to convert the continuous water level logger data to continuous estimates of discharge for the logger monitored period. The plots displaying the SF1 and SF2 stage-discharge relationships are attached (Figures 4 and 5, respectively). These relationships are based on the manual streamflow and stage data collected over the four sampling rounds of this study. In this case, four measurements are far less than would be ideal for accurate estimation of streamflow over a wide range of flow conditions. However, as a planning tool and source of information for groundwater model calibration, these rating curves are still useful despite their limitations. The

Town of Sharon may wish to conduct additional flow monitoring beyond the four rounds included in this memorandum to better inform more robust stage-discharge relationships in the future.

Figures 4 and 5 depict the graphical stage-discharge relationship between the SG-3 water level monitoring location and the SF1 flow monitoring station and between the SG-1 water level monitoring location and the SF2 flow monitoring station, respectively. The figures also display mathematical expressions describing each relationship, along with the correlation coefficients for each relationship. The mathematical expressions are used to convert records of data logger water elevations to estimates of streamflow over the same time period of record. The correlation coefficient is a measure of how well the expression fits the data, with a value of 1.0 indicating a perfect fit and 0.0 indicating no fit. The relationships at SF1 and SF2 exhibit excellent correlation coefficients of 0.97 and 0.96 respectively, admittedly with only four paired data points used.

The stage-discharge estimated streamflow discharge values at SF1 over the monitoring period are shown on Figure 6. The stage-discharge estimated streamflow discharge values at SF2 over the monitoring period are shown on Figure 7. Precipitation data from National Weather Service station Walpole 2 COOP is plotted on the secondary y-axis in both figures. As discussed above for water level responses to precipitation, streamflow responds quickly upwards from rain events in late February through mid-March, and then gradually declines as that water is conveyed out of the Great Cedar Swamp system.

Correlation of Great Cedar GCS Data to USGS Index Well Data

In order to provide longer historical context beyond the relatively short monitoring period of this study, correlations of Site water levels to a USGS index well were examined. The closest USGS index well to the Sharon GCS is MA-NNW27R in Norfolk, MA, approximately 4.5 miles from the Wolomolopoag Street culvert of the Cedar GCS drainage ditch. The water elevation data from the Norfolk index well was plotted alongside the water elevation data from the Sharon GCS (Figure 8). The water elevation trends between the two sites correlate very closely to one another in terms of timing and show similar responses to precipitation events (see Figure 2) over the monitoring period. One interesting difference in the water level records between the Index Well and the Site is the much more significant decline in water level at the Index Well relative to the Site for the dry period of early February. The reason for this difference is unknown. However, one possibility may be that the drainage ditch so effectively drains Site groundwater away, that Site groundwater levels declined more quickly (much of it potentially before the start of the monitoring period) in response to dry conditions than did Index Well groundwater levels.

Correlation of GCS Data to USGS Streamflow Data

In order to provide longer historical context beyond the relatively short monitoring period of this study, correlations of Site ditch flow to a USGS streamflow station were examined. The closest USGS streamflow station to the Sharon GCS is Station 01105500 East Branch Neponset River at Canton, MA. This streamflow station is approximately 5.2miles from the Wolomolopoag Street culvert. The flow data from this station was plotted with precipitation data on the secondary y-axis (Figure 9). As was the case for all other discussed Site and USGS water level and flow

data, the plot shows a general decrease in flow from the end of January into mid-February, with increases in flow correlating to precipitation events occurring near the end of February and into March. The most significant change in flow at the nearby Neponset River correlates to the mid-March precipitation event that resulted in 2.94 inches of precipitation being recorded at Station Walpole 2 COOP. Streamflow then generally trends downward through the end of the monitoring period.

The field flow data from the Sharon GCS drainage ditch was then plotted against the USGS Neponset River streamflow data (Figure 10). While only displaying four discrete sampling events, the drainage ditch data correlates well with the continuous USGS streamflow data in terms of timing and relative magnitude of flow variability.

Lastly, the full stage-discharge estimated flow record from the drainage ditch just above Wolomolopoag Street (SF1) was plotted against the USGS Neponset River streamflow data for the coincident period of Site monitoring (Figure 11). The drainage ditch data correlates well with the continuous USGS streamflow data in terms of timing and relative magnitude of flow variability. In general, Neponset River flows at any given time are approximately 20 times greater than those in the GCS drainage ditch.

Hydrologic Conditions During Monitoring Period Versus Long-Term Average Conditions

In order to place the hydrologic conditions observed during this project's relatively short, winter/spring 2023 monitoring period in longer term context, HW reviewed available long term water level and precipitation data. As discussed above, long term water level data was obtained from the USGS Index Well MA-NNW27R well in Norfolk and precipitation data was obtained from the Walpole 2 COOP NWS station (Station ID USC00198757). Daily water level data for the Index Well are available from August 2001 to present (22 years of data) and daily precipitation values from the NWS station are available since 1971, providing 51 years of historical precipitation and snowfall data.

Monitoring on-site occurred from January 31 through April 11,2023. Figure 12 depicts water levels at the index well during the 2023 monitoring period as well as the most recent 22 years of historical water level data from the Norfolk Index Well. As can be seen, this project's 2023 monitoring period occurred during wetter than average hydrologic conditions. Water elevations at the index well ranged from 157.0 feet to 157.9 feet during the monitoring period, with an average value of 157.3 feet; approximately 0.7 feet higher than the overall average water level for the past 22 years of 156.6 feet. The average water level during the monitoring period was in the 87th percentile of values. The highest index well water elevation during the monitoring period was in the 98th percentile of values, and the lowest was in the 74th percentile of values.

Statistics describing historic monthly total precipitation values and values recorded during the monitoring period are described in Table 5 below. February was significantly drier than the average February, receiving only 1.32 inches of rain compared to a year-over-year average of 3.41 inches. March was much wetter than February and wetter than the average March, with 5.18 inches of rain compared to an average March total of 4.26 inches. The two months preceding the monitoring period were also wetter than average with both December and

January exceeding long term average monthly values by 1.03 and 2.6 inches, respectively. Taken together, these four months averaged 4.75 inches per month – 0.70 inches per month higher than the average of the same three months in the preceding 50 years. The beginning of April started off quite dry with only 0.53 inches of rain falling during the monitoring period. Total rainfall during April (including after monitoring) was below average for April historically, and the majority of the rain that fell in April, 2023, fell after monitoring ended.

Table 5. Precipitation statistics for Walpole 2 COOP Weather Station and values during monitoring period

	December	January	February[M]	March[M]	April
Minimum Monthly Total	0.86 (1980)	0.77 (1981)	0.50 (1987)	0.35 (2006)	1.11 (1999)
Average Monthly Total	4.55	4.05	3.41	4.26	4.36
Maximum Monthly Total	10.49 (2020)	11.76 (1979)	8.17 (2008)	16.75 (2010)	11.75 (1987)
Standard Deviation of Monthly Totals	2.35	2.11	1.60	2.65	2.26
During Monitoring Period	5.98	6.65	1.32	5.18	0.54* 3.08**

[M]Monitoring period includes full month
*April rainfall during monitoring period (April 1st through April 11th)
**Total rainfall for the month of April

A small amount of snowfall fell during the monitoring period. A total of 1.00 inch was reported at the Walpole monitoring station during February and March, compared to an average amount of 2.3 inches for the same period in a typical year. December and January had a total snowfall of 0.93 inches, compared to an average of 1.21 inches in previous years. Some snow was present in the GCS during the first field visit on January 31st, 2023, which melted throughout the day. Light snow fell between February 22nd and 24th (total 0.29 inches), on February 26th (0.10 inches), February 28th and March 1st (0.25 inches), March 4th and 5th (0.20 inches) and March 15th (0.16 inches).

Next Steps

Using data collected during this phase of work, data collected by others in previous work, data supplied by the Town, and data available on Mass GIS, HW has created a local-scale, conceptual MODFLOW groundwater model for the area. Calibration and adjustments to the model are currently under way using the field data discussed in this memorandum. Once the model is suitably reflective of existing conditions, it will be used to evaluate the potential groundwater impacts to the Sharon GCS and adjacent Heights neighborhood from the potential installation and use of a water level control structure in the drainage ditch at the approximate location of the previously installed baffle.

Attached Figures:

Figure 1: Monitoring Locations and Surveyed Points Map
Figure 2: Sharon Great Cedar Swamp Water Elevations from Late January – Early April 2023
Figure 3: Sharon Great Cedar Swamp Drainage Ditch Streamflow from January 31st and April 11th 2023
Figure 4: Stage Discharge Relationship Between SG-3 and SF1
Figure 5: Stage-Discharge Relationship between SG-1 and SF2
Figure 6: Calculated Discharge at SF1 from SG-3 Logger Data
Figure 7: Calculated Discharge at SF2 from SG-1 Logger Data
Figure 8: Water Elevations in the Great Cedar Swamp Compared to USGS Index Well NNW27R Water Elevations in Norfolk, MA
Figure 9: Precipitation at National Weather Service Station Walpole 2 COOP, in Walpole, MA Compared to Neponset River Flow in Canton, MA
Figure 10: Sharon Great Cedar Swamp Drainage Ditch Flow Compared to USGS Neponset River in Canton, MA Streamflow
Figure 11: Calculated Sharon Great Cedar Swamp Drainage Ditch Flow at SF1 Compared to USGS Neponset River in Canton, MA Streamflow
Figure 12: Historic water elevations at Norfolk Index Well

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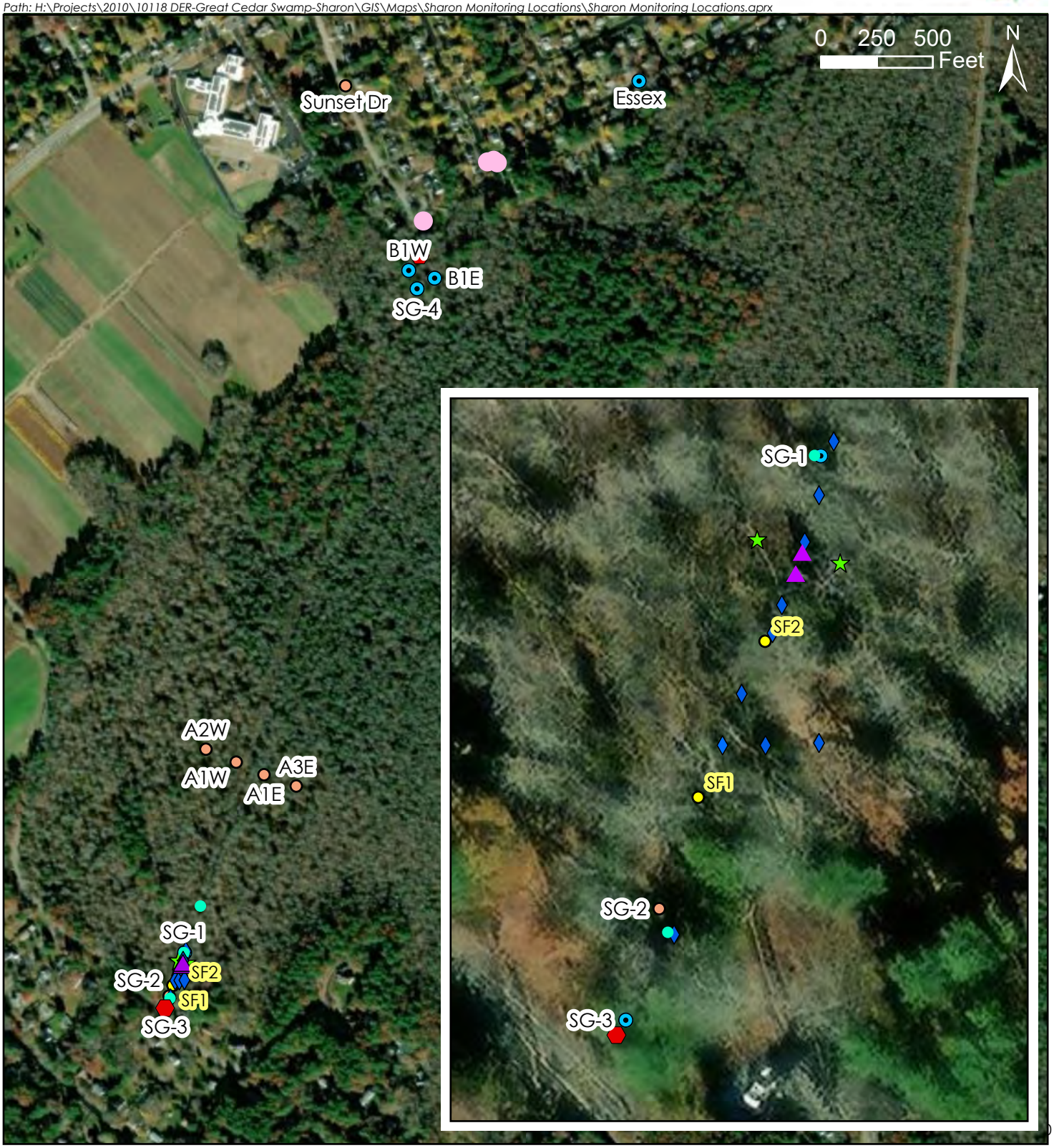
508-833-6600 • horsleywitten.com

Path: H:\Projects\2010\10118 DER-Great Cedar Swamp-Sharon\GIS\Maps\Sharon Monitoring Locations\Sharon Monitoring Locations.aprx



Date: 6/22/2023
Data Sources: Bureau of Geographic Information (MassGIS), ESRI

This map is for informational purposes and may not be suitable for legal, engineering, or surveying purposes.

- Location With Data Logger
- Location Without Data Logger
- Streamflow Locations

Survey Points

- Catchbasin/Manhole
- Ditch bank
- Ditch or Tributary Feature
- Edge of water
- Riffle Structure
- Culvert

10118F: Sharon Great Cedar Swamp
Sharon, MA

Figure 1
Monitoring Locations and Surveyed Points at Sharon Great Cedar Swamp.

Figure 2. Water Elevations From Late January - Early April 2023, Great Cedar Swamp, Sharon, MA

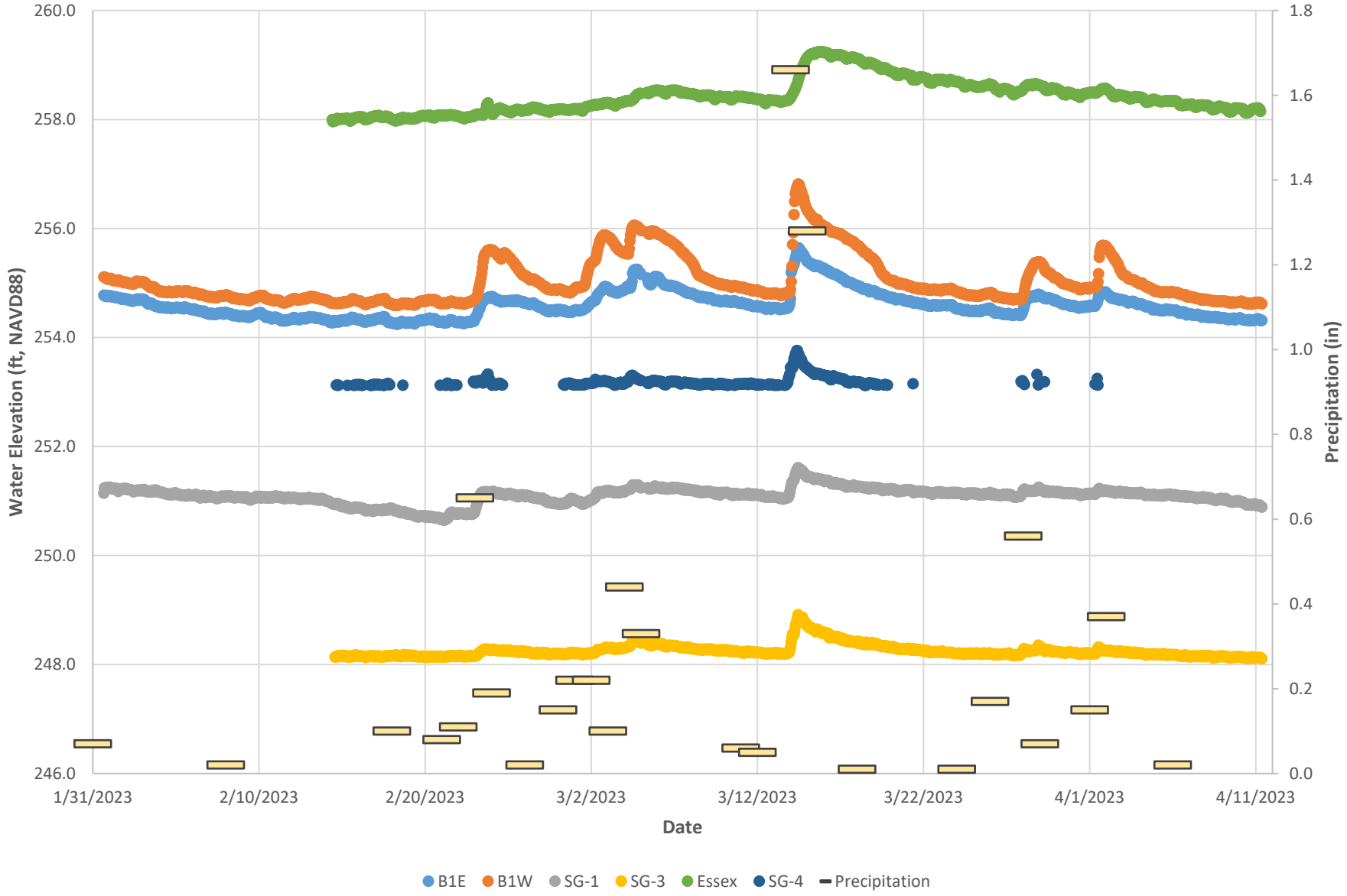


Figure 3. Sharon Great Cedar Swamp Drainage Ditch Streamflow Between January 31st, 2023 and April 11th, 2023

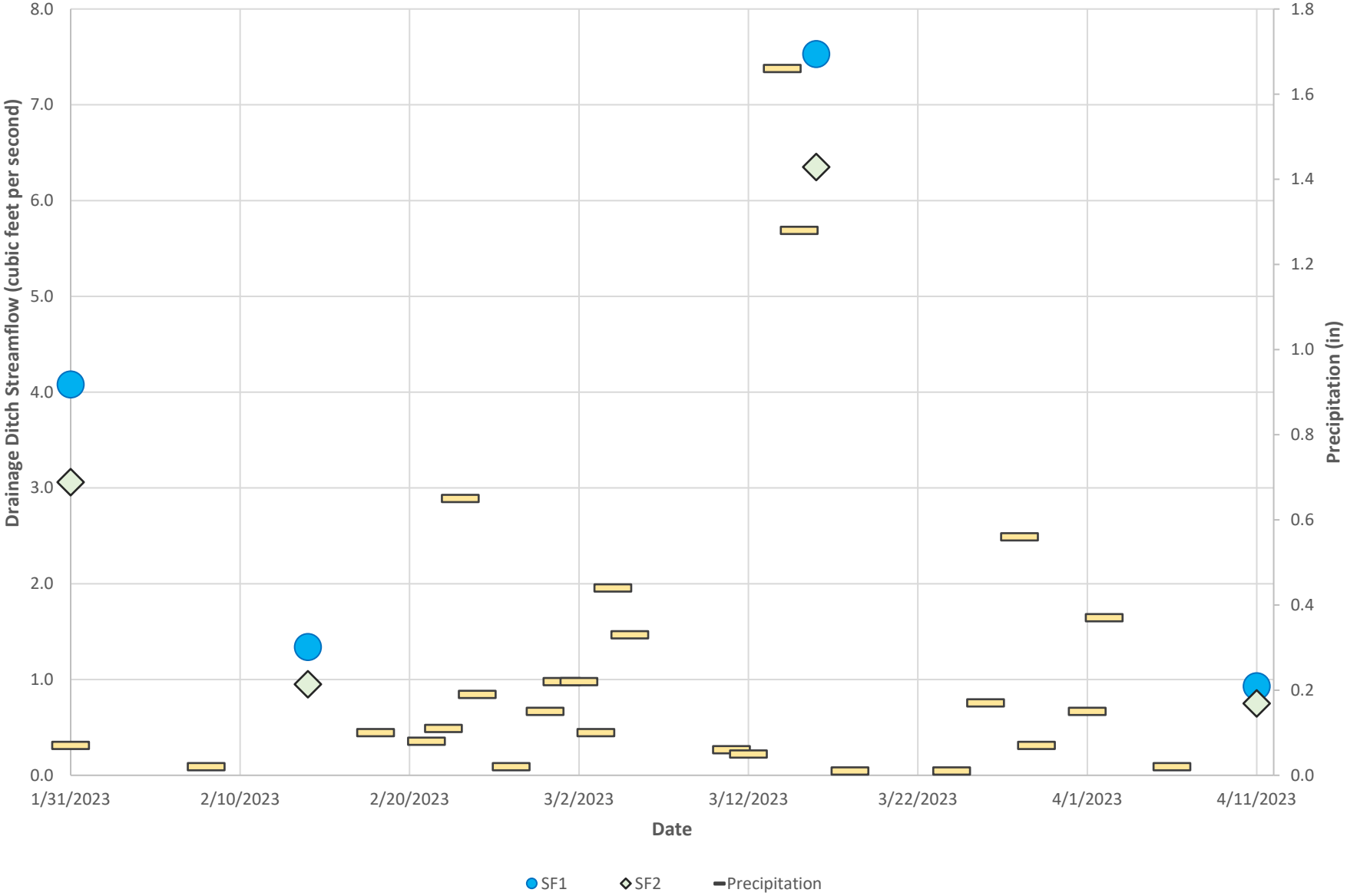


Figure 4. Stage-Discharge Relationship Between SG-3 and SF1

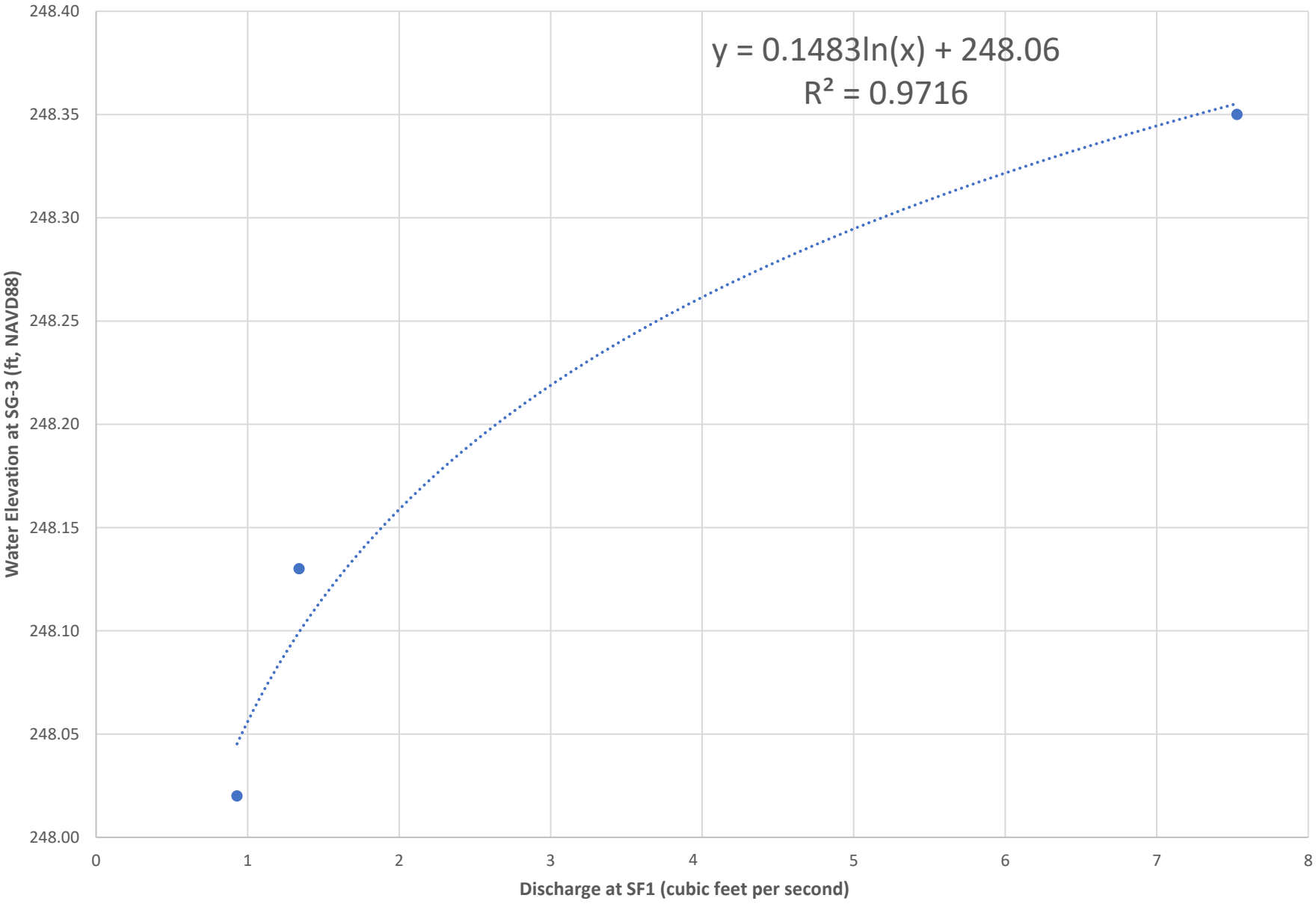


Figure 5. Stage-Discharge Relationship Between SG-1 and SF2

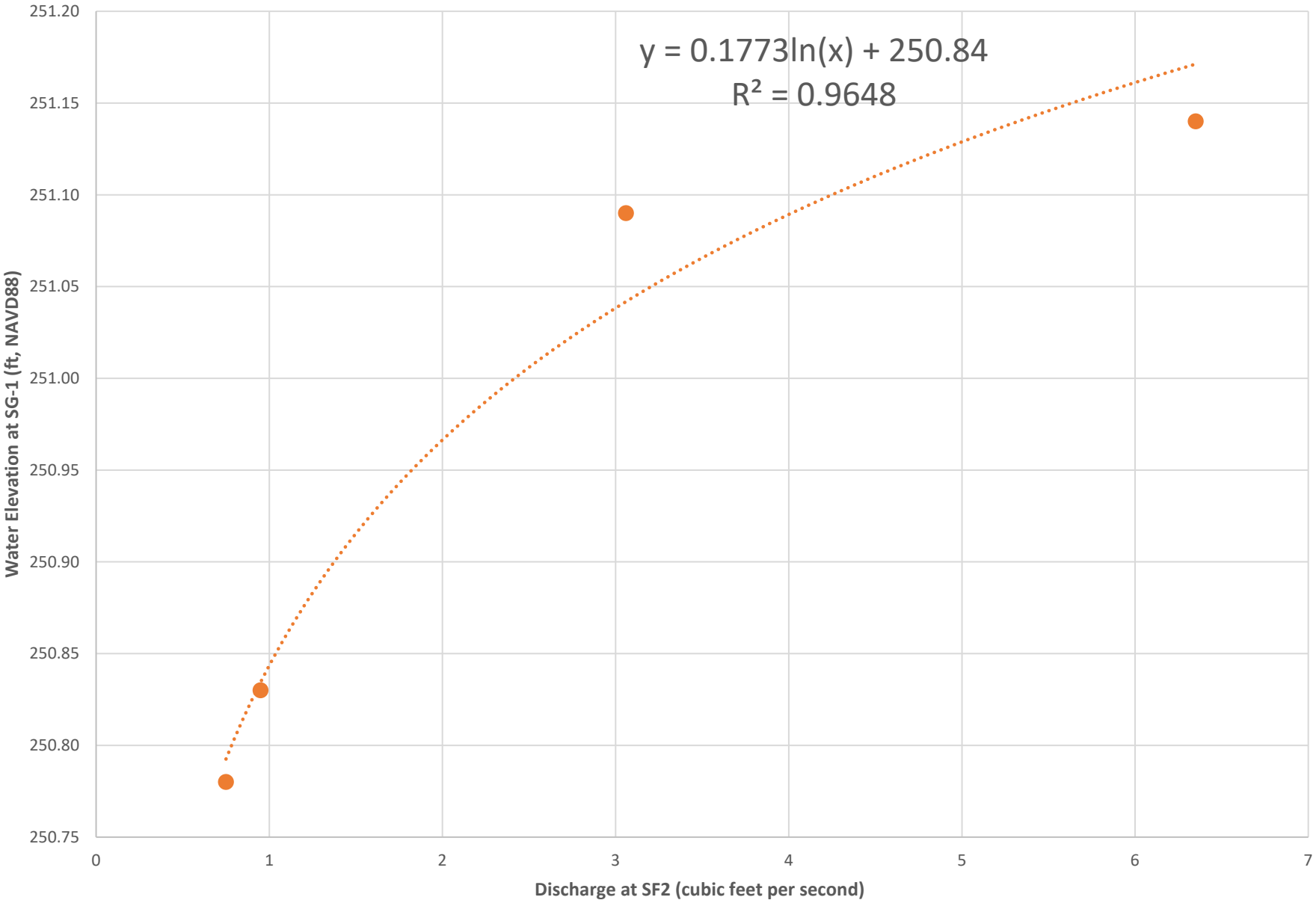


Figure 6. Calculated Discharge at SF1 From SG-3 Logger Data

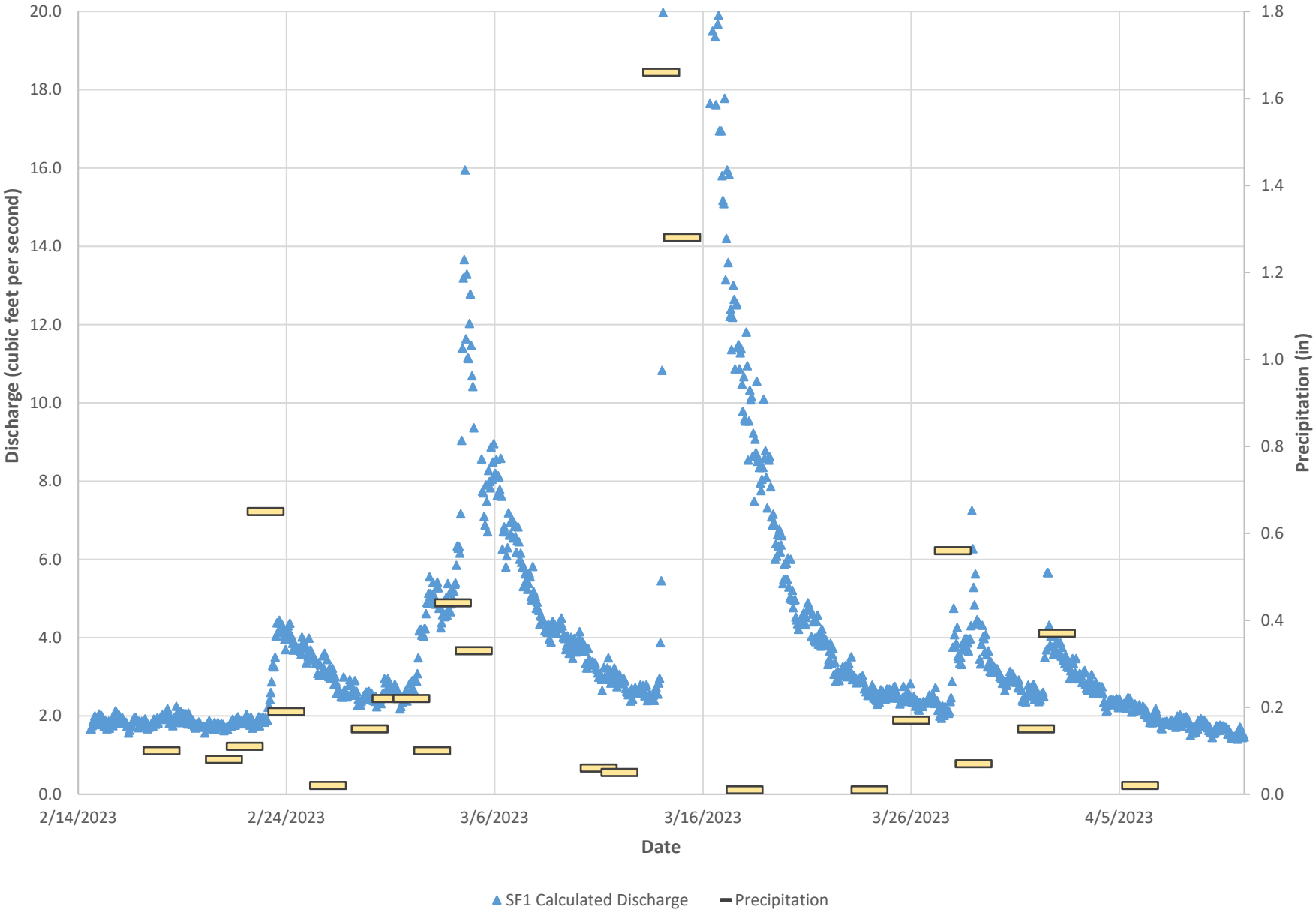


Figure 7. Calculated Discharge at SF2 From SG-1 Logger Data

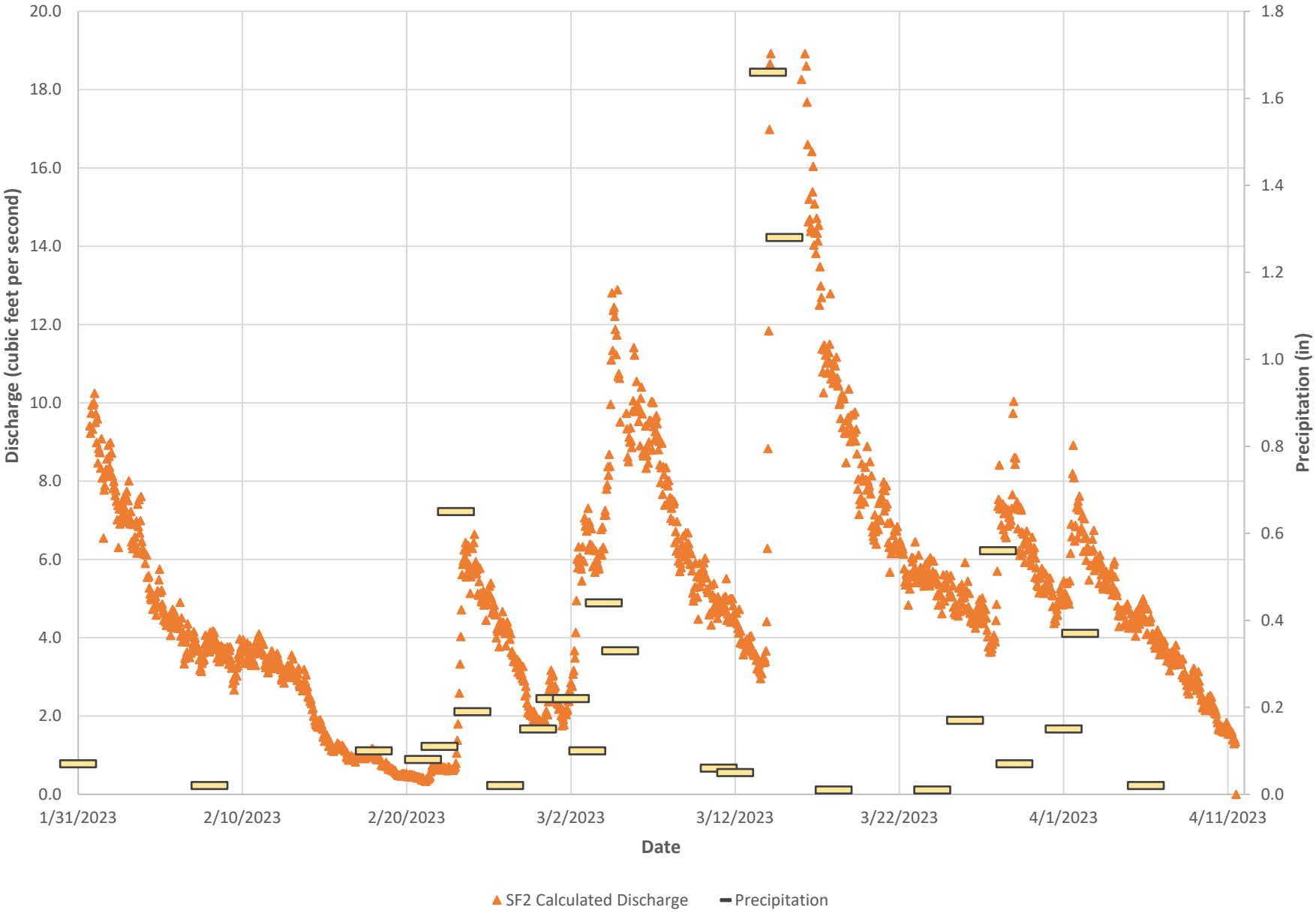


Figure 8. Water Elevations in Great Cedar Swamp Compared to USGS Index Well NNW27R Water Elevations in Norfolk, MA

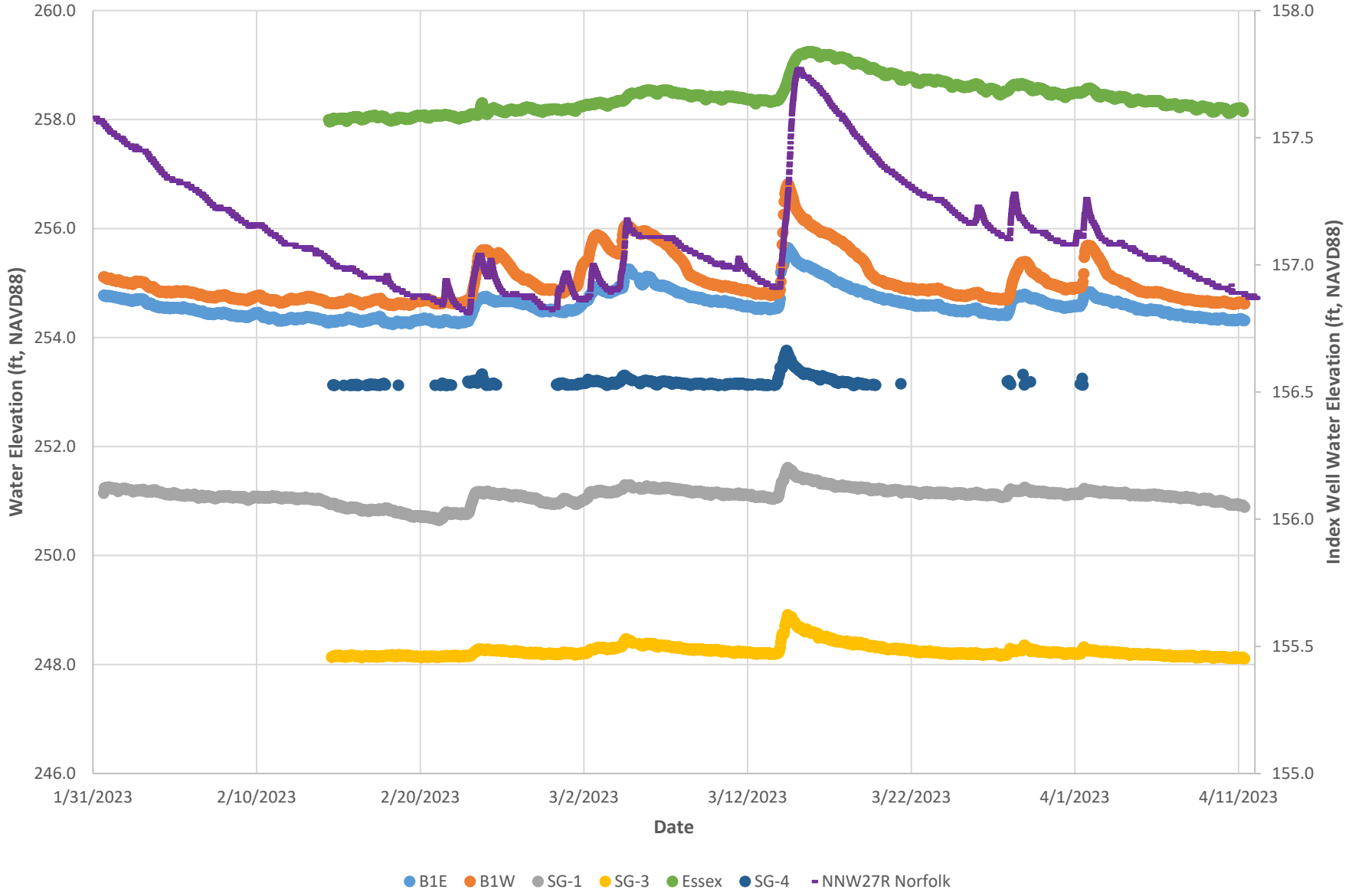


Figure 9. Precipitation at National Weather Service Station Walpole 2 COOP, in Walpole, MA Compared to Neponset River Flow in Canton, MA

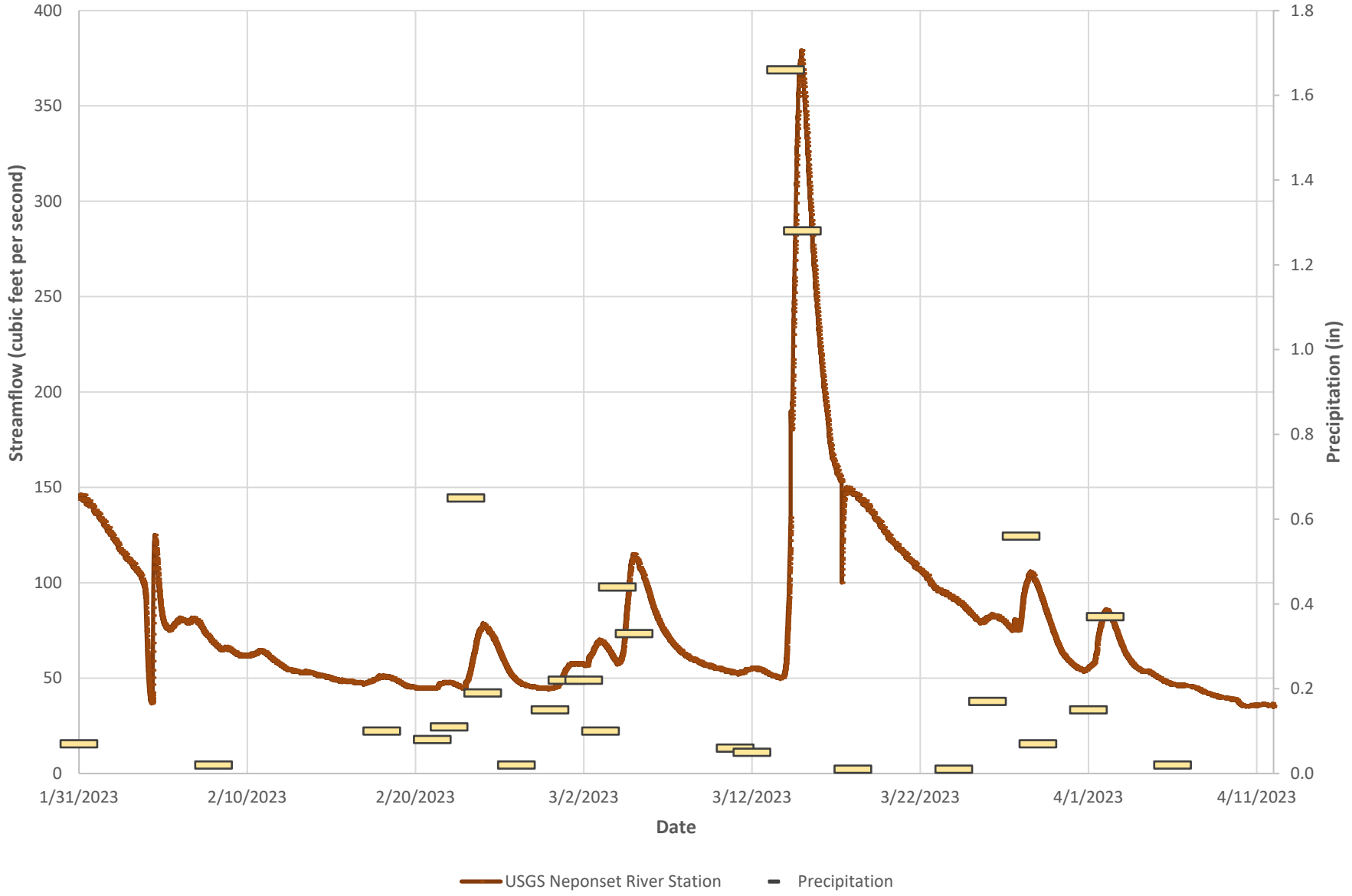


Figure 10. Sharon Great Cedar Swamp Drainage Ditch Flow Compared to USGS Neponset River in Canton, MA Streamflow

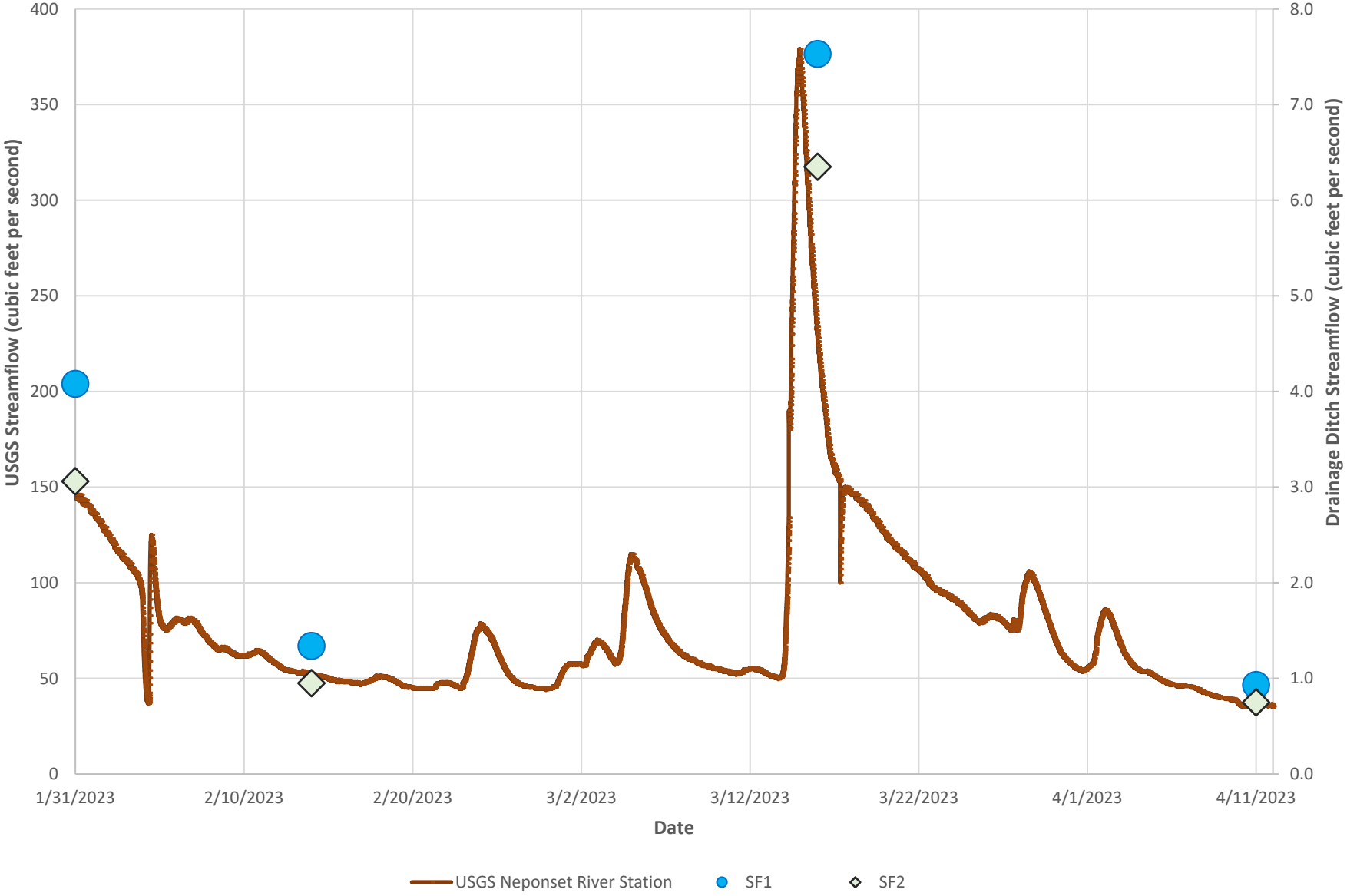


Figure 11. Calculated Sharon Great Cedar Swamp Drainage Ditch Flow at HW-SF1 Compared to USGS Neponset River in Canton, MA Streamflow

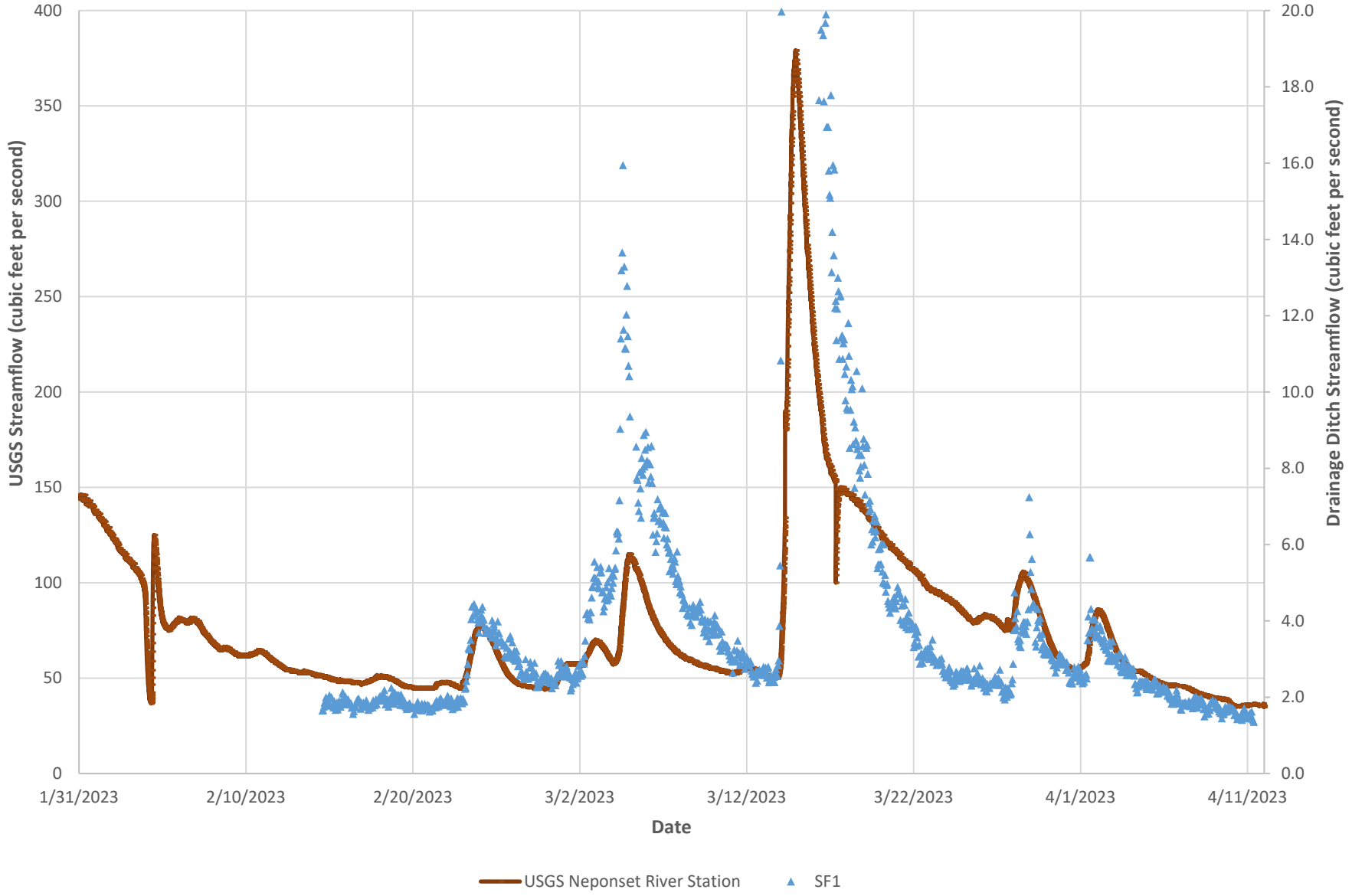
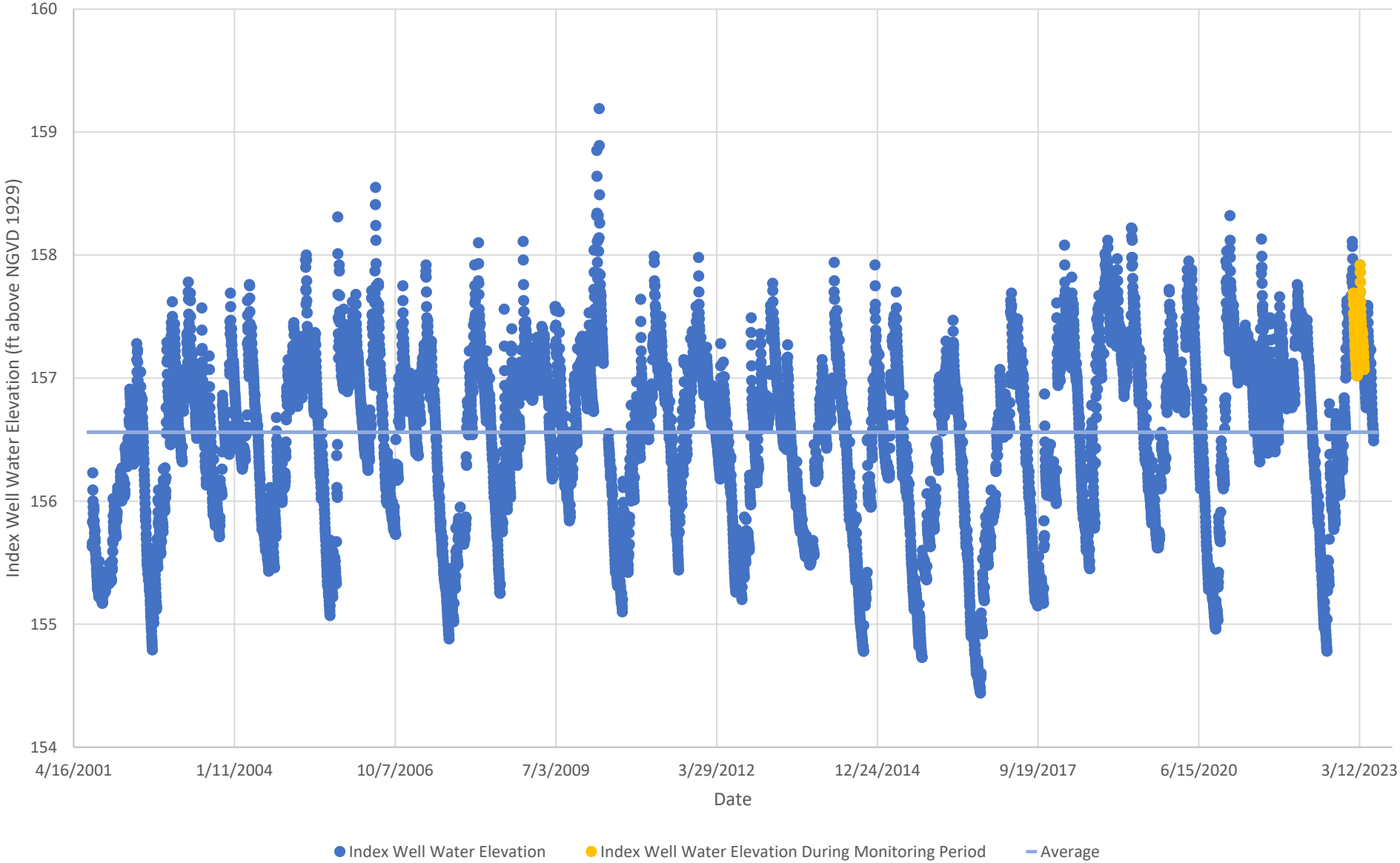
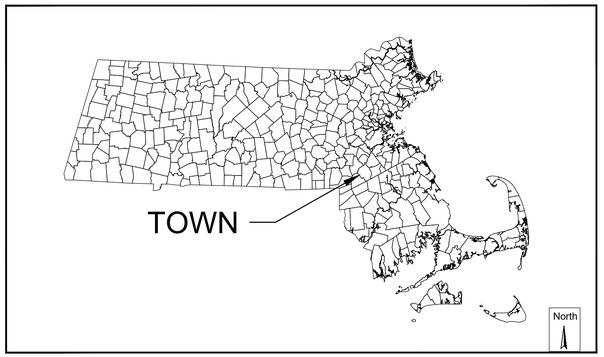


Figure 12: Index Well Water Elevation
USGS 420544071173701 MA-NNW 27R NORFOLK, MA



Appendix E: 75% Design Plan Set

WETLAND RESTORATION
GREAT CEDAR SWAMP
SHARON, MASSACHUSETTS
JUNE 2024



MASSACHUSETTS
Graphic Scale
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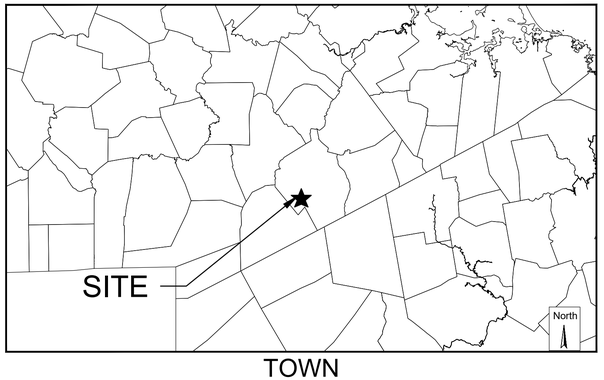
SCALE IN FEET
1:150000



TOWN
Graphic Scale
0 12000
SCALE IN FEET
1:12000



VICINITY MAP
Graphic Scale
1-inch = 1000-feet

Sheet List Table	
Sheet Number	Sheet Title
1	COVER
2	CONSTRUCTION NOTES
3	EXISTING CONDITIONS PLAN
4	LONGITUDINAL PROFILE & TRANSECT PLAN
5	SITE PREPARATION PLAN
6	WATER CONTROL STRUCTURE DETAIL PLAN
7	DETAILS

GENERAL NOTES:
1. THIS PLAN SET IS FOR PERMITTING ONLY AND NOT FOR CONSTRUCTION.
2. SITE INFORMATION:
   - PLAT: 49
   - LOT: 6
   - ADDRESS: 154R WOLOMOLOPOAG STREET
   - ZONING DISTRICT: RURAL DISTRICT 1

Plan Set:
WETLAND RESTORATION
GREAT CEDAR SWAMP
SHARON, MASSACHUSETTS

Prepared For:
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Date Issued: JUNE 2024
Designed By: JDP/JMP
Drawn By: JDP
Checked By: NP
Registration: DRAFT NOT FOR CONSTRUCTION

Revisions

Rev.	Date	By	Appr.	Description
1				
2				
3				
4				
5				
6				
7				

Project Number: 10118F
Sheet Number: 1 of 7
Drawing Number: C - 1

last modified: 06/27/24 printed: 06/27/24 by jp H:\Projects\20101\0118 DER-Great Cedar Swamp-Sharon\Drawings -10118\10118F\10118F CV.dwg

GENERAL CONSTRUCTION NOTES:

1. ALL SITE WORK TO COMPLETE THIS PROJECT, AS INDICATED ON THE DRAWINGS, IN THE SPECIFICATIONS AND AS DESCRIBED BELOW IS THE SOLE RESPONSIBILITY OF THE CONTRACTOR.
2. IMMEDIATELY CONTACT AND COORDINATE WITH THE ENGINEER AND OWNER IF ANY DEVIATION OR ALTERATION OF THE WORK PROPOSED ON THESE DRAWINGS IS REQUIRED.
3. UTILIZE ALL PRECAUTIONS AND MEASURES TO ENSURE THE SAFETY OF THE PUBLIC, ALL PERSONNEL AND PROPERTY DURING CONSTRUCTION IN ACCORDANCE WITH OSHA STANDARDS, INCLUDING THE INSTALLATION OF TEMPORARY FENCING BARRICADES, SAFETY LIGHTING, CONES, POLICE DETAIL AND/OR FLAGMEN AS DETERMINED NECESSARY BY THE TOWN OF SHARON. THE CONTRACTOR IS RESPONSIBLE FOR THE COST OF POLICE DETAIL AND FOR COORDINATING WITH THE LOCAL OR STATE POLICE DEPARTMENT FOR ALL REQUIRED POLICE DETAIL.
4. MAKE ALL NECESSARY CONSTRUCTION NOTIFICATIONS AND APPLY FOR AND OBTAIN ALL NECESSARY CONSTRUCTION PERMITS, PAY ALL FEES INCLUDING POLICE DETAILS AND POST ALL BONDS, IF NECESSARY, ASSOCIATED WITH THE SAME, AND COORDINATE WITH THE OWNER AND THE ENGINEER.
5. ALL EXISTING CONDITIONS SHOWN ARE APPROXIMATE AND ARE BASED ON THE BEST INFORMATION AVAILABLE. PRIOR TO THE START OF CONSTRUCTION VERIFY THAT THE PROPOSED IMPROVEMENTS SHOWN ON THE PLANS DO NOT CONFLICT WITH ANY KNOWN EXISTING OR OTHER PROPOSED IMPROVEMENTS. IF ANY CONFLICTS ARE DISCOVERED, NOTIFY THE OWNER AND THE ENGINEER PRIOR TO INSTALLING ANY PORTION OF THE SITE WORK WHICH WOULD BE AFFECTED.
6. MAINTAIN ALL EXISTING UTILITIES IN WORKING ORDER AND FREE FROM DAMAGE DURING THE ENTIRE DURATION OF THE PROJECT. REPAIR ANY DAMAGE TO EXISTING UTILITY LINES OR STRUCTURES INCURRED DURING CONSTRUCTION OPERATIONS AT NO COST TO THE OWNER. THE CONTRACTOR IS RESPONSIBLE FOR ALL COST RELATED TO THE REPAIR OF UTILITIES. EXCAVATION REQUIRED WITHIN THE PROXIMITY OF EXISTING UTILITY LINES MUST BE DONE BY HAND.
7. IMPORT ONLY CLEAN MATERIAL. MATERIAL FROM AN EXISTING OR FORMER 21E SITE AS DEFINED BY THE MASSACHUSETTS CONTINGENCY PLAN 310 CMR 40.0000 WILL NOT BE ACCEPTED .
8. ESTABLISH AND MAINTAIN ALL CONTROL POINTS AND BENCHMARKS DURING CONSTRUCTION INCLUDING BENCHMARK LOCATIONS AND ELEVATIONS AT CRITICAL AREAS. COORDINATE WITH THE ENGINEER THE LOCATION OF ALL CONTROL POINTS AND BENCHMARKS.
9. SITE LAYOUT REQUIRED FOR CONSTRUCTION IS TO BE PROVIDED BY THE CONTRACTOR AND SHOULD BE PERFORMED BY A MASSACHUSETTS' REGISTERED PROFESSIONAL LAND SURVEYOR. THE CONTRACTOR IS RESPONSIBLE FOR COORDINATING WITH THE SURVEYOR FOR ALL SURVEY LAYOUT. IF THE CONTRACTOR CHOOSES TO USE GPS, THEY ASSUME THE LIABILITY FOR THE ACCURACY OF ALL HORIZONTAL AND VERTICAL LAYOUT.
10. MAINTAIN ALL GRADE STAKES SET BY THE SURVEYOR. GRADE STAKES ARE TO REMAIN UNTIL A FINAL INSPECTION OF THE ITEM HAS BEEN COMPLETED BY THE ENGINEER. RE-STAKING OF PREVIOUSLY SURVEYED SITE FEATURES IS THE RESPONSIBILITY (INCLUDING COST) OF THE CONTRACTOR.
11. UNLESS OTHERWISE INDICATED ON THE DRAWINGS AND/OR IN THE SPECIFICATIONS, ALL SITE CONSTRUCTION MATERIALS AND METHODOLOGIES ARE TO CONFORM TO THE MOST RECENT VERSION OF THE MASSACHUSETTS DEPARTMENT OF TRANSPORTATION STANDARD SPECIFICATIONS .
12. PROVIDE ALL CONSTRUCTION SERVICE IN ACCORDANCE WITH APPLICABLE LAWS AND REGULATIONS REGARDING NOISE, VIBRATION, DUST, SEDIMENTATION CONTAINMENT, AND TRENCH WORK.
13. COLLECT SOLID WASTES AND STORE IN A SECURED DUMPSTER. THE DUMPSTER MUST MEET ALL LOCAL AND STATE SOLID WASTE MANAGEMENT REGULATIONS.
14. RESTORE ALL SURFACES EQUAL TO THEIR ORIGINAL CONDITION AFTER CONSTRUCTION IS COMPLETE PER SPECIFICATIONS. LEAVE ALL AREAS NOT DISTURBED BY CONSTRUCTION IN THEIR NATURAL STATE. TAKE CARE TO PREVENT DAMAGE TO SHRUBS, TREES, OTHER LANDSCAPING AND/OR NATURAL FEATURES. WHEREAS THE PLANS DO NOT SHOW ALL LANDSCAPE FEATURES, EXISTING CONDITIONS MUST BE VERIFIED BY THE CONTRACTOR IN ADVANCE OF THE WORK.
15. PROVIDE A UNIT PRICE COST IN CUBIC YARD MEASURE FOR LEDGE AND/OR BOULDER REMOVAL. LEDGE AND/OR BOULDERS LESS THAN 1 CUBIC YARD IN SIZE BASED ON THE AVERAGE DIMENSIONS WILL NOT BE CONSIDERED PAYABLE ROCK. PROVIDE UNIT PRICES FOR BOTH ON AND OFF SITE DISPOSAL. IF ADDITIONAL FILL MATERIAL IS REQUIRED INCLUDE THE COST OF ALL FILL MATERIAL.
16. REGULARLY INSPECT THE PERIMETER OF THE PROPERTY TO CLEAN UP AND REMOVE LOOSE CONSTRUCTION DEBRIS BEFORE IT LEAVES THE SITE. PROMPTLY REMOVE ALL DEMOLITION DEBRIS FROM THE SITE TO AN APPROVED DUMP SITE.
17. ALL TRUCKS LEAVING THE SITE MUST BE COVERED.
18. DO NOT WASH ANY CONCRETE TRUCKS ONSITE. REMOVE BY HAND ANY CEMENT OR CONCRETE DEBRIS LEFT IN THE DISTURBED AREA.
19. BURIAL OF ANY STUMPS, SOLID DEBRIS, AND/OR STONES/BOULDERS ONSITE IS PROHIBITED. DO NOT USE ROAD SALT OR OTHER DE-ICING CHEMICALS ON THE ACCESS ROADWAY.
20. AT THE END OF CONSTRUCTION, REMOVE ALL CONSTRUCTION DEBRIS AND SURPLUS MATERIALS FROM THE SITE. PERFORM A THOROUGH INSPECTION OF THE WORK PERIMETER. COLLECT AND REMOVE ALL MATERIALS AND BLOWN OR WATER CARRIED DEBRIS FROM THE SITE.

GENERAL DEMOLITION NOTES:

THIS PLAN SET DOES NOT INCLUDE DETAILS & SPECIFICATIONS FOR ALL DEMOLITION WORK REQUIRED WITHIN THE PROPOSED CONSTRUCTION LIMITS. IT IS THE CONTRACTOR'S RESPONSIBILITY TO COORDINATE WITH THE OWNER, AND OTHER PROJECT ENGINEERS INVOLVED WITH THE PROPOSED NEW CONSTRUCTION TO DEVELOP A SUITABLE DEMOLITION PLAN, WHICH WILL ALLOW THE STREAM TO REMAIN IN OPERATION DURING THE ENTIRETY OF CONSTRUCTION.

1. UNLESS OTHERWISE NOTED, THE CONTRACTOR IS RESPONSIBLE FOR THE RELOCATION, DEMOLITION, REMOVAL AND DISPOSAL, IN A LOCATION APPROVED BY ALL GOVERNING AUTHORITIES, OF ALL EXISTING SITE ELEMENTS AND STRUCTURES INCLUDING, BUT NOT LIMITED TO, GRAVEL, TREES, SHRUBS, AND ALL OTHER STRUCTURES SHOWN AND NOT SHOWN WITHIN CONSTRUCTION LIMITS, AND WHERE NEEDED, TO ALLOW FOR NEW CONSTRUCTION. ALL FACILITIES TO BE REMOVED ARE TO BE UNDERCUT TO SUITABLE MATERIAL AND BROUGHT TO GRADE WITH SUITABLE COMPACTED FILL MATERIAL PER SPECIFICATIONS.
2. PRIOR TO DEMOLITION OCCURRING, ALL EROSION CONTROL DEVICES ARE TO BE INSTALLED.
3. REMOVE ALL DEBRIS FROM THE SITE AND DISPOSE OF THE DEBRIS IN A PROPER AND LEGAL MANNER.
4. OBTAIN ALL PERMITS REQUIRED FOR DEMOLITION AND DISPOSAL.
5. MAINTAIN CONTINUOUS ACCESS AND OPERATION FOR SURROUNDING FACILITIES, AS DEEMED BY THE OWNER, AT ALL TIMES DURING DEMOLITION OF THE EXISTING FACILITIES.

BASIC CONSTRUCTION SEQUENCE:

THE FOLLOWING CONSTRUCTION SEQUENCE IS TO BE USED AS A GENERAL GUIDELINE. COORDINATE WITH THE OWNER AND ENGINEERS AND SUBMIT A PROPOSED CONSTRUCTION SEQUENCE FOR REVIEW AND APPROVAL PRIOR TO CONSTRUCTION.

1. SURVEY AND STAKE THE PROPOSED LIMIT OF DISTURBANCE AND LIMIT OF SEDIMENTATION BARRIERS.
2. PLACE SEDIMENTATION BARRIERS AS INDICATED ON DRAWINGS AND STAKED OUT IN THE FIELD. UNDER NO CIRCUMSTANCES IS THE LIMIT OF WORK TO EXTEND BEYOND THE SEDIMENTATION BARRIERS/LIMIT OF DISTURBANCE AS INDICATED ON DRAWINGS AS APPROVED BY THE LOCAL CONSERVATION COMMISSION AND DEPARTMENT OF ENVIRONMENTAL PROTECTION (DEP).
3. INSTALL TEMPORARY CONSTRUCTION ENTRANCES IN LOCATIONS INDICATED ON DRAWINGS. NO OTHER ENTRANCES ARE TO BE USED TO GAIN ACCESS TO THE SITE BY ANY CONSTRUCTION OR DELIVERY VEHICLES.
4. BEGIN SITE CLEARING THE SITE AS REQUIRED FOR ACCESS. PROTECT EXISTING ROADWAY, UTILITIES, AND STORM WATER INFRASTRUCTURE WHILE SECURING ACCESS.
5. INSTALL DEWATERING EQUIPMENT AS REQUIRED. PROVIDE DEWATERING PLAN TO ENGINEER FOR APPROVAL PRIOR TO INSTALLING ANY DEWATERING EQUIPMENT OR APPURTENANCES.
6. SURVEY AND STAKE THE LOCATIONS AND ELEVATIONS OF THE WATER CONTROL STRUCTURE AS SHOWN IN THE PLANS.
7. INSTALL THE WATER CONTROL STRUCTURE. THE EXISTING DITCH SHALL BE PROTECTED FROM SEDIMENTATION UNTIL ALL UN-STABILIZED AREAS ARE STABILIZED WITH STONE SUB-BASE OR VEGETATION. INSTALL SEDIMENT BARRIERS/INLET PROTECTION AT ALL POINTS OF ENTRY INTO THE DRAINAGE NETWORK. TAKE PARTICULAR CARE TO PROTECT THE UNDERGROUND STRUCTURES FROM SEDIMENT.
8. FINISH PERMANENT STABILIZATION. COMPLETE PERMANENT STORMWATER MANAGEMENT AREA SEEDING AND PLANTING AFTER THE CONTRIBUTING AREA TO THE BASIN HAS REACHED A MINIMUM OF 80% STABILIZATION AND IS NO LONGER REQUIRED AS A CONSTRUCTION SEDIMENTATION BASIN.
9. SWEEP THE ROADWAYS CONNECTED TO ACCESS ROADS TO REMOVE ALL SEDIMENTS. REPAIR DRAINAGE TO ROADWAYS AS REQUIRED. CLEAN AND FLUSH THE DRAINAGE STRUCTURES AND PIPES AT THE END OF CONSTRUCTION AND REMOVE ALL ACCUMULATED SEDIMENTS IN THE DITCH AS REQUIRED.
10.
11. ENGINEER TO APPROVE THE REMOVAL OF ALL TEMPORARY SOIL EROSION AND SEDIMENTATION CONTROL MEASURES FOLLOWING VEGETATIVE ESTABLISHMENT OF ALL DISTURBED AREAS AND DETERMINE WHEN THE CONTRIBUTING AREA HAS REACHED A MINIMUM OF 80% STABILIZATION.

GENERAL GRADING AND DRAINAGE NOTES:

1. ALL CUT AND FILL SLOPES SHALL BE 3:1 OR FLATTER UNLESS OTHERWISE NOTED.
2. EXISTING GRADE CONTOUR INTERVALS SHOWN AT 1 FOOT.
3. PROPOSED GRADE CONTOUR INTERVALS SHOWN AT 1 FOOT.
4. PROPOSED ELEVATIONS ARE SHOWN TO FINISH GRADE UNLESS NOTED OTHERWISE.

DEWATERING:

1. HIGH WATER TABLE IS ANTICIPATED. IF THE WATER TABLE IS ENCOUNTERED DURING EXCAVATION, TEMPORARILY LOWER THE WATER TABLE AS INDICATED IN THE SPECIFICATIONS.
2. PRIOR TO ANY DEWATERING, THE DEWATERING PLAN MUST BE APPROVED BY THE ENGINEER.
3. IF DEWATERING IS NECESSARY DURING CONSTRUCTION, IMPLEMENT THE PROPER ESC MEASURES ON SITE TO PREVENT EROSION OR SEDIMENT RUNOFF. THESE MEASURES CAN INCLUDE DEWATERING BAGS, TEMPORARY STRAWBALES, SILT FENCES, SILT SOCKS AND/OR OTHER APPROVED DEVICES AS INDICATED IN THE DETAILS.

EROSION & SEDIMENT CONTROL NOTES:

1. PRIOR TO THE START OF CONSTRUCTION, A STORMWATER POLLUTION PREVENTION PLAN (SWPPP) AND NOTICE OF INTENT (NOI) MUST BE PREPARED AND FILED WITH NPDES BY THE CONTRACTOR. REFER TO THE SWPPP REGARDING ALL EROSION CONTROL MATTERS. MAINTAIN A WORKING COPY OF THE SWPPP ONSITE AT ALL TIMES. FOLLOW THE SWPPP PROTOCOL FOR SITE MAINTENANCE, INSPECTIONS AND PROPER DOCUMENTATION UNTIL THE SITE HAS BEEN ACCEPTED BY THE OWNER. AT THE COMPLETION OF THE PROJECT THE CONTRACTOR OR OWNER MUST FILE A NOTICE OF TERMINATION WITH NPDES. IN ACCORDANCE WITH NPDES REGULATIONS, THE COMPLETED SWPPP MUST INCLUDE ALL OF THE SITE EROSION CONTROL DOCUMENTATION, WEEKLY EROSION INSPECTION REPORTS COMPLETED BY THE DESIGNATED SITE PERSONNEL, AND ANY OTHER PERTINENT SITE DOCUMENTATION MUST BE RETAINED FOR A MINIMUM OF 3 YEARS FROM THE DATE OF TERMINATION.
2. DESIGNATE THE SITE CONSTRUCTION FOREMAN AS THE ON-SITE PERSONNEL RESPONSIBLE FOR THE DAILY INSPECTION AND MAINTENANCE OF ALL SEDIMENT AND EROSION CONTROLS AND IMPLEMENTATION OF ALL NECESSARY MEASURES TO CONTROL EROSION AND PREVENT SEDIMENT FROM LEAVING THE SITE.
3. INSTALL ALL EROSION AND SEDIMENT CONTROL (ESC) MEASURES AS INDICATED ON DRAWINGS IN CONSULTATION WITH THE CONSERVATION AGENT, AND ENGINEER BEFORE ANY CONSTRUCTION ACTIVITIES BEGIN. INSPECT, MAINTAIN REPAIR AND REPLACE EROSION CONTROL MEASURES, AS NECESSARY, DURING THE ENTIRE CONSTRUCTION PERIOD OF THE PROJECT. THE SITE PERIMETER EROSION CONTROLS ARE THE DESIGNATED LIMIT OF WORK. INFORM ALL PERSONNEL WORKING ON THE PROJECT SITE THAT NO CONSTRUCTION ACTIVITY IS TO OCCUR BEYOND THE LIMIT OF WORK AT ANY TIME THROUGHOUT THE CONSTRUCTION PERIOD.
4. MAINTAIN A MINIMUM SURPLUS OF 100 FEET OF EROSION CONTROL BARRIER (SILT FENCE, STRAWBALE, &/OR SILT SOCK) AND ONE EXTRA DEWATERING BAG ONSITE AT ALL TIMES.
5. PROTECT THE EXISTING DRAINAGE INFRASTRUCTURE FROM SEDIMENTATION DURING PROJECT CONSTRUCTION UNTIL ACCEPTANCE BY THE OWNER & IN CONFORMANCE WITH THE ORDER OF CONDITIONS.
6. PROTECT THE ADJACENT RESOURCE AREA FROM SEDIMENTATION DURING PROJECT CONSTRUCTION UNTIL ACCEPTANCE BY THE OWNER & IN CONFORMANCE WITH THE ORDER OF CONDITIONS.
7. PROVIDE CONSTRUCTION EXITS AS INDICATED ON DRAWINGS TO SHED DIRT FROM CONSTRUCTION VEHICLE TIRES. CLEAN AND/OR REPLACE THE CRUSHED STONE PAD, AS NECESSARY, TO MAINTAIN ITS EFFECTIVENESS.
8. KEEP THE LIMIT OF CLEARING, GRADING AND DISTURBANCES TO A MINIMUM WITHIN THE PROPOSED AREA OF CONSTRUCTION. PHASE THE SITE WORK IN A MANNER TO MINIMIZE AREAS OF EXPOSED SOIL. IF TREES ARE TO BE CUT ON THE ENTIRE SITE, CLEAR AND GRUB ONLY THOSE AREAS WHICH ARE ACTIVELY UNDER CONSTRUCTION. PROPERLY INSTALL THE SEDIMENTATION CONTROLS PRIOR TO BEGINNING ANY LAND CLEARING ACTIVITY AND/OR OTHER CONSTRUCTION RELATED WORK.
9. MONITOR LOCAL WEATHER REPORTS DURING CONSTRUCTION AND PRIOR TO SCHEDULING EARTHMOVING OR OTHER CONSTRUCTION ACTIVITIES WHICH LEAVE LARGE DISTURBED AREAS UNSTABILIZED. IF INCLEMENT WEATHER IS PREDICTED, USE BEST PROFESSIONAL JUDGEMENT AND GOOD CONSTRUCTION PRACTICES WHEN SCHEDULING CONSTRUCTION ACTIVITIES AND ENSURE THE NECESSARY EROSION CONTROL DEVICES ARE INSTALLED AND FUNCTIONING PROPERLY TO MINIMIZE EROSION FROM ANY IMPENDING WEATHER EVENTS.
10. INSPECT EROSION AND SEDIMENT CONTROL DEVICES AND STABILIZED SLOPES ON A WEEKLY BASIS AND AFTER EACH RAINFALL EVENT OF .25 INCH OR GREATER. REPAIR IDENTIFIED PROBLEMS WITHIN 24 HOURS TO ENSURE EROSION AND SEDIMENT CONTROLS ARE IN GOOD WORKING ORDER. RESET OR REPLACE MATERIALS AS REQUIRED.
11. SURROUND THE PERIMETER OF SOIL STOCKPILES WITH SILT SOCK, SILT FENCE, STRAWBALES, OR A COMBINATION OF SILT FENCE WITH STRAWBALE, AS DETERMINED NECESSARY.
12. DISTURBED AREAS AND SLOPES MUST NOT BE LEFT UNATTENDED OR EXPOSED FOR EXCESSIVE PERIODS OF TIME SUCH AS THE INACTIVE WINTER SEASON. PROVIDE APPROPRIATE STABILIZATION PRACTICES ON ALL DISTURBED AREAS AS SOON AS POSSIBLE BUT NOT MORE THAN 14 DAYS AFTER THE CONSTRUCTION ACTIVITY IN THAT AREA HAS TEMPORARILY OR PERMANENTLY CEASED, REINFORCE TEMPORARY AREAS HAVING A SLOPE GREATER THAN 4:1 WITH EROSION BLANKETS OR APPROVED EQUAL UNTIL THE SITE IS PROPERLY STABILIZED. TEMPORARY SWALES MAY ALSO BE REQUIRED IF DETERMINED NECESSARY IN THE FIELD BY THE ENGINEER.
13. INSTALL A SILT SACK OR APPROVED EQUIVALENT IN EACH EXISTING CATCHBASIN RECEIVING RUNOFF FROM THE SITE. UPON THE INSTALLATION OF EACH NEW CATCH BASIN, INSTALL A SILT SACK OR APPROVED EQUIVALENT. INSPECT SILT SACKS, AFTER EACH SIGNIFICANT STORM EVENT AND REMOVE AND EMPTY AS NEEDED FOR THE DURATION OF THE CONSTRUCTION PERIOD.
14. TEMPORARY SEDIMENTATION BASINS AND SWALES WILL BE CONSTRUCTED ON AN AS-NEEDED BASIS DURING CONSTRUCTION TO AID IN THE CAPTURE OF SITE RUNOFF AND SEDIMENT. IT WILL BE THE RESPONSIBILITY OF THE SITE CONTRACTOR, IN CONSULTATION WITH THE ENGINEER, TO SIZE AND CREATE THESE BASINS IN APPROPRIATE LOCATIONS.
15. CONTAIN ALL SEDIMENT ONSITE. SWEEP ALL EXITS FROM THE SITE AS NECESSARY INCLUDING ANY SEDIMENT TRACKING. SWEEP PAVED AREAS AS NEEDED TO REMOVE SEDIMENT AND POTENTIAL POLLUTANTS ACCUMULATED DURING SITE CONSTRUCTION.
16. PROVIDE ON SITE OR MAKE READILY AVAILABLE THE NECESSARY EQUIPMENT AND SITE PERSONNEL DURING CONSTRUCTION HOURS FOR THE DURATION OF THE PROJECT TO ENSURE ALL EROSION AND SEDIMENTATION CONTROL DEVICES ARE PROPERLY MAINTAINED AND REPAIRED IN A TIMELY AND RESPONSIBLE MANNER. IF SITE WORK IS SUSPENDED DURING THE WINTER MONTHS THE CONTRACTOR MUST CONTINUE TO PROVIDE PERSONNEL AND EQUIPMENT EITHER ON SITE OR READILY AVAILABLE TO PROPERLY MAINTAIN AND REPAIR ALL EROSION AND SEDIMENTATION CONTROL DEVICES IN A TIMELY AND RESPONSIBLE MANNER
17. CONTROL DUST BY WATERING OR OTHER APPROVED METHODS AS NECESSARY, OR AS DIRECTED BY THE ENGINEER.
18. THE CONTRACTOR IS RESPONSIBLE FOR THE INSPECTION AND MAINTENANCE DURING CONSTRUCTION OF ALL STORMWATER FACILITIES INSTALLED OR AFFECTED BY THE PROJECT. REMOVE SEDIMENT OR DEBRIS COLLECTED WITHIN THESE FACILITIES AS REQUIRED FOR PROPER FUNCTION AND CONTROL OF SITE RUNOFF DURING CONSTRUCTION AND PRIOR TO THE OWNER'S ACCEPTANCE.

TOTAL LIMIT OF WORK	17,900	SF	
	Alteration		Notes
Resource Area	Temporary	Permanent	Most alteration will be temporary (i.e. within limit of work).
	(SF)	(SF)	
Bordering Vegetated Wetland (BVW)	0	0	No BVW within the LOW
Bank (linear feet)	100	0	
Land Under Water (area below MHW)	1650	30	
Flood Zone (BLSF)	640	0	
Buffer Zone			
altered from 0-50 feet	17,900	0	
altered from 50-100 feet	0	0	
Total Alteration 0-100 feet	17,900	0	
Riverfront Area			
Total RA altered within 0-100 feet	15,100	0	
Total RA altered within 100-200 feet	0	0	
Total Riverfront Area altered	15,100	0	
	Area	Volume	
Dredging & Filling (Below MLW)	(SF)	(CY)	
Dredging	-42	-5.5	
Filling	42	6.2	
Net Dredge/Fill	0	0.7	

Revisions

Rev.	Date	By	Appr.	Description

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Checked By: MP
Drawn By: JSP
Designed By: JSP/SMP
Date: JUNE 2024

Plan Set: WETLAND RESTORATION
GREAT CEDAR SWAMP
SHARON, MASSACHUSETTS

Plan Title: CONSTRUCTION NOTES

Prepared For:
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Survey Provided By:
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Dated: June 6, 2024

Registration:

DRAFT
NOT FOR CONSTRUCTION

Project Number: 10118F
Sheet : 2 of 7

Sheet Number: C - 2

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SURVEY NOTES

1. THE TOPOGRAPHY AND EXISTING SITE CONDITIONS DEPICTED HEREON ARE THE RESULT OF AN ON THE GROUND FIELD SURVEY CONDUCTED BY THE HORSLEY WITTEN GROUP, INC. APRIL 26, MAY 22, AND JUNE 7, 2024.
2. HORIZONTAL DATUM IS MASS STATE PLANE COORDINATE SYSTEM. DATUM ESTABLISHED BY GNSS OBSERVATIONS.
3. THE ELEVATIONS DEPICTED HEREON WERE BASED ON THE NORTH AMERICAN VERTICAL DATUM (NAVD) OF 1988.
4. NO PROPERTY LINE SURVEY WAS CONDUCTED FOR THIS EXISTING CONDITIONS PLAN.
5. THE PROPERTY LINES, EASEMENTS, AND RIGHTS OF WAYS DEPICTED ARE APPROXIMATE ONLY.
6. THE ACCURACY OF MEASURED PIPE INVERTS AND PIPE SIZES IS SUBJECT TO FIELD CONDITIONS, THE ABILITY TO MAKE VISUAL OBSERVATIONS, DIRECT ACCESS TO THE VARIOUS ELEMENTS AND OTHER CONDITIONS.
7. THE LOCATION AND/OR ELEVATION OF EXISTING UTILITIES AND STRUCTURES AS SHOWN ON THESE PLANS ARE BASED ON RECORDS OF VARIOUS UTILITY COMPANIES, AND WHEREVER POSSIBLE, MEASUREMENTS TAKEN IN THE FIELD. THIS INFORMATION IS NOT TO BE RELIED UPON AS BEING EXACT OR COMPLETE. THE LOCATION OF ALL UNDERGROUND UTILITIES AND STRUCTURES SHALL BE VERIFIED IN THE FIELD PRIOR TO THE START OF ANY CONSTRUCTION. THE CONTRACTOR MUST CONTACT THE APPROPRIATE UTILITY COMPANY, ANY GOVERNING PERMITTING AUTHORITY IN THE TOWN OF SHARON, AND "DIGSAFE" (1-888-344-7233) AT LEAST 72 HOURS PRIOR TO ANY EXCAVATION WORK IN PREVIOUSLY UNALTERED AREAS TO REQUEST EXACT FIELD LOCATION OF UTILITIES.
8. UTILITY PROVIDERS: ELECTRIC - EVERSOURCE TELEPHONE - VERIZON CABLE - COMCAST SEWER - MUNICIPAL WATER - MUNICIPAL DRAINAGE - MUNICIPAL
9. THE PROPERTY IS LOCATED WITHIN F.I.R.M ZONE AE REGULATORY FLOODWAY AS SHOWN ON COMMUNITY PANEL NO. 25021C0354E AND 25021C0352E DATED JULY 17, 2012.
10. PROPERTY LINES AND FEMA F.I.R.M. ZONE SHOWN ARE PROVIDED BY THE BUREAU OF GEOGRAPHIC INFORMATION (MASSGIS), COMMONWEALTH OF MASSACHUSETTS, EXECUTIVE OFFICE OF TECHNOLOGY AND SECURITY SERVICES.
11. THE WETLAND DELINEATION SHOWN HEREON WAS CONDUCTED BY THE HORSLEY WITTEN GROUP, INC. ON APRIL 26 AND JUNE 7, 2024.
12. REFERENCE PLANS:
NORFOLK COUNTY REGISTRY OF DEEDS
PLAN BOOK 5984 PAGE 713

MASS STATE PLANE

200' RIVERFRONT AREA
150R WOLOMOLOPOAG STREET
N/F
WAGNER TRUSTEE
100' RIVERFRONT AREA
APPROX. PROPERTY LINE
APPROX. EASEMENT
FLOW
TRANSECT
BORDERING VEGETATED WETLAND
154R WOLOMOLOPOAG STREET
N/F
TOWN OF SHARON
LONGITUDINAL PROFILE
127 WOLOMOLOPOAG STREET
N/F
SALEM
CONTROL POINT
SPK SET
N: 2859504.25
E: 735992.03
Z: 258.81
EDGE OF WATER
RIP RAP PILE
BORDERING VEGETATED WETLAND
MEAN ANNUAL HIGH WATER
RIP RAP BAFFLE
TOP EL. 250.97'
EDGE OF BANK
SIDE CHANNEL
CONTROL POINT
SPK SET
N: 2859335.61
E: 735923.33
Z: 258.36
18" DEAD
24" OAK
34" PINE
PATH
133 WOLOMOLOPOAG STREET
N/F
THIBEAULT
RIM: 257.17'
12" HDPE INV. OUT: 254.17
12" HDPE INV. OUT: 253.87'
RIM: 257.09'
PIPES NOT VISIBLE
SUMP: 250.79'
100' RESOURCE AREA BUFFER ZONE
50' RESOURCE AREA BUFFER ZONE
OUTFALL
FEMA REGULATORY FLOODWAY
UP 141/12
RIM: 258.01'
18" RCP INV. OUT: 253.21'
RIM: 257.35'
DROP INLET
WOLOMOLOPOAG STREET
OHW
12"HDPE INV. OUT: 253.68'
36" RCP INV. IN: 245.85'
30" RCP INV. IN: 246.07'
RETAINING WALL
RIM: 257.24'
18" RCP INV. OUT: 253.54'
FEMA REGULATORY FLOODWAY
154 WOLOMOLOPOAG STREET
N/F
LEVIS
APPROX. BUILDING LOCATION
BASEMENT EL. 254.59'
SUMP EL. 253.87'
139 WOLOMOLOPOAG STREET
N/F
SHULTZ
156 WOLOMOLOPOAG STREET
N/F
CAULFIELD

LEGEND:

GENERAL	SYMBOLS
BUILDING	CONTROL POINT
CONTOUR - MINOR	STAFF GAUGE
CONTOUR - MAJOR	EXISTING TREE
CURB	DRAIN MANHOLE
EDGE OF PAVEMENT	CATCHBASIN
PATH	UTILITY POLE
RIP RAP	GUY
TREE LINE	WETLAND FLAG
RETAINING WALL	ROCK
PROPERTY INFORMATION	
EASEMENT LINE	
PROPERTY, LOT, OR ROW	
UTILITIES	
DRAIN PIPE	
OVERHEAD WIRE	
ENVIRONMENTAL	
BORDERING VEGETATED WETLAND	
RESOURCE AREA 50' BUFFER ZONE	
RESOURCE AREA 100' BUFFER ZONE	
MEAN ANNUAL HIGH WATER	
EDGE OF BANK	
100' RIVERFRONT AREA	
200' RIVERFRONT AREA	
EDGE OF WATER	
FEMA FLOOD ZONE	

GRAPHIC SCALE
30 0 15 30 60 120
(in feet)
1 INCH = 30 FEET

Revisions

Rev.	Date	By	Appr.	Description

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Drawn By: JBP
Designed By: JBP/MP
Date: JUNE 2024

WETLAND RESTORATION
GREAT CEDAR SWAMP
SHARON, MASSACHUSETTS

Plan Set:

EXISTING CONDITIONS PLAN

Plan Title:

Prepared For:
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Survey Provided By:
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Dated: June 6, 2024

Registration:

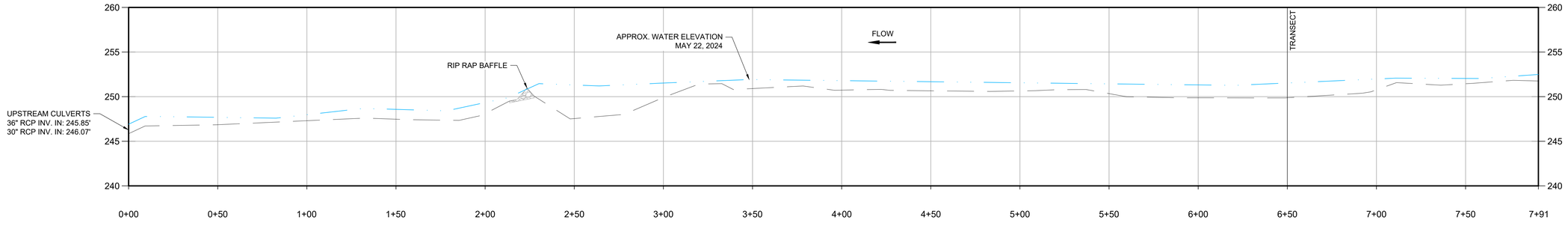
DRAFT
NOT FOR CONSTRUCTION

Project Number: 10118F
Sheet : 3 of 7
Sheet Number: C - 3

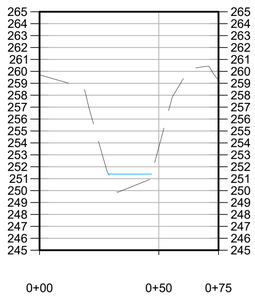
LEGEND - LONGITUDINAL PROFILE/TRANSECT CROSS SECTION

RIVER BED

WATER LEVEL EL. (MAY 22, 2024)



LONGITUDINAL PROFILE
HORIZONTAL SCALE: 1" = 30'
VERTICAL SCALE: 1" = 6'



CROSS SECTION
HORIZONTAL SCALE: 1" = 30'
VERTICAL SCALE: 1" = 6'

GRAPHIC SCALE

30 0 15 30 60 120

(in feet)

1 INCH = 30 FEET

Revisions

Rev.	Date	By	Appr.	Description

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Drawn By: JBP

Designed By: JBP/MP

Date: JUNE 2024

Plan Set:

WETLAND RESTORATION
GREAT CEDAR SWAMP
SHARON, MASSACHUSETTS

Plan Title:

LONGITUDINAL PROFILE & TRANSECT PLAN

Prepared For:

Massachusetts Division of Ecological Restoration
100 Cambridge Street, Floor 6
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Survey Provided By:

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Dated: June 6, 2024

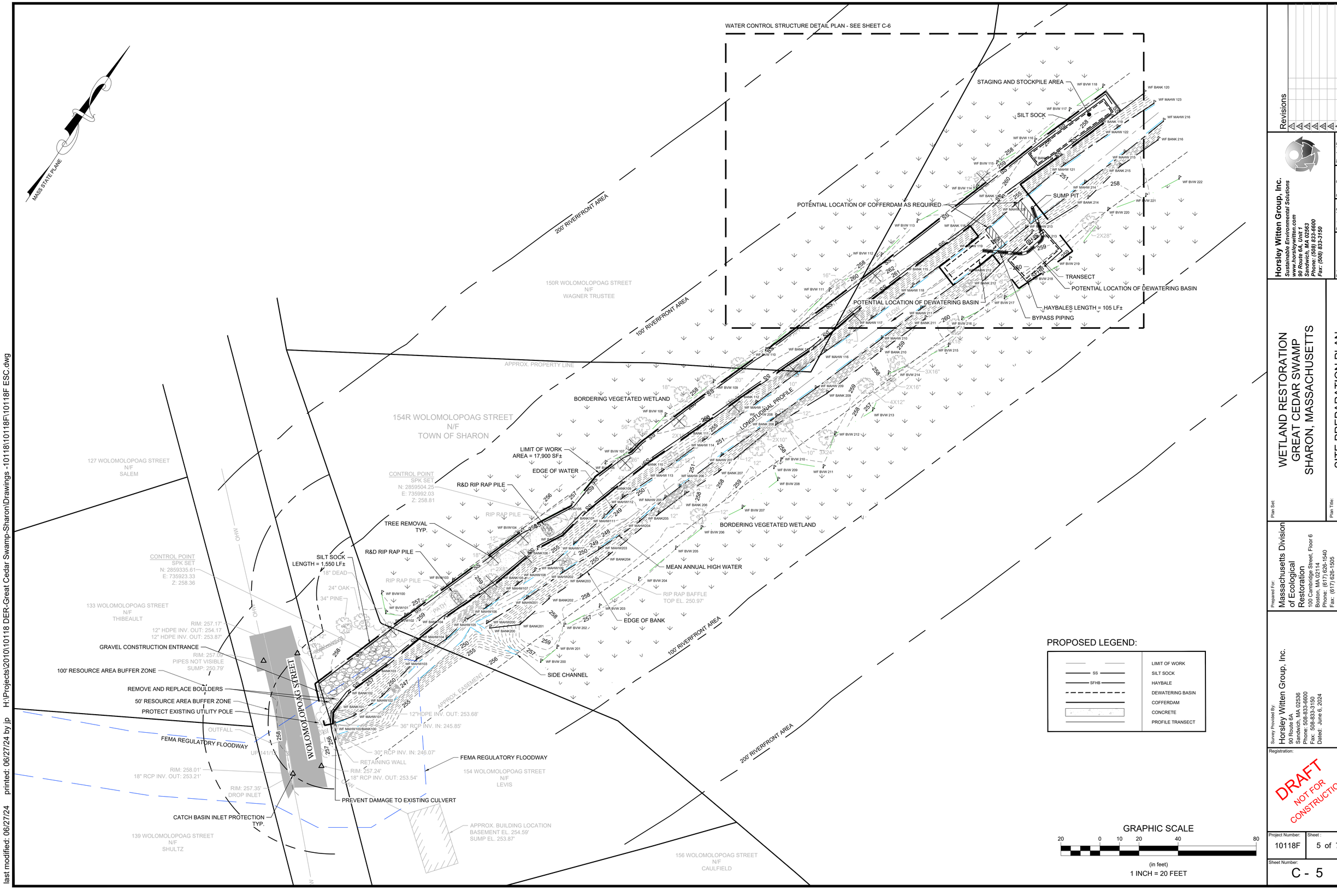
Registration:

DRAFT
NOT FOR CONSTRUCTION

Project Number: 10118F

Sheet : 4 of 7

Sheet Number: C - 4



Revisions

Rev.	Date	By	Appr.	Description

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Designed By: JSP/SMF
Date: JUNE 2024

Plan Set: WETLAND RESTORATION
GREAT CEDAR SWAMP
SHARON, MASSACHUSETTS

Plan Title: SITE PREPARATION PLAN

Prepared For:
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Dated: June 6, 2024

Registration:

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Project Number: 10118F
Sheet: 5 of 7

Sheet Number: C - 5

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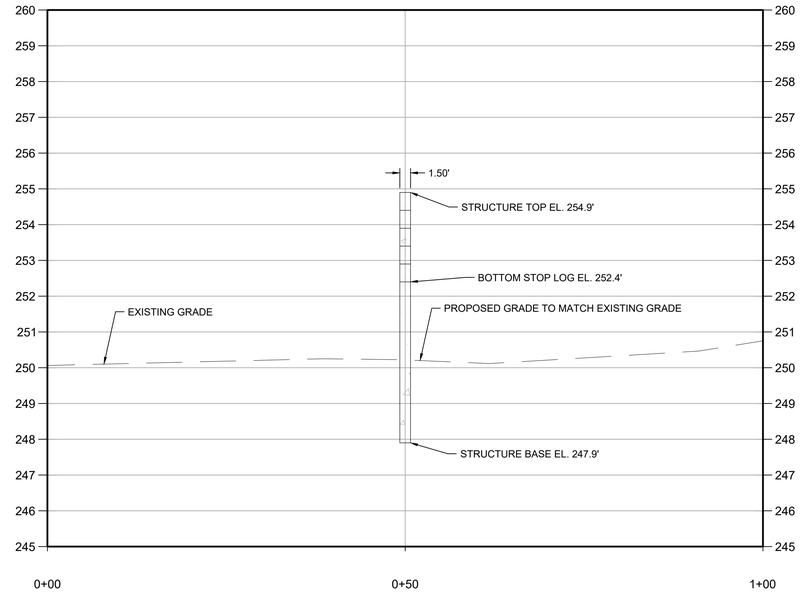
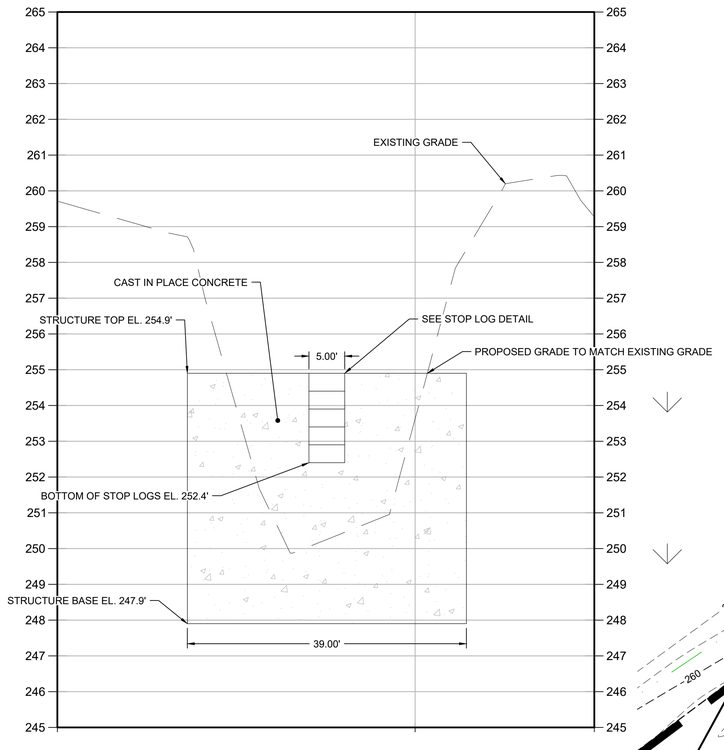


PROPOSED LONGITUDINAL PROFILE
HORIZONTAL SCALE: 1" = 10'
VERTICAL SCALE: 1" = 2'



PROPOSED CROSS SECTION
HORIZONTAL SCALE: 1" = 10'
VERTICAL SCALE: 1" = 2'



Revisions

Rev.	Date	By	Appr.	Description

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Designed By: JSP/SMP
Date: JUNE 2024

Plan Set:
WETLAND RESTORATION
GREAT CEDAR SWAMP
SHARON, MASSACHUSETTS

Plan Title:
WATER CONTROL STRUCTURE DETAIL PLAN

Prepared For:
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Registration:

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Project Number: 10118F
Sheet: 6 of 7
Sheet Number: C - 6

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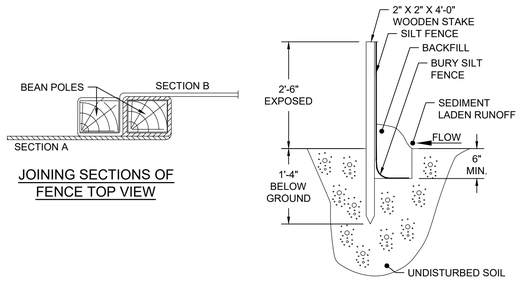
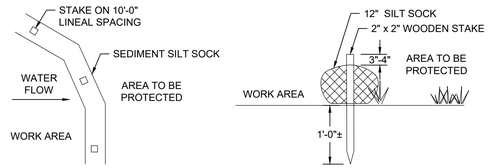


JOINING SECTIONS OF FENCE TOP VIEW

NOTES:
1. FENCE FABRIC WILL BE 3'-0" WIDE MINIMUM & WILL BE SECURELY FASTENED TO BEANPOLES. 6" MINIMUM OF FABRIC WILL BE BELOW GROUND SURFACE (BACKFILLED OR TOE-ED INTO GROUND).
2. WOODEN STAKE POST SPACING WILL BE 4'-0" MAXIMUM ON CENTER.

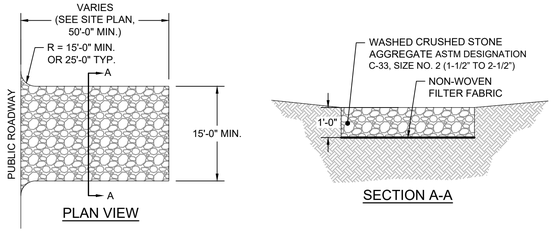
SILT FENCE
NOT TO SCALE



PLAN VIEW SECTION

NOTES:
1. SILT SOCK: SILTSOXX ORIGINAL, EXTREME, NATURAL ORIGINAL, NATURAL ORIGINAL PLUS+ WITH ALL NATURAL BIODEGRADEABLE COTTON FIBER OR MULTI-FILAMENT POLYPROPYLENE (MFPP) MANUFACTURED BY FILTREXX (WWW.FILTREX.COM) OR APPROVED EQUIVALENT.
2. ALL MATERIAL TO MEET MANUFACTURER'S SPECIFICATIONS.
3. SEDIMENT SILT SOCK TO BE FILLED WITH LOCALLY SOURCED FILTER MEDIA (LEAF COMPOST AND/OR WOODY MULCH) PER MANUFACTURER'S REQUIREMENTS.
4. FOLLOWING CONSTRUCTION AND SITE STABILIZATION, COMPOST MATERIAL TO BE REMOVED OR DISPERSED ON SITE, AS APPROVED BY THE OWNER AND/OR ENGINEER.

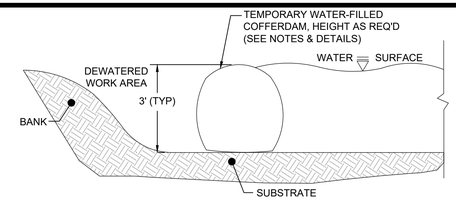
SEDIMENT SILT SOCK
NOT TO SCALE



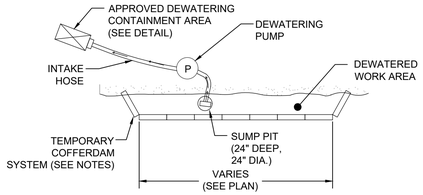
PLAN VIEW SECTION A-A

NOTES:
1. STONE CONSTRUCTION ENTRANCE(S) WILL REMAIN UNTIL INSTALLATION OF PAVEMENT SUB-BASE IS SET TO BEGIN.
2. SEE SITE PLAN FOR LOCATION & ACTUAL DIMENSIONS

STONE CONSTRUCTION ENTRANCE
NOT TO SCALE



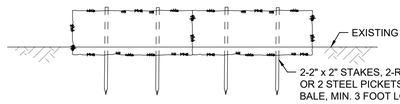
TEMPORARY COFFERDAM INSTALLATION DETAIL



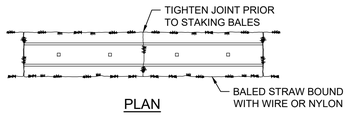
NOTE: DETAIL APPLICABLE FOR WORK ON EITHER SIDE OF MARSH. MIRROR DETAIL AS APPLICABLE.

DEWATERING PUMP PLAN

NOTES:
1. THE DEWATERING PLAN, INCLUDING THE TEMPORARY COFFERDAM SYSTEM TO BE USED, SHALL BE APPROVED BY THE ENGINEER. WATER-INFLATED BARRIERS BY AQUA-BARRIER OR OTHER APPROVED MANUFACTURERS ARE ACCEPTABLE ALTERNATIVES.
2. THE TEMPORARY COFFERDAM SYSTEM SHALL BE INSTALLED ACCORDING TO THE MANUFACTURER RECOMMENDATIONS, BEGINNING AT THE MOST UPSTREAM LOCATION.
3. ANY SANDBAGS USED IN THE DEWATERING PROCESS SHALL CONSIST OF MATERIALS WHICH ARE RESISTANT TO ULTRA-VIOLET RADIATION, TEAR AND PUNCTURE AND WOVEN TIGHTLY ENOUGH TO PREVENT LEAKAGE OF FILL MATERIAL (I.E. SAND, FINE GRAVEL).
4. ALL DEWATERING CONTAINMENT AREAS AND EROSION/SEDIMENT CONTROL DEVICES SHALL BE INSTALLED PRIOR TO ANY COFFERDAM INSTALLATION AND ACCORDING TO THE CONSTRUCTION PLANS AND DETAILS.
5. ALL EXCAVATED SEDIMENTS OR DEBRIS SHALL BE DISPOSED OF IN AN APPROVED STOCK PILE AREA AND PROTECTED WITH EROSION/SEDIMENT CONTROL BARRIERS.
6. EROSIONS/SEDIMENT CONTROL DEVICES ARE TO REMAIN IN PLACE UNTIL ALL DISTURBED AREAS ARE STABILIZED IN ACCORDANCE WITH THE SPECIFICATIONS.



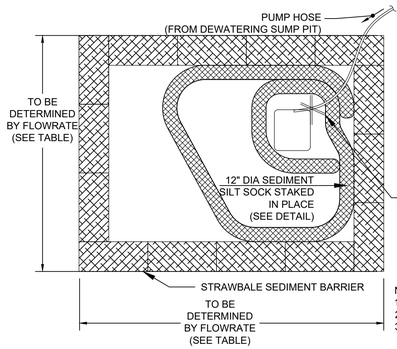
SECTION



PLAN

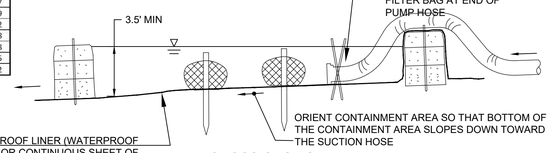
NOTES:
1. INSTALLATION LOCATION PER THE SITE PLAN.
2. EMBED BALES 4" TO 6".
3. OVERLAP BALES MINIMUM 6" IN SWALES.

HAYBALE
NOT TO SCALE



Flow Rate		Required Surface Area	Length/Width=2:1	
Q (gpm)	Q (ft^3/s)	As (ft^2)	L (ft)	W (ft)
25	0.0565	131.86	16.24	8.14
50	0.1130	263.82	22.97	11.48
100	0.2225	527.54	32.48	16.24
150	0.3355	791.36	39.80	19.88
200	0.4450	1055.19	45.93	22.97
250	0.5580	1318.90	51.35	25.69
300	0.6674	1582.73	56.27	28.12
350	0.7805	1846.44	60.76	30.38
400	0.8899	2110.26	64.96	32.48
450	1.0029	2374.09	68.90	34.45
500	1.1124	2637.80	72.64	36.32

PLAN VIEW

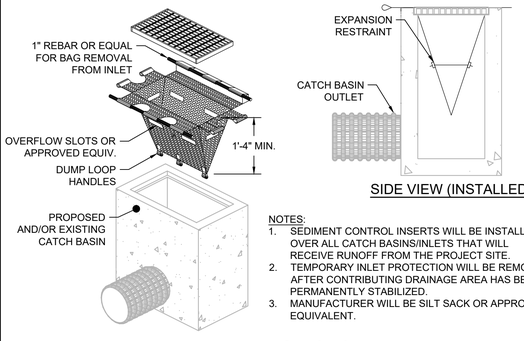


CROSS-SECTION

NOTE:
1. NUMBER OF BALES MAY VARY DEPENDING ON SITE CONDITIONS.
2. THE BASIN TO BE SIZED TO PREVENT WATER FROM OVERTOPPING THE STRAWBALES.
3. INLET HOSE INTAKE TO NOT SETTLE TO THE BOTTOM OF THE EXCAVATION AND SHOULD BE BE INSTALLED IN AN EXCAVATED SUMP PIT FILLED WITH CRUSHED STONE.

DEWATERING CONTAINMENT AREA DETAIL
NOT TO SCALE

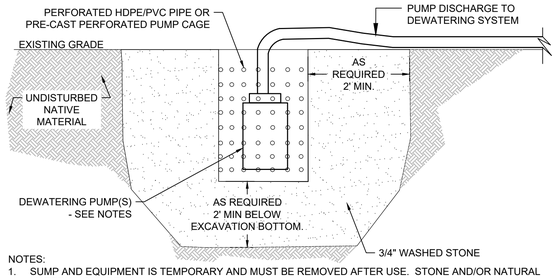
TEMPORARY COFFERDAM INSTALLATION
NOT TO SCALE



SIDE VIEW (INSTALLED)

NOTES:
1. SEDIMENT CONTROL INSERTS WILL BE INSTALLED OVER ALL CATCH BASINS/INLETS THAT WILL RECEIVE RUNOFF FROM THE PROJECT SITE.
2. TEMPORARY INLET PROTECTION WILL BE REMOVED AFTER CONTRIBUTING DRAINAGE AREA HAS BEEN PERMANENTLY STABILIZED.
3. MANUFACTURER WILL BE SILT SACK OR APPROVED EQUIVALENT.

INLET PROTECTION
NOT TO SCALE



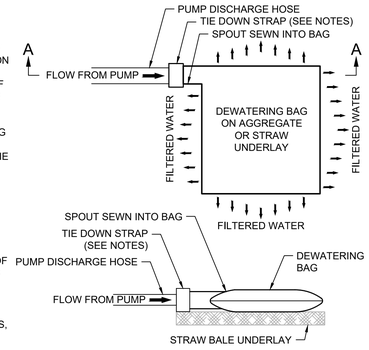
NOTES:
1. SUMP AND EQUIPMENT IS TEMPORARY AND MUST BE REMOVED AFTER USE. STONE AND/OR NATURAL MATERIAL CAN REMAIN.
2. LOCATE PUMP OUTSIDE OF THE MAIN EXCAVATION AREA TO MINIMIZE SEDIMENTATION.
3. SIZE PUMP TO SUFFICIENTLY DEWATER EXCAVATION. MULTIPLE PUMPS TO BE INSTALLED AS NECESSARY.
4. PUMP CHAMBER SIZE TO BE DETERMINED BASED ON PUMP DIMENSIONS.
5. FINAL DEPTH TO BE DETERMINED BY CONTRACTOR BASED ON EXISTING GRADE AND DEPTH OF WATER.

DEWATERING SUMP
NOT TO SCALE

INSTALLATION NOTES:
1. PLACE LIFTING STRAPS (NOT INCLUDED) UNDER THE UNIT TO FACILITATE REMOVAL AFTER USAGE
2. UNFOLD DANDY DEWATERING BAG ON STABILIZED AREA OVER DENSE VEGETATION, STRAW, OR GRAVEL (IF AN INCREASED DRAINAGE SURFACE IS NEEDED)
3. INSERT DISCHARGE HOSE FROM PUMP INTO DANDY DEWATERING BAG A MINIMUM OF SIX INCHES AND TIGHTLY SECURE WITH ATTACHED TIE DOWN STRAP TO PREVENT WATER FROM FLOWING OUT OF THE UNIT WITHOUT BEING FILTERED
4. IF USING OPTIONAL ABSORBENTS, PLACE ABSORBENT BOOM INTO THE DANDY DEWATERING BAG

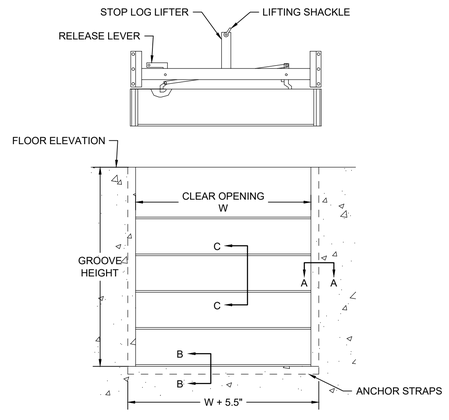
MAINTENANCE:

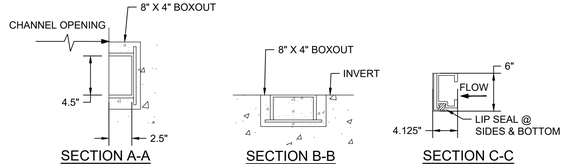
5. REPLACE THE UNIT WHEN 1/2 FULL OF SEDIMENT OR WHEN SEDIMENT HAS REDUCED THE FLOW RATE OF THE PUMP DISCHARGE TO AN IMPRACTICAL RATE
6. IF USING OPTIONAL OIL ABSORBENTS, REMOVE AND REPLACE ABSORBENT PILLOW WHEN NEAR SATURATION



SECTION A - A

DEWATERING BAG
NOT TO SCALE





NOTES:
1. DIMENSIONS MAY VARY PER MANUFACTURER

STOP LOGS
NOT TO SCALE

Revisions

Rev.	Date	By	Appr.	Description

Horsley Witten Group, Inc.
Sustainable Environmental Solutions
www.horsleywitten.com
90 Route 6A, Unit 1
Sandwich, MA 02563
Phone: (508) 833-6600
Fax: (508) 833-3150

Checked By: MP
Drawn By: JDP
Designed By: JDP/SMP
Date: JUNE 2024

WETLAND RESTORATION
GREAT CEDAR SWAMP
SHARON, MASSACHUSETTS

Plan Set:

DETAILS

Plan Title:

Prepared For:
Massachusetts Division of Ecological Restoration
100 Cambridge Street, Floor 6
Boston, MA 02114
Phone: (617) 626-1540
Fax: (617) 626-1505

Survey Provided By:
Horsley Witten Group, Inc.
90 Route 6A
Sandwich, MA 02536
Phone: 508-833-6600
Fax: 508-833-3150
Dated: June 6, 2024

Registration:

DRAFT NOT FOR CONSTRUCTION

Project Number: 10118F

Sheet : 7 of 7

Sheet Number: C - 7

last modified: 06/27/24 printed: 06/27/24 by jp H:\Projects\2010\10118 DER-Great Cedar Swamp-Sharon\Drawings -10118\10118F\10118F DE.dwg

Appendix F: Engineer's Opinion of Probable Cost

Horsley Witten Group
Sustainable Environmental Solutions
90 Route 6A • Unit 1 • Sandwich, MA 02563
508-833-6600 • horsleywitten.com

Project: GCS Flow Control Structure
Location: Sharon, MA

Submission: Permit-Level Design
Date: 6/30/2024

Estimator: MD **Checked By:** NP

GENERAL SITEWORK	Unit	Quantity	Unit Cost	Total Cost
SITE PREPARATION & EROSION CONTROL				
Site Mobilization / Demobilization	LS	1	$ 20,000.00	**$ 20,000.00**
Tree Removal	EA	14	$ 2,750.00	**$ 38,500.00**
Gravel Construction Entrance	CY	28	$ 120.00	**$ 3,333.33**
Silt Sock Erosion Control	LF	1,720	$ 8.20	**$ 14,104.00**
Haybales	LF	105	$ 8.20	**$ 861.00**
General site access improvement	LS	1.00	$ 5,000.00	**$ 5,000.00**
Dewatering & Flow Control	LS	1	$ 30,000.00	**$ 30,000.00**
Remove two excess rip rap piles along berm and transport to Town DPW facility	LS	1	$ 3,000.00	**$ 3,000.00**
Protect existing utility pole	EA	1	$ 450.00	**$ 450.00**
Catchbasin inlet protection with silt sacks	EA	4.0	$ 250.00	**$ 1,000.00**
Subtotal Site Preparation & Erosion Control			**$ 116,248.33**	
FLOW CONTROL STRUCTURE				
Cast-in-place Concrete	CY	15	$ 5,500.00	**$ 79,750.00**
Aluminum stop logs	LS	1	$ 15,750.00	**$ 15,750.00**
Stop log lifting beam (optional)	LS	1	$ 5,950.00	**$ 5,950.00**
Total Flow Control Structure			**$ 101,450.00**	
POST-CONSTRUCTION RESTORATION				
Seeding	SY	222	$ 4.00	**$ 888.80**
Repair slope after construction to match existing grade	CY	36	$ 71.29	**$ 2,574.99**
			$ 3,463.79	
TOTAL GENERAL SITEWORK			**$ 221,200.00**	

TOTAL ESTIMATED CONSTRUCTION COST		**$ 221,200.00**
General Conditions	10%	$ 22,200.00
ESTIMATED CONSTRUCTION COSTS (Including General Conditions)		**$ 243,400.00**
Construction Overhead and Profit	15%	$ 36,600.00
Contingency	15%	$ 36,600.00
TOTAL ESTIMATED CONSTRUCTION COSTS		**$ 316,600.00**
ALLOWANCES		
None		
TOTAL ESTIMATED CONSTRUCTION COSTS INCLUDING ALLOWANCES		**$ 316,600.00**
RANGE (-15% TO +20%)		
Low	**-15%**	**$ 269,200.00**
High	**20%**	**$ 380,000.00**

Qualifications:
The following items are not included in the scope of work:

Preparation of NPDES SWPPP
Soil Management Plan
Contaminated Soil

Quantities provided are based on permit-level design plans "Wetland Restoration Great Cedar Swamp Sharon, Massachusetts" June 2024, Horsley Witten Group

Unit prices provided are based upon typical 2024 construction costs and data. Unit prices are subject to change due to adjustments to material and labor costs, site conditions and inflation.